

Advances in Concrete Technology

Proceedings

Second CANMET/ACI International Symposium
Las Vegas, Nevada, USA, 1995



Editor

V. M. Malhotra



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PREFACE

The Canada Centre for Mineral and Energy Technology (CANMET) of Natural Resources Canada, Ottawa, has played a significant role in Canada over thirty years in the broad area of concrete technology. In recent years, CANMET has become involved increasingly in research and development dealing with the supplementary cementing materials, high-performance normal-weight and lightweight concretes and alkali-aggregate reactions. As a part of CANMET's technology transfer program, an international symposium on Advances in Concrete Technology was sponsored jointly with the American Concrete Institute and other organizations, and held in Athens in May 1992.

In June 1995, CANMET, in association with the American Concrete Institute and other organizations sponsored the Second CANMET/ACI International Symposium on Advances in Concrete Technology in Las Vegas, U.S.A. The main purpose of the symposium was to bring together representatives from industry, universities, and government agencies to present the latest information in the subject area of the symposium, and to explore new areas of needed research and development.

More than 48 papers from 20 countries were received and reviewed in accordance with the policies of the American Concrete Institute. Thirty-two of the refereed papers were accepted for presentation and publication. In addition to the refereed papers, 45 nonrefereed papers were presented at the symposium, and were published in a supplementary volume for distribution at the symposium.

Thanks are extended to the technical review panel which met in Las Vegas in January 1995, to review the papers. Without their prompt review and constructive comments, it would not have been possible to bring out the ACI special publication for distribution at the symposium in Las Vegas. The cooperation of the authors in accepting reviewers suggestions, and in revising their manuscripts accordingly is greatly appreciated. Particular thanks are extended to H. S. Wilson, Consultant, Ottawa, and G. D. Brearley and Maria Venturino, both of CANMET, for their help in the processing of the draft manuscripts.

The help of Messrs. A. Bilodeau (Chairman), B. Fournier, and R. Chevrier (Members) of the Slide Review Panel is greatly appreciated.

Helayne Beavers of ACI and her staff deserve very special mention for their assistance in getting this publication ready on time for distribution in Las Vegas, in view of the fact that the final revised manuscripts were delivered to ACI Headquarters only on March 9, 1995.

V. M. Malhotra, Editor
Chairman, Second CANMET/ACI International
Symposium on Advances in Concrete Technology

June 1995

**SECOND CANMET/ACI INTERNATIONAL
SYMPOSIUM ON
ADVANCES IN CONCRETE TECHNOLOGY**

Las Vegas, U.S.A.

June 11-14, 1995

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High Performance Lightweight Concrete — A Review

by T. W. Bremner and T. A. Holm

Synopsis: The intrinsic nature of lightweight concrete is to produce a material which, in addition to having high strength, can also have high performance in severe service conditions. The reason for high performance is examined in light of physical, chemical and mechanical properties of the vesicular aggregate used to produce lightweight concrete. The manufacturing process usually involves heating the aggregate to 1200°C which, in addition to rendering it more stable than conventional aggregates when concretes made from it are exposed to fire, also results in a less stiff aggregate inclusion that more closely matches the stiffness of the cement paste matrix. The use of less stiff aggregates results in a reduction in internal stress concentrations in the concrete which in turn leads to reduced microcracking. The role that this plays in enhancing the performance of this type of concrete is discussed. The special nature of lightweight concrete provides opportunities for design professionals, and recommendations on how best to achieve high performance concrete using lightweight aggregate are provided.

Keywords: Aggregates; density (mass/volume); fire resistance; high-performance concretes; history; lightweight concretes; manufacturing; strength; thermal resistance

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INTRODUCTION

The current use of the phrase "High Performance Concrete" (HPC) produces a surprisingly uniform image in the minds of practitioners in the English speaking world. It is usually but not exclusively restricted to high strength applications that necessitate water to binder ratios near that required for the chemical reaction between cement and water to go to completion when concrete is moist cured for a relatively long time (1,2).

Concomitant with this high strength and low W/B, is enhanced impermeability to aggressive gases and fluids. The net effect is to render HPC structures significantly less vulnerable to the distress exhibited by existing concrete structures in the public domain. In this connotation it appears to current practitioners to be new and of vital importance even though the words "high" and "performance" as defined by most dictionaries do not individually or collectively give that impression.

Dictionaries and Thesauruses fail to define "high performance", however the definition of the two words collectively in the context of concrete technology conveys the following:

High Performance Concrete: concrete that has enhanced performance and functional characteristics for a designated application.

This definition will be used to further define "High Performance Lightweight Concrete" (HPLC) where high strength and durability, although of importance, might not always be as great a concern as with normal weight concrete (NC). In this context, high performance might refer to enhanced insulating properties and fire resistance as well as to reduced density. All aspects that are derived from the intrinsic nature of lightweight concrete (LC) and that are not new and are not the result of recent development do not preclude it from being High Performance.

What is new and purported to be better does not always stand the test of time. However, with lightweight concrete most of the desirable properties

associated with High Performance have an excellent track record. To instill confidence in its use, the history of lightweight concrete will be reviewed as it relates to these attributes.

This discussion will be restricted to lightweight aggregate concrete, i.e. those concretes in which some or all of the aggregate inclusions are of a vesicular nature with vesicular being defined as containing, composed of, or characterized by having small cavities in a mineral or rock (3). Low density concretes which achieve their low density from the use of entrained air or preformed foam are not included. Although lightweight concrete was formerly defined as concrete having a density of less than 1850 kg/m³ (115 pcf) and compressive strength of 17.2 MPa (2500 psi) this restriction is not usually imposed (4). The essential part is that it contain at least some aggregates that are of a vesicular nature, and that a concrete of reduced density as compared to normal weight concrete is produced.

Based on the foregoing, the following definition of High Performance Lightweight Concrete (HPLC) is offered:

High Performance Lightweight Concrete: High Performance concrete that contains aggregates in which some or all are of a vesicular nature.

As with HPC, it is essential that a satisfactory workability be achieved at the time of placing and this usually requires the use of a superplasticizer. Depending on the application, silica fume, fly ash, ground granulated blast furnace slag cement and retarders may be desirable, however they are not essential (1).

GENESIS AND PAST PERFORMANCE OF HPLC

Port of Cosa: Long Term Durability

In the year 273BC, Roman engineers constructed five concrete piers reaching out into the Mediterranean on the West coast of Italy a few kilometres north of where the Tiber empties (5). These piers which are approximately 7 m wide x 10 m long x 5 m high served as a breakwater for the thriving fishing and manufacturing town of Cosa. That they exist to this day is a testament to the skill and knowledge of those ancient builders whose prior experience had taught them to travel large distances to get the essential ingredients to make HPLC. In addition to using a pozzolana from Possuoli, which is a distance of several hundred kilometres from Cosa, they used

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volcanic tuff - a natural volcanic lightweight aggregate from the Volsinian complex some 60 to 80 km northeast of Cosa. The piers, continuously submerged in salt water and with significant abrasion being caused by lateral drift of beach sands and gravel, have had their dimensions reduced by a surprisingly small amount which indicates that this construction will be around for many more millennia. Of particular importance to modern concrete engineers is that this concrete contained manufactured aggregates. One of the local industries in Cosa was an amphora plant where they fired two-handled clay containers that were used to ship wine around the known world. Broken pieces of amphora were incorporated into the concrete mix in a reasonably systematic way indicating that it was part of the mixture proportions.

Those fired clay pieces differ from those produced in a modern rotary kiln producing expanded clay aggregates in the degree of expansion developed by differing firing temperature regimes. Both expansion processes would normally use essentially the same maximum temperature. With brick manufacture, if the clay is heated too fast a vesicular expanded dimension brick not suitable for masonry construction is produced. It took almost two millennia before a materials pioneer in Kansas, USA recognized the potential of this supposedly inappropriate material. The suitability and long term stability of vitrified clay in concrete construction is verified by the success of this work which should be reassuring to modern engineers. Also, knowing of this good performance would have been of particular interest to the founder of the rotary kiln expanded shale industry some eight decades ago, for no doubt this question was raised when his product was first put on the market.

Pantheon: High Structural Efficiency

Roman engineers established a level of construction of public building that was not surpassed for 1500 years. This era of high profile construction started about 300 BC and was essentially completed by 270 AD, establishing standards of quality that following generations were unable to duplicate let alone advance. The best example of their work is probably the still functional Pantheon with its lightweight concrete dome of 43.3 m diameter which stands as a testament to their understanding of structural design and materials utilization (6). At Hadrian's direction, the builder formed the dome with its panelled recesses from wood and then cast concrete over it. The grain of the wood remains imprinted in the concrete to this day. The bare exposed concrete was covered with lead sheeting until the fifteenth century when it was stripped off to cover a building in the Vatican. The lightweight concrete dome roof with no waterproof layer appears admirably suited to this

application with no sign of deterioration or leaking taking place. Of particular importance is that the density of the concrete decreased as they moved up the dome with the tension ring in the bottom being of higher density than the top where essentially only compressive stresses were involved. To achieve this gradation in concrete density the inherent variability of natural lightweight aggregate deposits was used to good advantage. The deposit was apparently selectively mined to extract aggregates of the prescribed densities, a process not compatible with modern mining methods. Modern engineers usually cannot cope with a wide variation in material properties and, as a result, the manufacture of lightweight aggregate by the rotary kiln method came about so that density could be closely controlled.

Armenia: Reduced Seismic Inertial Forces

No country seems to be as well supplied with natural vesicular aggregates as Armenia and they have a long history of using it as the aggregate of choice for concrete construction mainly because they must look to quarried aggregates rather than to alluvial deposits and vesicular aggregates are the easiest to quarry. In the 1870's, Armenian engineers had recognized the need for further research on this material. Their building research institute at Erevan has provided a continuous record of conducting research on natural lightweight aggregates to this day. Being in an earthquake region, they learned early the advantages of reducing the density of the concrete so as to reduce the mass of the structure and thereby reduce lateral forces. The wide variation of aggregates available also enabled them to come up with multicolored decorative panels for the exteriors of their buildings. Because these panels required steel reinforcing, research starting in the 1950's was conducted on this type of construction to evaluate the corrosion potential of steel in lightweight concrete made from volcanically expanded aggregates. These findings were incorporated into the Soviet Union's building code for panelised construction using exposed natural lightweight aggregates.

Manufacturer of Vesicular Structural Grade Aggregates

Large areas of the western United States share a similar geology to Sicily, the Sakhalin Island Region of Russia, Armenia and other volcanic regions in that they are well supplied with natural vesicular aggregates. These aggregates enable moderate strength concrete to be produced that has high performance thermal and fire resisting properties as well as good

strength to weight ratios. Achieving higher performance with these materials unfortunately may not be possible because of the inherent wide variability of the natural deposits in terms of degree of expansion and non uniform vesicular diameter which limits its use.

To overcome these disadvantages and to produce a more functionally useful product Mr. Stephen J. Hayde patented in 1917 a method of manufacturing expanded shale aggregate using a rotary kiln. This patent formed the basis for the subsequent wide spread production of lightweight aggregate, that enabled concrete of significantly higher quality to be produced, and dramatically changed building practice. The intrinsic advantage of a uniformly expanded vesicular aggregate that was available in the size fractions necessary for the production of high structural efficient concrete was quickly recognized by engineers (7).

Ships Built During World War I: High Durability, Low Cracking

Prior to World War I, concrete ships had been built and proved functional and durable. The first concrete boat reinforced with steel was a small rowboat built by Lambot in 1848; one of his two row boats is on display in the museum at Brignoles in the south of France, the other is in storage at the Musée d'Orsay in Paris. These concrete row boats, though functional, served no commercial role. In World War I, the U.S. Marine Commission, charged with finding an alternative to the steel ships, found that only structural grade lightweight concrete could be a viable material. Initial construction experience showed that only a rotary kiln produced aggregate with high cement content concrete incorporating chemical admixtures could meet their needs. Just such a ship, the U.S.S. Selma, was launched in June 1919. This ship although subject to various marine disasters such as being run aground on a stone sea wall while fully loaded with oil, had a satisfactory service record and, when inspected in 1953 and again in 1988, the concrete was found to be in exceptionally good condition. This is in marked contrast to the Selma's predecessor, the USS Atlantis that was launched some nine months previous. The Atlantis was made of relatively low strength concrete (20-30 MPa) and with a lower quality aggregate made in a bee hive kiln rather than in a rotary kiln. The Selma's rotary kiln produced aggregate, plus enhanced concrete strength (50 MPa) as well as special attention paid to the consistency of the concrete (the "slump cone" was developed and first used during the construction of the Selma) resulted in an advancement in technology equal to what is now being claimed for HPC in concrete bridge deck construction (8).

Bridges and Buildings in the US: High Durability, High Structural Efficiency

The lightweight concrete industry benefited immensely from the research done by the U.S. Marine Commission, and at the end of the war, the design procedures and construction practices developed by the Commission were adopted by industry. In 1922, the Westport High School in Kansas City became the first lightweight structure built with concrete using rotary kiln aggregates and demonstrated the favourable cost and construction reliability of the product.

Factors found effective for producing durable ships were also incorporated in concrete bridge design. By 1935 over 34 lightweight concrete bridges were built in North America including nine in Canada (9). The fact that chemical admixtures that also sometimes entrained air were found desirable in placing lightweight concrete may, in part, account for the good long term performance of many lightweight concrete bridges built in regions where deicing agents are frequently applied. Deicing agents induce deterioration from both increased cycles of freezing and thawing, which the entrained air had a beneficial effect upon and from corrosion of the steel reinforcement. The corrosion is enhanced by the chloride based deicing agents acting as depassivating agents when the chloride ions have diffused into the concrete to the depth of the steel reinforcement. Various factors associated with lightweight concrete tend to reduce the seriousness of this effect and contribute to High Performance. This will be discussed later.

Ships Built During World War II, High Serviceability

The 14 World War I lightweight concrete ships were largely experimental, nevertheless, they were successful with one ship still currently in use in Powell River in British Columbia, Canada. When the same acute shortage of steel plate and surplus reinforcing bars developed at the start of World War II, the U.S. Government built 104 concrete ships, of which 24 were large sea-going vessels and 80 were sea-going barges. All reports indicated good handling and good performance, and they proved sufficiently water tight in service that they could be used to carry and store sugar and wheat. Additionally they were superior to steel in that they experienced minimum sweating and as a result, dry goods did not mould or cake (7).

As in the first world war, all ships built during war time were found unsuitable for peace time use. The steel ships were broken up for scrap and the concrete ships were usually used for piers and breakwaters. The authors

have visited many of these ships in the last decade and have found them all in essentially good condition considering the use to which they have been put.

Approximately 10 World War II cargo vessels can be seen near Cape Charles in Virginia where they serve as a breakwater for a now abandoned ferry terminal. All ships were sunk and then filled full of sand. The nature of the loading and the undermining of the bows and sterns by the current has caused most of them to break amidships however this is not unexpected. The only significant area of distress in these ships and in the USS Selma is in the thin, highly reinforced decks where improper concrete placing procedures appear to be the main cause of the problems. The hulls, where not damaged by very hard berthing or from overloading, are generally in excellent condition. The condition of all of the ships surveyed conveys confidence in the state of the art some 55 years ago. With the lessons learned from close examination of these ships, good long term performance for lightweight concrete ships today is certainly achievable.

Rotary Kiln Expanded Aggregate Experience In The Former Soviet Union: High Thermal Resistance

The early 1950's saw the Soviet Unions' commitment to make housing available to the Soviet people in the form of industrially produced precast concrete panels to form multi-storey apartment buildings. It was essential that the panels for the exterior walls be both load bearing and have high thermal insulating properties and be composed of fire resistant materials. The use of rotary kiln produced expanded clay (keramzite) concrete was found ideally suited for making the panels in all climatic regions in the former USSR. While most panels were made with monolithic lightweight concrete of high thermal resistance, in some cases a sandwich panel type of construction was used with high strength lightweight concrete skins about 75 mm thick being separated by a highly insulating concrete core. The skins had a density of about 1800 kg/m^3 and the core had a density of about 1200 kg/m^3 (10). To implement this type of construction in a country where only local areas such as Armenia had experience with lightweight concrete required the establishment of a research institute to investigate and come up with designs not only for the buildings and the building components but for the infrastructure to build them as well. This work needed to include everything from the economical production of aggregates from suitable clay sources near population centres to the finished building. To accomplish this, starting in 1950, a group of about 100 scientists and engineers worked at a special research institute in Samara (formerly Kuybyshev). These people provided the designs and guidance that enabled great strides to be made in

housing the nation. An important consideration was that combustible materials and materials generating noxious gases should not be incorporated in the building.

For four decades this talented team of highly qualified specialists worked to achieve optimum performance in the material used for the exterior panels. The extremely functional material developed - High Performance Lightweight Concrete - has yet to be fully appreciated by their current government or by the West.

Offshore Structures: High Durability, High Specific Strength

Oil production in the offshore requires either fixed or floating platforms and, given the success of the lightweight concrete ships in World War I and II, it is not surprising that the oil industry turned to structural grade lightweight concrete. Prior to embarking on this work a group of eleven companies funded a joint industrial study to investigate the use of high strength lightweight aggregate concrete for Arctic applications. The results have wide applicability as the effect of typical construction scenarios were incorporated into the testing program. Such design parameters as modulus of elasticity, splitting tensile strength and thermal properties were evaluated and fully reported for high strength concrete, areas which previously were not well defined (11). The net effect of this multi-million dollar research program is that now, high performance high strength lightweight concrete is as well defined in the technical literature as normal weight concrete. Given the large number of aggregate types and the even larger number of aggregate suppliers world wide, it is unlikely that all the natural aggregates being used in concrete will ever be defined as closely as this multi-national research group did for lightweight.

As part of their on-going research program on concrete for offshore construction, CANMET carried out an extensive testing program to develop high strength concrete using a Canadian lightweight aggregate. This work established that 70 MPa concrete with a density of 2000 kg/m³ could be produced commercially and that it would have freezing and thawing resistance suitable for cold northern waters (12).

With over 70 years experience with lightweight concrete ships and over 40 years experience with lightweight concrete barges and platforms along the coast of the Gulf of Mexico, it was natural for the oil industry to select lightweight concrete for the Tarsuit and Super CIDS that were installed in the Beaufort Sea. To gain experience with placing of lightweight concrete mixtures for high strength application, Norwegian Contractors slipformed

part of a leaning tower at their Stravanger Yard using German lightweight aggregated in lightweight concrete and slipformed lightweight concrete shafts containing US lightweight aggregates in Corpus Christi, Texas in both winter and summer to demonstrate the practicality of both the material and the construction method. Based on this experience they and others have used lightweight concrete successfully in many applications in the North Sea involving floating and fixed facilities for the oil industry as well as for pontoons for two floating bridges (13).

NATURE OF HPLC

Lightweight Aggregates as Vesicular Inclusions

During the high temperature production of either natural or manufactured lightweight aggregates there is a partial decomposition of some of the compounds within the aggregate mass takes place. This decomposition results in the release of gases which coalesce to form vesicles or cells within the aggregate mass. This expanded or distended form is retained upon cooling and accounts for the aggregate being light weight. The nature of the rock must be such that it has a reasonably wide bloating range and will remain sufficiently viscous at high temperature to retain the gas bubbles. As with foam plastics, the object is to develop a closed cell foam structure and to avoid producing an excessively expanded open type cell system.

The closed cell foam is characterized by being rigid with low water permeability. With an open cell foam, a flexible and highly permeable material results. Unlike cellular plastics, the vesicles in aggregates are formed at very high temperature and upon cooling the gases contract inside the pores with the result that with a normal degree of expansion there appears to be some connectivity between individual pores. These conduits are not easily seen in a scanning electron microscope even at magnifications in excess of 3000 indicating that if there are pores, they either self-seal at high temperature or are of dimensions less than 30 nm (14).

Most lightweight aggregates have a silica rich vitreous ceramic like matrix surrounding the vesicles that will be resistant to fluid flow. Evidence to support the limited connectivity of all internal pores is the fact that the aggregates will not normally become fully saturated if stored under water for several years. Most aggregates have a rapid absorption rate in the first 30 minutes that precludes the use of air dry or oven dry aggregates in concrete mixtures. If submerged in water for 24 hours (24 hour absorption test), the subsequent rate of water absorption is so low as to have a negligible

effect on slump loss during mixing and placing and renders most concrete mixtures pumpable. Achieving this level of aggregate moisture content is best accomplished at the production plant and is not a field problem.

Surface characteristics of lightweight aggregates depend on their origin and the manufacturing process. Lightweight aggregates that are crushed to meet various size requirements generally have a rough texture but, if they have a reasonable particle shape, there is little effect on water demand.

The surfaces of aggregates that have been subjected to pyroplastic temperatures (1200°C) during production have experienced a temperature above what is normally required to activate shales and clays so as to form a commercial pozzolan. This activation results in a more effective bond between the paste and the aggregate which is the primary reason why failure planes pass either through the cement paste matrix or the aggregate inclusion but almost never through the aggregate cement paste interface (15).

Fracture can be in the form of microcracking brought about by volume changes resulting from temperature and humidity gradients in the concrete or from the effects of superimposed forces. These result in a multitude of microcracks which may subsequently network to form a few large cracks that lead to ultimate compression or tension failure. Observation of failure surfaces of both normal weight and lightweight concrete clearly shows the importance of the matrix quality and aggregate strength for lightweight and the importance of matrix to aggregate adhesion in normal weight concrete.

The use of a relatively low permeability vesicular aggregate provides inherent advantages as a receptor for water that can be stored and dispensed at a later date into the cement paste matrix to extend and enhance moist curing. This is of profound importance in the reduction of the permeability of the concrete composite system. Water in a porous media is attracted to the regions with the finest microstructure by capillary forces. When a partially saturated aggregate is incorporated into a concrete mixture, and the concrete is kept fully saturated there is not likely to be a significant flow of water from one component of the composite to the other. When the cement paste matrix hydrates to a finer microstructure and starts to dry out, the water in the coarse pores of the aggregate will give up the absorbed water to the cement paste matrix. This effectively extends the moist curing period significantly for lightweight concrete members.

Water contained within the aggregate increases the early age concretes vulnerability to damage due to freezing and thawing and to spalling in a fire. Special precautions need to be taken with HPLC as the cement paste is

especially impermeable as the result of very low W/B and the possible use of silica fume, fly ash and ground granulated blast furnace slag cement. Concretes containing high moisture content lightweight aggregates need to be allowed to dry for at least 7 days before being exposed to freezing and thawing. Longer times are needed for the case of fire resistance where air drying of both normal weight and lightweight full sized structural elements can take several years if explosive spalling is to be avoided. The incorporation of a small amount (1-2%) of polypropylene fibers has been found to essentially eliminate the spalling tendency of highly saturated high strength lightweight concrete when subjected to rapid temperature increases such as is likely to be experienced by oil production facilities (13).

Particulate Composite with Vesicular Inclusions

Concrete can be considered as a composite material in that it is composed of two or more materials separated by a distinct interface. Also it is a particulate composite in that it is composed of well graded particles surrounded by a cement paste matrix. With most particulate composites, the two phases (inclusion and the matrix) are not of the same material and as a result have different physical and mechanical properties such that various types of incompatibilities exist. Of prime concern with the strength of particulate composites is that the stiffness of the inclusion usually is different from that of the matrix. In the initial stages of cement hydration the cement paste matrix is of low strength and of low stiffness. The aggregates - both lightweight and normal weight, are strong and stiff in comparison. Because of their rigidity they attract and carry a disproportional amount of the load. Unfortunately, the transfer of stress between matrix and inclusion places high stresses at the aggregate/cement paste interface with the result that premature failure (in the form of microcracking) takes place prior to the ultimate strength of the weakest of the two components being reached. As the concrete matures the cement paste matrix gains both strength and stiffness such that for moderate strength lightweight concrete the stiffness of both the inclusion and the matrix are essentially equal and little microcracking takes place prior to reaching ultimate compression or tension strength leading to effective utilization of the weaker component. With normal weight concrete, the cement paste matrix, even when fully mature and with low W/B and high addition rates of silica fume, will reach less than one half the stiffness of most aggregates. Significant stress concentrations develop that lead to ineffective utilization of the weaker component or more frequently to interfacial failure at the aggregate-matrix interface. This failure, usually in the form of microcracking, commences at about 65% of the ultimate compressive strength and becomes more severe as ultimate load is approached. High levels of microcracking will substantially increase the air,

gas and chloride ion permeability of the concrete. Normal weight concrete can approach the desirable condition where there are no stress concentrations at the aggregate cement paste interface, however this is possible only at ultra high strength levels. HPLC that is exposed to effective moist curing can be programmed to be elastically compatible by either controlling the degree of expansion of the aggregate or by proportioning concrete mixtures so that the matrix will have a stiffness at maturity that matches the aggregate inclusion.

Because of the foregoing, the stress level in the lightweight aggregate is less than that in the natural aggregates at the same concrete strength. This is fortuitous as the strength of most materials decreases rapidly as the degree of vesicularization increases. Fortunately high strength lightweight concrete is possible as the reduction of the stress concentration plus enhanced bond strength between the aggregate inclusion and the cement paste matrix makes up for the limited particle strength of the expanded aggregate (15).

There is a strength threshold above which for a particular expanded aggregate, it will be difficult to increase strength by simply increasing the cement paste matrix strength. For expanded aggregates of a particular type, this strength threshold may be increased by increasing particle density or reducing particle top size. Normally it is not economically feasible to increase strength above this level by increasing the cement factor (16).

Aggregate Microstructure Conducive to Good Performance

A relatively uniform degree of expansion of all aggregate particles with vesicles of approximately equal size contributes to high strength concrete. If some aggregates contain a limited number of very large pores the aggregates will inevitably produce low strength concrete.

Aggregates produced in a rotary kiln and not crushed tend to have a more dense outer skin which can in some cases lower the rate of water absorption, however this is not as significant as initially believed (14). Apparently the smooth outer skin, although appearing more dense may be prone to crack and these cracks tend to enhance water absorption. In the work for the offshore HPLC applications (11), the rate of absorption and the 24 hour absorption were both lower for the crushed and graded aggregates as compared to the pelletized, uncrushed lightweight aggregate.

PROPERTIES OF HPLC

Strength and Stiffness of a Vesicular Aggregate

Optimum strength at a given degree of vesicularization is achieved when a uniform pore structure has formed in the aggregate and the heating and cooling regime is such that the aggregates are relatively free from surface and interior cracks. A reliable method of determining the extent of surface cracking is to soak aggregate particles in red drafting ink which, after drying, are broken and the extent of cracking observed. A stereo or scanning electron microscope can be used to evaluate the extent of internal cracking.

Aggregates with a wide range of vesicle sizes tend to be weak. The majority of commercially used aggregates that are most suitable for HPLC normally have a relatively uniform degree of expansion of each of the individual particles and the vesicles all tend to be of one size. To produce HPLC, it is essential that all raw material mined have a reasonably uniform expansion and that the shape and size of the vesicles in the aggregate have a limited range.

The stiffness of a wide range of natural and manufactured lightweight aggregates of diameter between 12 and 16 mm was measured dynamically by J. Muller-Rochholz to be as follows:

$$E = 0.008 \rho^2$$

where ρ is the density of the aggregate particle (16). Unexpanded ceramic like aggregates have a density (ρ_s) of about 2500 kg/m³. The above equation gives a modulus of elasticity E_s of 50 GPa which is in line with most dense aggregates. With 50% voids, the density reduces to 1250 kg/m³ and the stiffness reduces to 12.5 GPa. With closed cell plastics the following expression is used to calculate the stiffness of a foam:

$$E = \phi E_s (\rho/\rho_s)^2$$

When Muller-Rochholz results are used to solve for ϕ a value of 0.99 is obtained which is in agreement with data reported by Gibson and Ashby (17).

Holm (16) reports that, for typical North American structural grade lightweight aggregates, the dry particle densities range from 1.2 to 1.5 (1200 to 1500 kg/m³). At these densities, the modulus of elasticity of the individual particles of lightweight aggregates "approaches that measured on the mortar

fraction of air-entrained commercial strength" lightweight concrete thereby reducing the stress concentration to almost 1.0 (16).

Mechanical Properties of HPLC

As discussed in the section on the Nature of HPC, because most structural grade lightweight aggregates have a modulus of elasticity the same as the cement paste matrix the stress concentration approaches unity (elastic compatibility) in mature HPLC. As a result of this uniform distribution, the aggregate need only carry the average stress applied. Unexpanded aggregates being as much as five times as rigid as the matrix attract additional stress which it usually can readily sustain. Unfortunately the transfer of these large stresses from aggregate to matrix results in premature failure at the aggregate-cement paste interface at a stress significantly less than the strength of the paste matrix or the inclusion.

The lack of stress concentrations at the aggregate-cement paste interface and the good bond achieved between the matrix and the lightweight aggregate has many advantages including enhanced fatigue resistance. Hoff (18) points out that this "greatly reduces the formation of microcracks at lower stress levels and thus delays the damaging effects caused by cyclic loading" leading to the conclusion that under similar test conditions the fatigue behavior of high strength lightweight concrete is comparable or somewhat better than high strength normal density concrete.

For strengths of lightweight concrete above usual structural levels (i.e. $W/B < 0.4$) failure of the aggregate will limit the strength of the composite. Attempts to have the strength of the concrete exceed the strength ceiling of the aggregate is difficult, requiring the cement content to be increased greatly to achieve even modest strength increases (16).

Thermal Properties of HPLC

Valore (19) reported on the extensive work done by Rowley and Algren who tested a wide range of densities of concretes made from essentially all types of vesicular aggregates in common use and showed how this information could be used to calculate the thermal properties of concrete masonry units. This work revealed that when the density of concrete is reduced by one third though the use of vesicular aggregates, the thermal resistance increased by a factor of 3 or 4. When the effect of using masonry units in lieu of solid concrete walls is considered, then an essentially poor

insulating wall made with NC can be replaced with LC masonry units thus significantly reducing heat flow and resulting in a high performance type of construction.

Fire Resistance of HPLC

Expanded shale has a linear stress-temperature relationship whereas most silicious type natural aggregates are nonlinear and may expand about 3 or 4 times as much as expanded shale when heated to 900°C. When incorporated in a concrete mixture these two types of aggregates resist the propensity of the cement paste matrix to contract when heated above about 300°C. Because the vesicular shale lightweight aggregate expands less, the concrete incorporating this type of aggregate tends to expand less. As a result internal disruption is reduced such that the loss of strength of the concrete at high temperature is decreased. Also with less expansion of the concrete, the bowing of walls exposed to fire on one side is lessened and, as a consequence, structural stability in terms of resistance to buckling of the wall is greater leading to high performance compared to when NC is used (20).

PRODUCTION OF HPLC

The production of HPLC is not more difficult than the production of HPC. However, as with LC and NC, there are differences that necessitate a different approach to achieving quality construction. The important points to keep in mind are:

- A) Use only air-entrained concrete regardless of whether it is subject to freezing and thawing or not as the air entrained matrix tends to prevent floating of the lighter aggregate inclusions during transporting and placing of the fresh concrete.
- B) All lightweight aggregates should be prewetted to a moisture content equal to or exceeding their 24 hour absorption to enhance internal curing and to avoid excessive slump loss and concrete mixtures that are difficult to pump.
- C) HPLC's with slumps in excess of 180 mm are difficult to use and are not normally required. If a slump in excess of 180 mm is used, the aggregate will segregate (float to the surface) when normal handling and placing procedures are used. This problem is often eliminated in mixtures containing more than 6% silica fume. The added stickiness provided by the silica fume tends to reduce segregation.
- D) Similar placing characteristics can be achieved when the slump of HPLC

- is about 2/3 the slump of HPC.
- E) Use less vibration time with HPLC than is used for HPC. As with any concrete, the length of time needed to vibrate the concrete to achieve full compaction can be easily determined by direct observation.

FUTURE ROLE OF HPLC

Current international experience with the offshore platforms, both fixed and floating, have demonstrated the advantages of HPLC. This has lead to its extensive use by companies in Japan, USA, UK, Australia, Canada and Norway. From the research funded by these firms, design information is now readily available on high strength lightweight concrete and new and novel designs for such facilities as long span bridges and stadiums that exceed that possible with normal weight concrete can be expected.

Effective containment of aggressive liquids stems directly from the use of high quality concrete. If the structure is reinforced however, flexural cracks may provide easy flow through the concrete. A logical remedy is to use prestressing which will eliminate flexural cracks perpendicular to the prestressing. HPLC, with a more uniform stress distribution within the mass of the concrete, can be designed to have a minimum level of microcracking at service loads, leading to enhanced impermeability and consequently less damage from aggressive media. Current interest in secondary confinement of surface oil storage using precast prestressed lightweight concrete will no doubt lead to HPLC use for this as well as other, perhaps more demanding, uses such as in the hazardous chemical and nuclear waste field.

CONCLUDING REMARKS

The development, nature and future role of high performance lightweight concrete were covered in the hope that owners, designers and builders will use it with confidence to produce more functional, economical and more serviceable structures.

It is suggested that it is the structure that needs to be high performance. High performance structures need to be built with high quality concretes that satisfy the functional requirements which may be, but are not limited to:

- structural efficiency (strength/weight)
- resistance to weathering and chemical attack
- thermal resistance (energy conserving)

thermal stability (fire resistance)
cracking resistance (strain capacity).

From the above it may be observed that the current, commonly held assumption that the expression High Strength and High Performance are synonymous is an unduly restrictive and sometimes erroneous linkage. Indeed High Performance need not be high strength and high strengths achieved without adequate concern for other qualities may not be high performance (e.g. cracking).

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SP 154-2

Flexural Cracking Evaluation Using Fiber Optic Sensors in High Strength Composite Beams Reinforced with Prestressed Prisms

by E. G. Nawy and B. Chen

SYNOPSIS

This paper covers analytical and experimental investigation of high strength concrete beams reinforced with high strength prestressed concrete prisms as main reinforcement. Fiber optics technology has been developed and used in this investigation to measure the flexural crack widths developed throughout the full loading history of the specimens.

Thirteen beams, 8 in. x 12 in. (200 x 300 mm) in cross-section and having a 9.0 ft (2.74 m) span were tested to failure. The embedded prestressed prisms had a length of 9 ft. 6 in. (2.90 m) and cross-sectional dimensions ranging between 1.5 in. x 3.0 in. (38 mm x 76 mm) and 4.5 in. x 3.0 in. (114 mm x 76 mm). The prisms were prestressed with 7-wire 3/8 in. (10 mm) diameter, 270 ksi (1860 MPa) tendons. Concrete strength in both the prisms and the beams was in excess of 14,000 psi (100 MPa) using silica fume as a partial cementitious replacement as well as a high range water reducer (superplasticizer) in order to attain the desired workability and compressive strength.

A study of the extensive data accumulated in this research program, supported by the National Science Foundation, resulted in expressions for the evaluation of flexural crack widths in ultra-high strength concrete composite beams. Test results also showed that the embedded prisms delayed the development of cracks, while the additional use of non-prestressing steel significantly reduced the crack spacing in the beams and limited the crack width at the onset of prism cracking.

Keywords: Beams (supports); cracking (fracturing); crack width and spacing; fiber optic sensor; high-strength concretes; loads (forces); prestressing; prisms; reinforced concrete; stabilization; strains; stresses

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INTRODUCTION

Crack control has long been considered a critical issue in concrete construction (1-15). Use of precast pretensioned prisms as tension reinforcement can effectively improve the cracking behavior of concrete members. Previous works in this area was limited. Burns (5) described a testing program using prisms to control cracking in continuous members at negative moment regions. Hanson (10) studied crack control in T-beam flange. Bishara and Almeida (4) used embedded prisms as tension reinforcement in normal strength simple and continuous rectangular beams. All these tests showed a consistent improvement of flexural performance at working load levels and also an adequate plastic rotation capacity up to failure. Since 1976, this type of concrete construction was also experimentally studied by a number of foreign (Paramewaran (16)). Two major advantages have emerged from using prestressed prisms as main reinforcement in concrete members using normal strength concrete. These are higher cracking capacity and the ability of cracks to close with unloading. Hence, it is beneficial to investigate the use of such composite construction in high strength and ultra high strength beams and develop expressions for evaluating the crack width and spacing in such members as an aid to the designers in controlling serviceability behavior.

Fiber optic sensors technology was developed and used in this investigation in order to provide an accurate measurement of steel reinforcement strain and mean crack width in the tested specimens. As a result, a mathematical model and expressions were developed for crack control evaluation as well as a new approach for long-term and remote monitoring of civil engineering structural systems.

TESTING PROGRAM

The testing program included thirteen high strength concrete rectangular beams with prestressed prisms as main reinforcement. The beams were 8 in. $\times$ 12 in. (200 mm $\times$ 300 mm) in cross section, were simply-supported over a 9 ft (2.74 m) span and subjected to the third point loading. All the precast prestressed prisms had the same length of 9.5 ft (2.90 m). Typical section details are given in Fig. 1 and Table 1.

High strength concrete having a compressive strength, f'_c , up to 14,200 psi (100 MPa) was used, with silica fume replacing part of the ASTM Type III portland cement, with the mixture proportions as follows and concrete strengths of the test beams as in Table 2.

Component	Weight	
	lb/yd ³	Kg/m ³
Portland Type III cement	720	425
Powder silica fume	180	106
3/8 in. coarse aggregate	1850	1092
Fine aggregate (coarse sand)	1100	649
Liquid superplasticizer	54	32

Uncoated stress-relieved 270 ksi (1860 MPa) 7-wire 3/8 in. (10 mm) diameter tendons conforming to the ASTM A416 were used to prestress the precast prisms. No. 5 (15.9 mm dia.) deformed bars conforming to ASTM A616 were used as additional tension reinforcement in beams A-1 through A-5. No.3 (9.5 mm dia.) deformed bars were used as web stirrups for all the test specimens.

At load levels corresponding to the service loads, all crack widths in the constant-moment region were measured with an illuminated microscope of 0.001 in. (0.025 mm) accuracy. Crack spacings of all the developed cracks in this region were also recorded.

Fiber Optic Bragg Grating (FOBG) sensors were used to measure the average strain developed in the reinforcement and in the concrete. The FOBG sensor was either internally mounted along the reinforcing bars or externally mounted on the beam surface with a gage length of 16 in. (406 mm) as shown in Fig.1. Five to seven electric strain gages were mounted inside each beam on the main reinforcement within the constant moment region. A group of LVDTs were also installed at the the maximum moment region.

TEST RESULTS

General Cracking Characteristics

In beams reinforced with prisms only (beams A-6 through A-13), the first visible crack initially propagated to a depth essentially equal to the thickness of the prism concrete cover. Thereafter the cracks extended beyond the prism level with further increase in load even though the prisms remained **uncracked**. This was an important characteristic which indicated that the beam would retain a much higher flexural rigidity after initial cracking of the concrete cover. At load levels causing the prism concrete to crack, the maximum crack widths ranged between 0.006 to 0.010 in. (0.15 to 0.25 mm) at the beam bottom surface and 0.003 to 0.006 in. (0.076 to 0.15 mm) at the tension reinforcement level. These crack widths fell within the tolerable crack width ranges given by ACI Committee 224 (1). Once prism cracking occurred, a sudden increase in width and height of the beam crack was observed corresponding to a sharp increase in beam deflection. At ultimate loading, only one or two major wide cracks propagated into the compression zone.

Cracking Capacity of Beam Cover

All thirteen test beams showed that the loads causing the first visible crack were practically at the same level and corresponded to a relatively low stress level in the reinforcement in the range of 3500 psi (24 MPa). This meant that the prisms did not result in a substantial increase in the cracking capacity of beam cover. Because the prisms continued to maintain some of the initial precompression, they remained uncracked. However, the restraining effect of the prisms on the cracking behavior of the cover concrete seemed to be insignificant.

Cracking Capacity of Prestressed Prisms

In comparison with normal prestressed concrete flexural elements, prisms used as main reinforcement in these beams had relatively small cross-sections and a precompressive stress level as high as possible. The prism concrete cracks only at a stress level that equal to the sum of the precompressive stress due to prestressing and the concrete tensile strength. Thus, the cracking capacity of the prism was considerably higher than that of the beam cover. Predicting the cracking moment capacity M_{per} of the prism depends on the strain distribution and tensile strength of the concrete developed as in Chen (7).

PROPOSED APPROACH FOR CRACK CONTROL

Stabilized Mean Crack Spacing

Figure 2 shows the variation of average crack spacing with the applied load

(bending moment ratio) for beams A-4 and A-5. This type of composite member is similar in behavior to a regularly reinforced concrete member in that crack spacing stabilizes at particular overload levels. Crack spacing in a member with combined prism and bar reinforcement stabilized after cracking of the embedded prisms at 30 to 60 % of ultimate load. For members reinforced with embedded prisms only, the stabilized crack spacing was usually reached before cracking of the embedded prisms at 25 to 45 % of ultimate load. The magnitudes of stabilized mean crack spacing, a_{cs}' , for all test specimens are presented in Table 3. Other observations and findings from the test results are summarized below.

The stabilized mean crack spacing in the test specimens with deformed bars as additional reinforcement (beams A-2 through A-5) was normally less than that reinforced only with prestressed prisms. Also, cracks in beams A-3 through A-5 stabilized at a steel stress level of 28,000 to 36,000 psi (193 to 248 MPa) and were more evenly distributed than those of the rest of test beams, indicating the importance of mild deformed bar reinforcement in crack control.

For beams reinforced only with prisms, the crack spacing stabilized at a reinforcement stress level of 4000 to 7000 psi (27.6 to 48.3 MPa). In addition, crack distribution in beam cover showed more randomness in irregular spacing. This can be due to the difficulty of controlling the bond between the precast prisms and the surrounding situ-cast beam concrete. The stabilized mean crack spacing as developed by Nawy and Huang (13) using normal strength concrete beams seems to be equally accurate for the high strength composite beams in this investigation:

$$a_{cs} = c \frac{f_t'}{\sqrt{f_c'}} \frac{A_t}{\Sigma o} \quad (1)$$

where c = a constant to be determined from tests
 A_t = area of concrete in tension
 Σo = sum of the perimeters of reinforcement
 f_c' = cylinder compressive strength of concrete
 f_t' = tensile splitting strength of concrete

In this study, factor c was found to have a mean value of 0.097 with standard deviation of 0.016. The ratio $(f_t' / \sqrt{f_c'})$ has a mean value of 5.783 with standard deviation of 0.657, as indicated in Table 3. Hence, the expression for predicting the stabilized average crack spacing for this type of member becomes as follows:

$$a_{cs} = 0.561 \frac{A_t}{\Sigma o} \quad (2)$$

The area A_t of concrete in tension is defined as,

$$A_t = [h_p + c_p] \times b \quad (3)$$

where h_p = height of the prisms

c_p = clear concrete cover to the prestressed prism

b = width of the rectangular beam section

The experimental results of stabilized crack spacing for beams A-3 through A-13 are presented and compared with that obtained from Eq. 2. Results are summarized in Table 3.

Mean Crack Width

Because of the randomness of cracking behavior in concrete, correlation of the maximum crack width, w_{max} , as a function of reinforcement stress with the maximum crack spacing, $a_{cs,max}$, might induce some unpredictable scatter unless the reinforcement strain is accurately measured. Use of advanced fiber optic sensor technology in this study accomplished this aim, hence the mean crack width, w_{mean} , can therefore be well defined. The mean width of stabilized crack at the reinforcement level is a function of the difference in elongation between the reinforcing bars and the surrounding stretched concrete over a length a_{cs} (distance between two adjacent cracks) as follows:

$$w_{mean} = \epsilon_s a_{cs} - \epsilon_c a_{cs} \quad (4)$$

or

$$w_{mean} = \alpha \epsilon_s a_{cs} \quad (5)$$

where $\alpha = (1 - \epsilon_c / \epsilon_s)$ is a parameter defining the influence of concrete elongation on cracking development and ϵ_c and ϵ_s are the average strain in the concrete and reinforcement, respectively.

Eq. 5 shows that mean crack width w_{mean} is proportional to the product of ϵ_s and a_{cs} , experimentally evaluated as shown in Figure 3. It should be noted that this linear relationship applies even before cracking of the prisms, indicating that directly correlating the average strain in the reinforcement encased in the prisms to the cracks on beam surface is reasonable.

The magnitude of factor α in Eq. 5 can be experimentally evaluated. It has been found that the strain in steel ϵ_f measured from the externally mounted FOBG sensor essentially equals to the average strain ϵ_s measured by the internally mounted electric strain gages [Chen (7) and Chen and Nawy (8)]. Hence,

$$\epsilon_s = \epsilon_f \quad (6)$$

Eq. 6 indicates that the total elongation in the reinforcement is approximately equal to the total stretch in concrete, demonstrating that the influence of the tensioned concrete between the cracks is negligible. Hence, the factor α is approximately equal to 1.

By mounting the FOBG sensor at the beam extreme tension fibers for **direct** measurement of the crack width, the mean crack width is expressed as:

$$w_{mean}^f = \epsilon_f \frac{\ell_g}{n_c} \quad (7)$$

with the factor α being more accurately defined by

$$\alpha = \frac{w_{mean}^t}{w_{mean}^f} \quad (8)$$

where ℓ_g is the effective gage length and n_c is the number of cracks covered by the FOBG sensor, w_{mean}^t represents the mean crack width obtained from the readings of a hand-held microscope. The mean crack width, w_{mean}^t is plotted against w_{mean}^f for beams A-9, A-10, A-12 and A-13 as shown in Figure 4. It is observed from this plot that the slope of the regression line is 1. This again indicates that factor α can be approximated by a value of 1.

It has been demonstrated in this work that the FOBG sensor gives a considerably more accurate estimate of the mean crack width at the tensile face of the concrete than other means as long as good bonding is developed between the optical fiber and the concrete face. This is because the optical fiber is flexible and can closely adjust itself to the deformed beam surface.

Using a value of $\alpha = 1$ in Eq. 5 as reasonably accurate for prediction of mean crack width,

$$w_{mean} = \epsilon_s a_{cs} \quad (9)$$

where the stabilized mean crack spacing a_{cs} is given by Eq. 2, and hence,

$$w_{mean} = 0.561 \frac{A_t}{\Sigma o} \epsilon_s \quad (10)$$

Eq. 10 is verified from the experimental test results of beams A-3 through A-13 as given in Table 4. In general, there is good agreement. The average reinforcement strain can be defined as,

$$\epsilon_s = \rho \frac{f_s}{E_s} \quad (11)$$

where ρ = parameter reflecting the stress distribution along the reinforcing bars

E_s = Young's modulus of the reinforcement

f_s = stress in the normal steel reinforcement at the cracked section or the change in stress in the prestressing steel of the prisms after decompression, namely, after the prism cracks.

By substituting Eq. 11 into Eq. 9, the mean crack width, w_{mean} , is correlated with the steel stress, f_s , giving

$$w_{mean} = \rho \frac{f_s}{E_s} a_{cs} \quad (12)$$

the average value of parameter $\rho = 0.843$ in these tests.

Maximum Crack Width

Once the mean crack width w_{mean} is determined, the maximum crack width w_{max} under short-term loading can be determined by applying a multiplier τ to the w_{mean} , i.e.,

$$w_{max} = \tau w_{mean} \quad (13)$$

The τ -value was statistically evaluated for the ratio of the test values of the maximum to the mean crack widths in beams A-3 through A-13. These ratios are shown in the normal probability plot of Fig. 5, based on 69 observations. The mean value of $\tau = 1.501$ with a standard deviation of 0.328. Introducing the multiplier τ into Eq. 12, the maximum crack width becomes

$$w_{max} = \tau \rho \frac{f_s}{E_s} a_{cs} \quad (14)$$

Taking Young's modulus as 29×10^6 psi and using the results of the statistical

analysis, Eq. 14 becomes the following

$$w_{\max} (\text{inch}) = 2.45 \times 10^{-5} \frac{A_t}{\Sigma o} f_s \quad (15)$$

with f_s in ksi units, A_t , sq. in. and Σo , in.
Equation 15 in SI units becomes

$$w_{\max} (\text{mm}) = 3.6 \times 10^{-5} \frac{A_t}{\Sigma o} f_s \quad (16)$$

with f_s , MPa; A_t , cm^2 and Σo , cm.

The relative frequency histogram in Figure 6 for the ratio of the test to theoretical maximum crack from Eq. 15 shows a confidence interval $R_2 = w_{\max}^t / w_{\max}$ of about 80%. This scatter in crack width evaluation is expected due to the random behavior associated with cracking [Nawy (11)]. Tests by Billah and Nawy (3) on high strength normally prestressed non-composite partially prestressed beams gave a multiplier in Eq. 15 with a range of 2.63 and 3.82×10^{-5} and a mean value of 2.75×10^{-5} for non-composite prestressed beams having a compressive strength, f'_c , in the range of 10,000 psi.

Adjustment expressions of crack width as a function of the concrete strength are given in Ref. 14 and 15. This range confirms that the maximum crack width in this type of composite members is generally similar to other types of concrete members. The present investigation also confirms that the crack width in high strength concrete beams is considerably smaller than the width developed in normal strength beams, $2.45 - 2.75 \times 10^{-5}$ versus 5.85×10^{-5} multiplier in normal strength beams [Nawy (13, 15)].

The maximum crack width at tensile beam surface can be predicted by,

$$w'_{\max} = 2.45 \times 10^{-5} R_i \frac{A_t}{\Sigma o} f_s \quad (17)$$

where R_i is the ratio of distance from neutral axis to tension face to the distance from neutral axis to centroid of reinforcement. For test beams A-3 through A-5, $R_i=1.39$ and for test beams A-6 through A-13, $R_i=2.28$. Analytical results obtained from Eq. 17 are compared with measured maximum crack width at the beam tensile face as indicated in Fig.7. The ratio $R_3 = (w'_{\max})^t / w'_{\max}$ is shown to have a confidence interval of approximately 90%.

CONCLUSIONS

1. Fiber Optic Bragg Grating Sensors (FOBG) are effective in continuous monitoring of flexural surface mean crack width development in concrete structures. The maximum crack width is estimated by using

an experimentally determined multiplier $\tau = 1.5$ in this investigation.

- From these tests, the maximum crack width (in.) at the level of the reinforcement for composite reinforced concrete beams can be predicted from

$$w_{\max} = 2.45 \times 10^{-5} \frac{A_t}{\Sigma o} f_s$$

The maximum crack width (in.) at tensile reinforcement level is

$$w'_{\max} = 2.45 \times 10^{-5} R_i \frac{A_t}{\Sigma o} f_s$$

For normal prestressed concrete beams, the multiplier is 2.75×10^{-5} , to be adjusted for concrete strengths above 10,000 psi (68.9 MPa).

- Use of prestressed prisms as main reinforcement in composite concrete beams substantially reduces the flexural crack widths and contributes to better serviceability performance with more stiffness.
- Cracking of the embedded prisms started at much higher load levels than the concrete cover even if the cover is small. The crack width in the prisms, similar to cases of non-composite beams, can be accurately predicted by the proposed equations.
- Visible crack spacing stabilized at 30 to 60 % of the ultimate load in beams with combined prism and bar reinforcement, and at 25 to 45 percent of the ultimate load in beams with only prism reinforcement.
- The presence of non-prestressed reinforcement significantly reduces the crack spacing and limits the crack width at the onset of the prism cracking.
- Flexural crack width in high strength concrete beams can be almost 40 percent of the width in normal strength beams (2.45×10^{-5} versus 5.85×10^{-5} multiplier in the crack width equation).

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NOTATION

A_t	=	area of concrete section in tension, in. ²
a_{cs}	=	stabilized mean crack spacing, in.
b	=	beam width, in.
c_p	=	cover to prestressed prism, in.
d	=	effective depth to center of mild steel reinforcement, in.
d_p	=	effective depth to center of prism prestressing reinforcement, in.
E_s	=	Young's modulus of reinforcement, ksi
FOBG	=	Fiber Optic Bragg Grating Sensor
f'_c	=	cylinder compressive strength of concrete, psi
f_s	=	Stress in the mild steel reinforcement or <u>change</u> in stress of the prestressing prism reinforcement after cracking (decompression), ksi
f'_t	=	tensile splitting strength of concrete, psi
h_p	=	prism height, in.
ℓ_g	=	effective optic fiber gage length, in.
M_{cr}	=	cracking moment of the beam and prism
M_{ult}	=	ultimate moment capacity
n_c	=	number of cracks within the FOBG sensor gage zone
R_i	=	ratio of h_2 / h_1 where h_2 = distance from neutral axis to extreme tension fibers, and h_1 = distance from neutral axis to centroid of the reinforcement
w_{max}	=	maximum flexural crack width, in.
w_{mean}	=	mean flexural crack width, in.
$\sum o$	=	sum of all perimeters of steel reinforcing steel



TABLE 1 — SECTIONAL PROPERTIES OF TEST BEAMS

Beam	Section (in. ²)	No. of Rebars	No.-Size of Prisms	No. of Strands	d (in.)	d _p (in.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
A-1	8×12	3 No.5	0	-	10.125	8.625
A-2	8×12	2 No.5	1-1.5"×3"	1	10.125	8.625
A-3	8×12	2 No.5	1-1.5"×4"	1	10.125	8.625
A-4	8×12	1 No.5	2-1.5"×3"	1	10.125	8.625
A-5	8×12	1 No.5	2-1.5"×4"	1	10.125	8.625
A-6	8×12	0	3-1.5"×3"	1	-	8.625
A-7	8×12	0	3-1.5"×4"	1	-	8.625
A-8	8×12	0	1-3"×3"	1	-	8.625
A-9	8×12	0	1-3"×3"	1	-	8.625
A-10	8×12	0	1-4.5"×3"	1	-	8.625
A-11	8×12	0	1-4.5"×3"	1	-	8.625
A-12	8×12	0	1-4.5"×3"	1	-	8.625
A-13	8×12	0	2-1.5"×3"	1	-	8.625

1 in. = 25.4 mm

TABLE 2 — CONCRETE CHARACTERISTICS OF TEST BEAMS

Beam	Age (days)	Strength f_c' (psi)	Strength f_t' (psi)	w/c+sf Ratio
(1)	(2)	(3)	(4)	(5)
A-1	11	10,240	520	0.30
A-2	10	11,350	560	0.28
A-3	33	14,210	600	0.28
A-4	46	12,300	550	0.28
A-5	26	11,930	540	0.28
A-6	15	12,410	600	0.28
A-7	17	12,100	660	0.28
A-8	23	12,660	700	0.28
A-9	16	12,540	660	0.28
A-10	17	12,900	670	0.28
A-11	8	10,300	700	0.30
A-12	13	12,500	740	0.28
A-13	9	11,190	600	0.30

1 psi = 0.006897 MPa

TABLE 3 — STABILIZED AVERAGE CRACK SPACINGS

Beam	$\frac{f'_t}{\sqrt{f'_c}}$	A_t (in. ²)	Σo (in.)	a_{cs}^t (in.)	a_{cs} (in.)	a_{cs}^t/a_{cs}
(1)	(2)	(3)	(4)	(5)	(6)	(7)
A-3	5.043	43	5.105	4.708	4.725	0.99
A-4	4.987	39	4.319	4.763	4.506	0.94
A-5	4.935	43	4.319	5.393	5.584	0.97
A-6	5.394	39	3.534	6.979	6.190	1.13
A-7	6.028	43	3.534	5.572	6.399	0.87
A-8	6.222	39	2.356	10.46	9.285	1.13
A-9	5.876	39	2.356	10.89	8.706	1.17
A-10	5.917	39	2.356	6.900	8.706	0.79
A-11	6.928	39	3.534	7.000	6.190	1.13
A-12	6.601	39	3.534	7.075	6.190	1.14
A-13	5.682	39	2.356	7.100	9.285	0.76
Mean	5.783					1.00

1 in. = 25.4 mm

TABLE 4 — OBSERVED AND CALCULATED MEAN CRACK WIDTHS

Normalized Bending Moment M/M_u									
Beam	0.3			0.4			0.6		
	w^l	w	w^l/w	w^l	w	w^l/w	w^l	w	w^l/w
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
A-3	2.30	3.46	0.66	4.80	7.30	0.66	8.00	10.4	0.77
A-4	3.20	3.53	0.89	5.30	5.90	0.89	11.6	9.63	1.20
A-5	4.20	4.05	1.04	7.40	6.23	1.18	16.2	12.8	1.26
A-6	1.50	1.31	1.14	1.90	1.74	1.09	3.80	3.09	1.20
A-7	3.20	2.81	1.13	3.90	4.04	0.97	6.80	9.64	0.71
A-8	1.00	1.00	1.00	1.88	1.54	1.20	13.5	12.2	1.11
A-9	0.75	0.82	0.91	2.38	2.25	1.06	3.38	3.18	1.06
A-10	0.83	1.25	0.67	1.17	1.70	0.69	1.90	2.97	0.64
A-11	1.63	1.39	1.17	2.10	2.03	1.03	5.83	6.43	0.91
A-12	1.08	1.00	1.08	2.42	1.92	1.26	5.67	5.45	1.04
A-13	1.13	1.37	0.82	1.92	2.44	0.79	3.75	4.85	0.77
Mean			0.96						0.97

Note:

All Crack widths are measured in the unit of $\text{inch} \times 10^{-3}$ (1 in. = 25.4 mm), w^l and w are the observed and calculated values respectively.

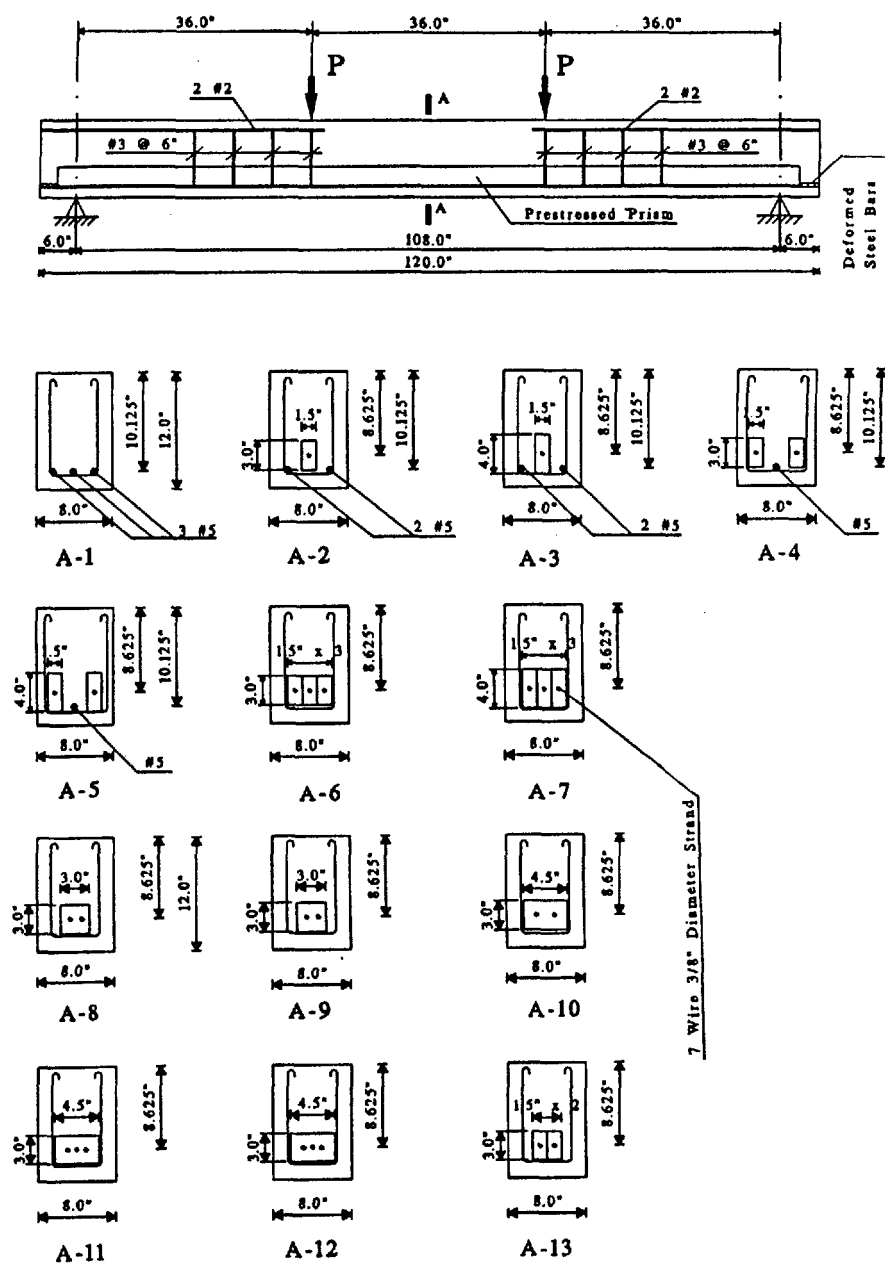


Fig. 1—Cross-sectional properties of tested beams

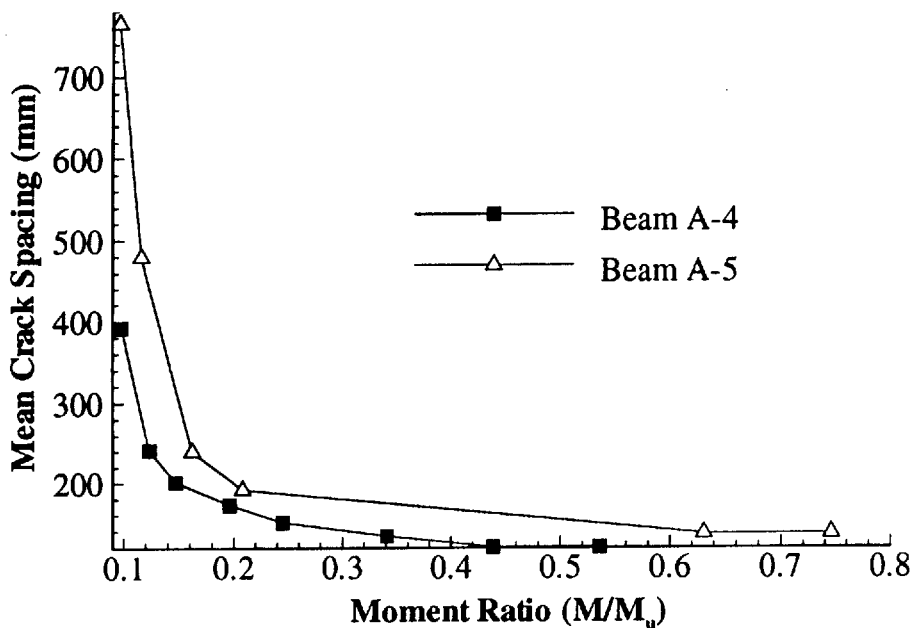


Fig. 2—Typical relationship of applied bending moment versus mean crack spacing (Beams A-4 and A-5)

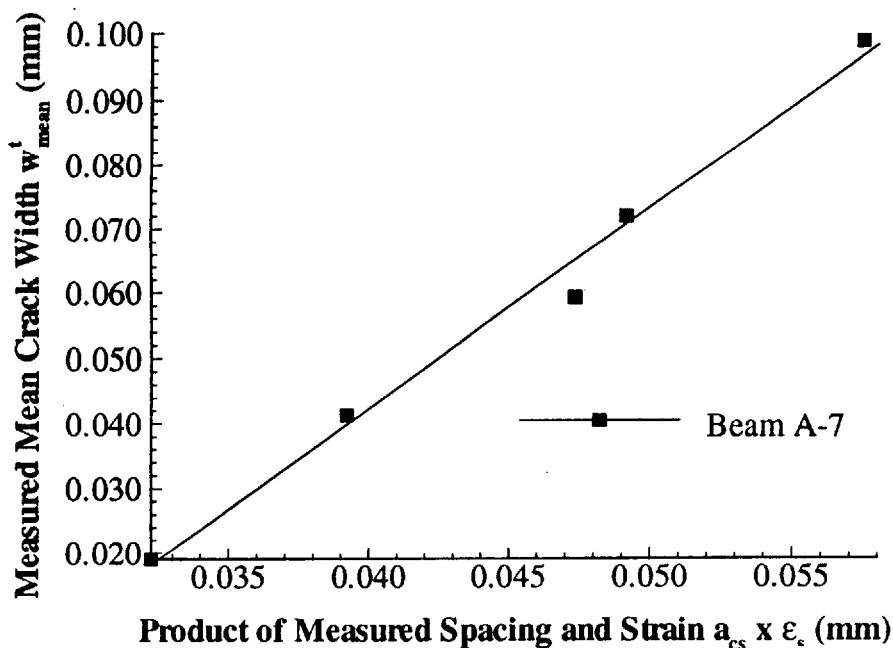


Fig. 3—Typical mean crack width variation with the product of reinforcement strain and crack spacing ($\epsilon_s \times a_{cs}$) for Beam A-7

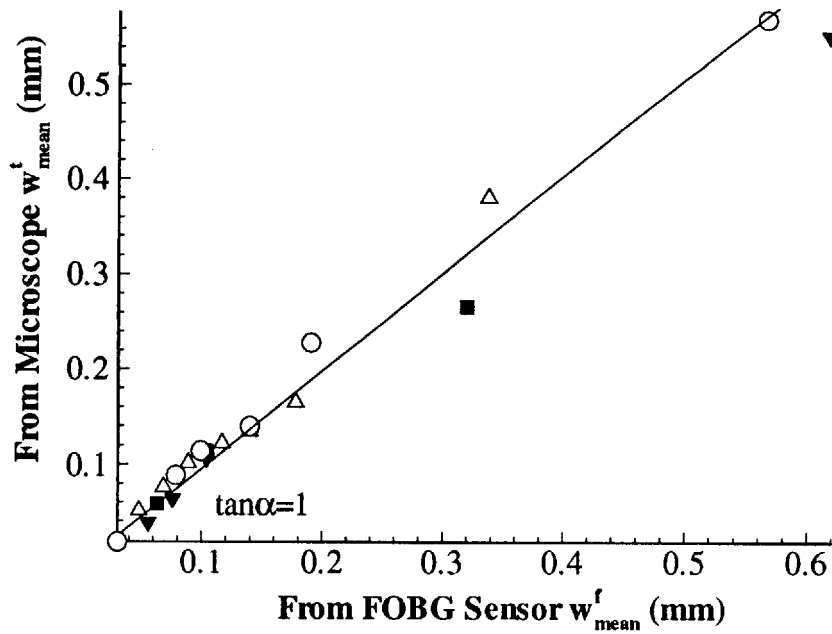


Fig. 4—Mean crack width measurement using FOBG sensors and microscope

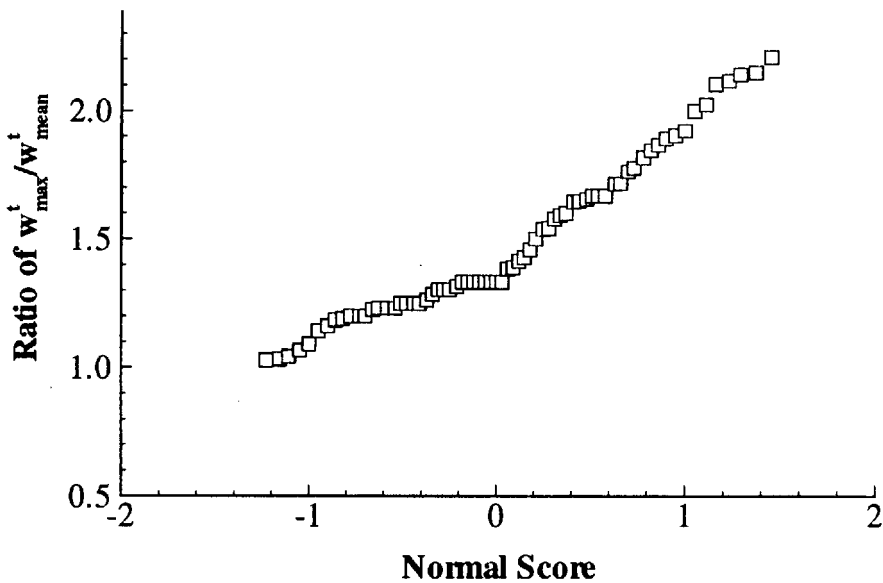


Fig. 5—Normal frequency plot for the ratio (w_{max}^t / w_{mean}^t) for Beams A-3 through A-13 at the reinforcement level

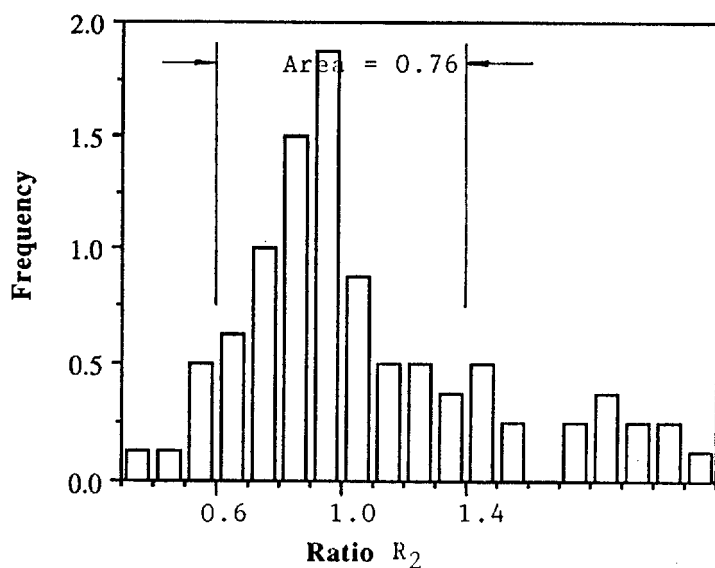


Fig. 6—Relative frequency histogram of observed to theoretical maximum crack width ratio at the reinforcement level (w'/w_{max})

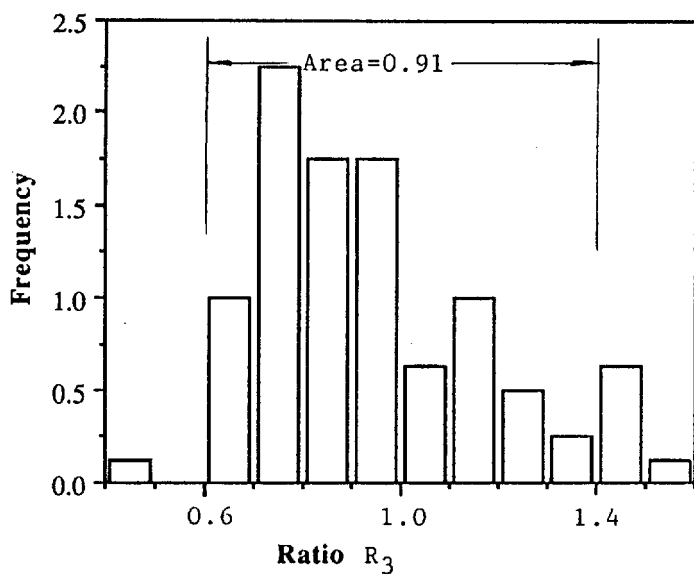


Fig. 7—Relative frequency histogram of observed to theoretical crack width ratio at the concrete tensile face of beam $(w'_{max})' / w'_{max}$

SP 154-3

The Influence of Controlled Permeability Formwork on the Durability and Transport Properties of Near Surface Concrete

by A. E. Long, A. A. Sha'at, and P. A. M. Basheer

Synopsis: The durability of reinforced concrete structures can be improved by resorting to methods which ensure a better resistance of concrete to various aggressive environments. Some commonly used methods include subjecting concrete to a better curing practice, the use of modified concretes and the application of surface treatments on concrete surfaces. In addition to these, efforts have been made in the recent past to develop new techniques by which the water-cement ratio in the near surface region can be lowered and a dense matrix achieved. One way of achieving this is to use a controlled permeability formwork system (CPF), in which the surplus mixing water and entrapped air are removed from the fresh concrete via a fibre liner. This produces a surface layer of concrete with a very low permeability which is likely to be highly resistant to various forms of environmental attack.

Relatively little information is available at present on the efficiency of CPF in improving the protection of the concrete against various mechanisms of deterioration, and how it compares with other techniques such as the application of better curing practices. Therefore, an experimental investigation was carried out with three water-cement ratios, five different curing regimes (i.e. air curing, wet hessian curing and the use of three different curing compounds) and the application of a CPF liner system. Measurements of gas permeability, sorptivity, chloride diffusivity, surface tensile strength, freezing and thawing resistance and carbonation resistance have indicated that the use of CPF can enhance the durability of concrete and the extent of this improvement is significantly more than that obtained for the various curing regimes.

This paper details the experimental programme and presents results which are used to evaluate critically the use of CPF for normal concrete.

Keywords: Carbonation; chlorides; curing; diffusion; durability; formwork; freeze thaw durability; permeability; tensile strength

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INTRODUCTION

The durability of exposed concrete structures to a large extent depends on the quality of the cover concrete ("covercrete" as it is often described) because this provides the first line of defence against various aggressive substances. Thus, the quality of the covercrete is extremely important and for this reason a range of different curing techniques can be applied to improve its durability. Typically, in order to maintain a satisfactory moisture content in the concrete after the removal of the formwork, the surface is sprayed with water at regular intervals or a curing compound/plastic sheet is applied. This ensures that adequate hydration of the cementitious materials takes place but these curing processes are expensive and time-consuming. Therefore, in the 1980s, Japanese engineers developed a permeable formwork system (1) which had the objective of reducing the water-cement ratio in the near to surface region of the concrete. A similar technique was used in 1955 for the main draw off chamber of the Glen Shira dam. In this case, water absorbing Celotex soft boards were attached to the formwork and these were found to be very effective in reducing the near to surface water-cement ratio (W/C). In spite of the fact that the resulting concrete surfaces have proven to be very durable, the process was discontinued as it was difficult to remove the board from the hardened concrete.

Within the last few years, a fabric formwork liner has been developed which gives comparable benefits to the Japanese permeable formwork system at a significantly reduced cost. The idealised behaviour of this controlled permeability formwork (CPF) liner is illustrated in Fig. 1. During the vibration of the concrete, the entrapped air and the surplus mixing water escape, via the CPF liner which acts as a vertical conduit. Provided the excess water is allowed to drain freely at the bottom of the formwork then a surface with improved properties and an absence of blow-holes is produced. As this product has only been commercially available for four years, relatively little detailed technical information has been available up until recently on the effectiveness of this liner