

Four Decades of Progress: Personal Perspective

by A.E. Naaman

Synopsis: This paper reviews progress spanning a period of about four decades during which the author was intimately involved in research and teaching in three distinct yet related fields of civil engineering: prestressed concrete, fiber reinforced concrete, and ferrocement and thin cementitious products. In retrospect and for each area, key contributions are mentioned, milestones recalled, and prospects for the near future envisioned. Issues related to partial prestressing, external prestressing, high performance fiber reinforced cement composites, strain-hardening FRC composites, 3D textiles and the like are addressed. Technical advances are webbed with some personal milestones as well. Important research issues to address in the near future are pointed out.

Keywords: 3D textiles; external prestressing; ferrocement; fibers; fiber-reinforced concrete; HPFRCC; partial prestressing; prestressed concrete; SIFCON; strain-hardening; strain-softening; textile reinforced concrete.

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BRIEF BACKGROUND ON EDUCATIONAL AND PROFESSIONAL LIFE

I was born in Beirut, Lebanon, and graduated from Notre Dame high school in 1959 with a French Baccalaureate 2nd Part, mathematics option. I then went to Paris, France, first to College Stanislas (1959-1961) to prepare for engineering, and then to Ecole Centrale of Paris (ECP) where I obtained a diploma in Engineering in 1964. I continued for a specialty degree in reinforced and prestressed concrete at the CHEC in Paris in 1965. Following 4 years in consulting engineering practice, first in Lebanon for 1 year, and then in Montreal, Canada, I enrolled at the Massachusetts Institute of Technology (MIT) in Cambridge in the Fall of 1969 to pursue graduate studies. I obtained my MS degree in 1970, and my PhD degree in 1972 from MIT, both in civil engineering. It is during this period that I decided to pursue an academic career. I joined first California State University in Los Angeles for the 1972-1973 academic year, then the University of Illinois at Chicago from 1973 to 1983, and then the University of Michigan in Ann Arbor from 1983 until my retirement in 2007.

MY STORY WITH PRESTRESSED CONCRETE

When young, there was no question that I would become an engineer. I dreamt of airplanes and aeronautics. I built flying airplanes models using balsa wood. My first serious toys were the Meccano systems of punched steel plates, rods, bolts, and screws, used to build models of everything and spark imagination. When I was ten, my Christmas gift was a subscription to *Système D*, a French monthly magazine describing “handy-man” special projects and ideas, the equivalent of self-help *Popular Mechanics*, wood-working, and the like. I used to fiddle with everything, trying to understand how and why. After graduation from high school (Notre Dame of Jamhour, Lebanon, run by the Jesuits) I went to Paris, France to study engineering, a 5-year program. The fifth year is when students have to choose a specialty. Being of Lebanese descent, it would have been hard to choose Aeronautics, my first choice. Much of the practical work, labs, training, and technical visits to manufacturing sites required special clearance from the French government. Also, there was no aerospace industry in Lebanon or the Middle East. Upon consulting with my father and reevaluation of my future, I decided to take Civil Engineering, a specialty that would allow me total freedom to practice my profession anywhere in the world.

Following graduation with a Diploma in Engineering from Ecole Centrale des Arts et Manufactures (ECP), in Paris, in 1964, I selected to specialize in what

was considered then the most technically advanced subject in structural engineering, and where the French were considered ahead in the field. That was Prestressed Concrete. Thus I enrolled in the CHEC (Center for High Studies in Construction), section CHEBAP (Center of High (advanced) Studies in Reinforced and Prestressed Concrete) in 1964, and earned a specialty degree in 1965, that is equivalent to a U.S. Master's degree. For prestressed concrete, we used copies of manuscripts hand-written in French by Y. Guyon, a pupil of Freyssinet. My first job was in Lebanon in a newly created governmental agency baptized "Executive Council for Large Projects," where I was assigned the supervision of a highway segment with two RC bridges. Since the job required mostly supervision with little design and analysis and since I complained to my boss that I was not using my knowledge, he suggested to me to prepare an internal document to introduce prestressed concrete to the division where I worked. This is when I prepared my first analysis-design notes on prestressed concrete, entirely based on the documents I had collected during my study of the subject in France. This is also, I believe, when I discovered the pleasure of writing, synthesizing thoughts and information, and possibly teaching.

In 1966 I left Lebanon bound to France and Canada, where I worked with a company (Potenco Inc.) representing the Freyssinet prestressing system in Quebec. My field assignment (at Habitat 67 in Montreal) involved the supervision of prestressing operations on site, with particular attention to the sequence of prestressing and prestress losses. This is where I developed a much better understanding of prestress losses and how to minimize their effects. My position in the field eventually led to an office position as a design engineer with a medium size consulting engineering firm then called Lalonde Valois Lamarre Valois and Associates. It later became one of the largest consulting firms in Canada (snc-Lavalin). My first assignments included mostly the design of footings and retaining walls, then reinforced concrete elevated structures and tunnels of the Trans-Canada Highway segments going through Montreal. To improve my knowledge, I took evening extension courses at McGill University in the Fortran language and Numerical Analysis. My love of prestressed concrete pushed me to select, as a project for the course, the writing of a computer program for the analysis and design of simply supported prestressed concrete beams. This was the time where each command required a punch card and there was little room for error. However, my program eventually worked very well and soon it was noticed by Professor Alfio Seni, from the Ecole Polytechnique of Montreal, who did consulting work for the company, and whom I approached to discuss some unusual findings. I was thereafter reassigned to work on a new project dealing with the analysis and design of precast prestressed concrete highway bridges for the province of New Brunswick. While working on this project, I improved my program to include optimum design as well as analysis and design of prestressed beams for service and ultimate limit states; I then developed design charts utilizing the then AASHTO-PCI standard prestressed concrete I-beams in composite bridge decks. This work eventually led to my first paper on prestressed concrete which was published in the *PCI Journal*, in the Jan.-Feb. issue in 1972.

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I would like to note that when I completed my specialty degree in prestressed concrete at the CHEC in Paris, in 1965, there were very few books on the subject of prestressed concrete. In France, we used the notes of Guyon, which were initially published in 1953 and supplemented by many handed-out manuscripts. I was not aware of any other French published book on the subject and Guyon's work was considered (with understandable French bias) the only true and tested solid source of information and knowledge on prestressed concrete. Later when I moved to Canada, in 1966, I searched for other books on prestressed concrete to further my knowledge and found the books by Magnel (*Prestressed Concrete*, third edition, 1954), Evans and Bennett (1958), Libby (1961), Lin (*Design of Prestressed Concrete*, 1963), Abeles (*An Introduction to Prestressed Concrete*, 1964), Leonhardt (*Prestressed Concrete Design and Construction*, English translation, 1964), and Preston and Sollenberger (*Modern Prestressed Concrete*, 1967). I purchased them all for my own learning. There were many similarities in their introduction and examples of applications, but clearly the subject was ripe and ready to expand with increasing research and applications. Before my PhD graduation in 1972, three other books were published in English, one by Mikhailov from Russia (1969), one by Katchatourian and Gurfinkel (1969), and one by Gerwick (1971). During the 10 years I taught at the University of Illinois in Chicago, that is, between 1973 and 1983, additional books on prestressed concrete became available including those by Ramaswamy (1976), Libby (second edition, 1977), Nilson (1978), and Warner and Faulkes (1979).

I first taught a graduate level course on Prestressed Concrete at the University of Illinois in Chicago in 1974, and then offered it practically on a yearly basis. I quickly realized that much of the fundamental knowledge and thinking I had learned during my studies in France needed to be adapted to U.S. approaches, economic conditions, the need to simplify, and field experience. Precast-prestressed concrete was way ahead in the U.S. while post-tensioning was the norm in France. I also realized that the U.S. experience could gain from the rationality of French analysis and design approaches. This led me to undertake the writing of a textbook on prestressed concrete, first published in 1982, and at the same time allowed me to accumulate a broader knowledge and expertise on the subject.

In the meantime

During my engineering studies in France, I met Ingrid Schneider (from Berlin) in 1964 and we got married in 1967, while in Montreal, Canada. There, while working as a structural engineer, I felt the need to seek additional knowledge in my field. In the Fall of 1969, I joined the Civil Engineering Department at MIT which was then famous for developing the new structural analysis software "STRESS." However, my intent was to pursue my expertise in prestressed concrete. Unfortunately, upon my arrival, Professor Pahl, who was responsible for prestressed concrete at MIT, had left for Germany and there was no course on prestressed concrete. This is when I met then visiting Assistant Professor S.P. Shah, who introduced me to his dream subjects of Ferrocement

and Fiber Reinforced Concrete. I had never heard of them before and, at first, they did not seem too exciting to me; but I needed the research assistantship and thus I joined the team, did my best, and got hooked.

I never took a course on prestressed concrete at MIT or in the U.S. However, my love of the subject subsided. When I joined the University of Illinois at Chicago in September 1973, I was asked what new courses I would like to teach, and I immediately mentioned Prestressed Concrete. It was promptly approved and the next year I taught a graduate level course on prestressed concrete for the first time. I then got involved in the PCI's technical activities and won a Student Fellowship award in 1975 to study Partially Prestressed Concrete. Essentially it was about 10 years since I had completed my specialty in prestressed concrete in France, and so doing research on the subject was to become the beginning of a long dream overdue. Thus started the real foundation of a productive work that eventually led to numerous contributions with my students to the field of prestressed concrete.

We started working on partial prestressing about 1975 and eventually published a paper in 1979 on the analysis and design of partially prestressed beams which won the ASCE T.Y. Lin award and the PCI Martin P. Korn award (Fig. 1 and 2). I also introduced a second graduate level course on advanced prestressed concrete in which I taught subjects related to partial prestressing, optimum design, unbonded tendons and the like. I joined the University of Michigan in 1983 and I continued offering two graduate level courses on prestressed concrete, one meant primarily for Master's students and the other for Doctoral students. In the second course, I taught additional topics on nonlinear analysis, analysis and design with unbonded tendons in the cracked and ultimate limit states, and external prestressing. Following collaboration with Professor J. Breen from UT at Austin, and Michel Virlogeux from France, I co-organized the first symposium on External Prestressing in Bridges during the ACI Fall convention in Houston, in 1988.

Partial prestressing, external prestressing, and the use of unbonded tendons were becoming common practice in prestressed concrete. With my students we conducted a number of studies focusing on their analysis and design and we published a number of related papers (Fig. 3) including how to predict the stress in unbonded prestressing tendons at ultimate (Fig. 4). These served as a basis for several sections related to bending and compression members in the AASHTO LRFD Bridge Design Specifications first published in 1995.

During the late 1980s and early 1990s, fiber reinforced polymeric reinforcements became available for use in reinforced and prestressed concrete structures. I immediately got interested in the subject but quickly realized the limitations on ductility and shear resistance when using such reinforcements. While my fears were later confirmed in extensive experimental and analytical studies, the experience we gained helped us shift our focus on repair and strengthening using such materials.

Prestressed concrete: looking ahead

I believe that the following statement made by Freyssinet in the 1950s remains

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valid today: *“There exists no field of structural activity, and I say none, after careful thought, to which the idea of prestressing does not provide the possibility of solutions frequently unforeseen.”*

Since the time of Freyssinet, prestressed concrete has advanced far beyond the developmental stage and has established itself as a major structural material. Similarly, prestressing techniques have evolved into a reliable technology. Prestressed concrete has made major contributions to the construction and the cement industries. It has led to an incredible array of structural applications from bridges to nuclear power vessels and offshore structures. Seldom is a major project planned today without considering prestressing as one of the viable alternative solutions. Moreover, some innovative large scale structures could not be conceived without prestressed concrete.

A careful analysis of prospects and opportunities indicates that the future of prestressed concrete is very bright. Constructed facilities and infrastructure systems will keep expanding not only in volume but also in reach. To achieve daring limits and efficiency, prestressed concrete will be needed. This is true for terrestrial structures as well as marine structures, underwater structures, underground structures, space structures, and structures needed to support new technologies such as wind turbines and solar towers.

New technologies will also open new directions for prestressing. Shape memory materials may one day become sufficiently cost effective to be used in self-stressed prestressed concrete applications. Imagine even fibers and microfibers randomly and uniformly distributed in a concrete matrix acting as stressing reinforcement at all levels.

Current market penetration for prestressed concrete in the United States is estimated at about 10%, and it is believed that a penetration of 30% will be achieved eventually. The same trend is expected in the future at the educational level. Design courses in prestressed concrete will very likely move from the list of elective courses to the list of required courses in structural engineering curricula.

Prestressed concrete structures contain all the beneficial attributes of concrete as a construction material plus those inherent to prestressing. The current widespread use of prestressed concrete in developed countries is astonishing. Because of its many advantages, particularly its durability, it is expected to remain one of the strongest construction systems on the market for the foreseeable future.

Knowing too much on a given subject can blur innovation. Although it is generally simpler to measure achievements in terms of longer bridge spans, “longer span” is not necessarily the most important or critical research issue today for prestressed concrete. Some more rewarding and exciting topics include the use of fiber reinforced concrete matrixes in combination with prestressed concrete structures (precast, pretensioned or post-tensioned), the use of concrete matrices that are not only of higher strength but also of significantly higher durability (such as ultra high performance fiber reinforced concrete) and more impervious to various environmental attacks; the

possible use of advanced fiber reinforced composites to replace prestressing strands; the development of shape memory materials for self-stressing; and seismic applications.

MY STORY WITH FERROCEMENT

The use by Joseph Louis Lambot of ferrocement to build a small boat in 1849, and the patent granting that followed in 1855 for “Fer-ciment” represent in effect the birth of modern reinforced concrete. While reinforced concrete took off thereafter at a rapid pace, ferrocement saw a period of stagnation with very slow progress for almost 100 years. Only in the 1940s and 1950s did Pier Luigi Nervi of Italy recognize the possible advantages of ferrocement not only for boat building but for terrestrial applications as well, and carried out some experiments on its mechanical properties. He reported that ferrocement showed exceptional elasticity, flexibility, strength, and resistance to cracking. Technical reports to support these statements, with sufficient details as would be expected with similar reports today, were simply not available, not synthesized, or were written in a different language. Significant interests by mostly amateur boat builders in ferrocement and its applications grew in the early 1960s, particularly in the UK, Canada, New Zealand and Australia. Some activities were also reported in Russia and China. Many fisheries were interested in the new material. Reports of experiences with ferrocement by boat builders were available in publications targeted to boat building and fisheries. These were largely anecdotal in nature (such as this boat survived a 15 knots wind and 20 ft waves), and provided very little technical information. The notion that ferrocement is a crack free material was common in these reports. The extension of the use of ferrocement to terrestrial structures was not yet clearly recognized and was greatly hindered by the lack of clear analysis, design methods, design criteria and guidelines.

Prior to joining MIT in 1969, I had never heard of ferrocement. I went to MIT hoping to further my graduate studies in prestressed concrete; however, since Professor Pahl, who taught prestressed concrete, had left (back to Germany) just prior to my arrival, I turned down the next topic offered to me for research (structural reliability) and took on the subject of ferrocement and fiber reinforced concrete introduced to me first by none other than Suru Shah, then visiting assistant professor at MIT. As part of my Master’s thesis, I started a comprehensive experimental program on the mechanical properties and reinforcing mechanisms of ferrocement in tension. However, I could find very little solid technical information. Even the engineering library at MIT and its search and borrowing resources could hardly locate more than a handful of references on the subject. I dismissed anecdotal information suggesting that ferrocement does not crack and is impervious to water, and eventually realized the importance of the specific surface of reinforcement and its effect on reducing crack width and spacing. I thoroughly enjoyed understanding the significant influence of this relatively new parameter. My M.S. thesis, completed in 1970, was perhaps the first fundamental research study on the tensile behavior of ferrocement and its modeling.

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Increased scientific approach to studying and predicting ferrocement properties was encouraged by a panel of the US National Academy of Sciences, Chaired by J. Romualdi, formed in 1972, and whose report was released in 1973. This led to the formation of the American Concrete Institute's Committee 549 on Ferrocement, in 1975, and the establishment, shortly thereafter, in 1976, of the International Ferrocement Information Center (IFIC) at the Asian Institute of Technology in Bangkok, Thailand. Publication of the Journal of Ferrocement (which started as a publication by the New Zealand Ferrocement Marine Association) was then consolidated at IFIC. A RILEM scientific committee on ferrocement was later formed in 1979. Progress accelerated during the 1980s through fundamental research, publications, symposia, short courses and applications

I served as Chair of ACI Committee 549, Ferrocement, starting in 1981 over a period of 6 years. During my tenure the committee produced the first "State-of-the-Art Report on Ferrocement." In a second phase, in 1988, the Committee developed the first "Guide for the Design, Construction and Repair of Ferrocement," which gained instant success worldwide among practitioners using ferrocement and became the basis of many local codes on ferrocement. The guide contained two chapters, one on design criteria, and one on testing, almost entirely based on research work I had carried out with my students.

The International Ferrocement Society (IFS) was founded in 1991, under the Chairmanship of Rick Pama from the Asian Institute of Technology, and I was one of the founding members. Its main objectives were to foster development, disseminate knowledge, and encourage practical applications of ferrocement, particularly terrestrial structures. The most urgent need of this new professional society was to develop a building code for ferrocement. I was selected to chair its committee, and the first Ferrocement Model Code was published in 2001, essentially setting the stage for guided expansion, imagining and developments.

I never lost enthusiasm for ferrocement. I participated in many related conferences and symposia, the first such symposium dedicated to Nervi, in Bergamo, Italy, 1979, co-organized by Oberti and Shah. About two decades later, in 1998, I organized the 6th International Symposium on Ferrocement, which was dedicated to Lambot. I have been closely involved with the organization of the Seventh, Eighth, and incoming 9th Symposium on the subject. Feeling the need to synthesize the knowledge I had gained on ferrocement and the urge to disseminate that knowledge, I wrote the first existing textbook on the subject, *Ferrocement and Laminated Cementitious Composites*, which was published in 2000.

Ferrocement: looking ahead

What are the near-future prospects for ferrocement materials, applications, and technology? It should be noted that many advances in materials and technology as summarized next simultaneously influence ferrocement and fiber-reinforced concrete.

We are witnessing a considerable evolution in materials and materials technology. At the matrix level, while the basic cement and sand components remain the same, an increasing number of additives are becoming available. These additives allow for many properties to be easily obtainable, such as improved workability, flowability, setting time, strength, bonding, high durability, and high impermeability. Mortar compressive strengths exceeding 100 MPa (14.5 ksi) can be readily achieved and will be used. High performance and ultra-high performance cement matrices, containing finer particles such as silica fume, glass powder, and the like will be increasingly used in cost competitive applications.

To decrease the reinforcement, improve efficiency, and reduce labor cost the idea of replacing several layers of mesh reinforcement in a ferrocement by only two extreme layers, while adding fibers to the matrix, was shown to be technically effective, provided the mesh reinforcement is of high strength and high modulus (Fig. 5). To further reduce labor cost, the idea of using a fiber mat as a core spacer between the two extreme layers of mesh, thus forming a sandwich-like self-contained reinforcing system was explored and also proven to be effective, performance and cost wise (Fig. 6). At the reinforcement level, while steel meshes remain the primary reinforcing material, other materials such as carbon fiber meshes and organic synthetic meshes offer some advantages, particularly with respect to corrosion resistance and weight (Figs. 6 and 7). In industrialized countries, including the U.S., synthetic meshes such as polypropylene, polyvinyl alcohol (PVA), carbon, Kevlar, and polyethylene (Spectra) meshes can be produced to specification and delivered to site, some at a cost competitive level with steel or organic natural meshes (sisal, jute, bamboo). Three-dimensional (3D) meshes (fabrics, textiles) of either steel or polymeric materials or hybrids will become increasingly available. Examples suitable for ferrocement are shown in Fig. 8. While 3D meshes have taken a separate path as part of textile reinforced concrete, they will be most competitive in thin reinforced cementitious products such as ferrocement. Since a single 3D mesh may replace several layers of plain two-dimensional (2D) mesh, substantial savings in labor cost are expected. Further savings can be achieved when the mortar matrix is mechanically applied as in shotcreting, extrusion, or pultrusion. Also, given modern trends in manufacturing, it is likely that robotics techniques will be applied to the production of ferrocement precast elements, eventually leading to substantial reduction in labor cost. The use of 3D textile using polymeric fibers will also see increasing applications in products using lightweight cement matrices.

Also, with the increasing concern about green environment and ecology, there will be growing efforts to use natural fibers (jute, fique, bamboo) in the form of textiles for ferrocement applications, particularly in terrestrial structures (housing, agricultural structures) and in combination with lightweight cement matrices. Treating the textile fiber or the cement matrix to improve durability remains a key obstacle for taking full advantage of this solution.

At the other extreme, stronger reinforcements, whether steel or polymeric

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such as carbon or aramid, will become common in ferrocement and thin cement-based products. Current examples, using steel, include Fleximat® from Bekaert and Hardwire® which provide tensile strengths in the range of 2400 to 3100 MPa (348.1 to 449.6 ksi) (Fig. 9). These will be particularly compatible with high performance and ultra-high performance cement matrixes. Figure 10 illustrates the bending response of (0.5 in. [12.5 mm]) thick ferrocement plates using only two layers of these high strength steel systems and fibers where equivalent elastic bending strength or modulus of rupture can exceed 130 MPa (18.9 ksi). Given that the total volume fraction of reinforcement in these composites, including the fiber, is less than 3.7% and given that with conventional steel wire mesh reinforcement, a modulus of rupture of only about 50 MPa (7.3 ksi) could be achieved at about 8% total reinforcement, it can be observed that a four-fold increase in performance is achievable.

Applications of ferrocement in small size structures and structural elements have mushroomed in developing countries. Examples include water tanks, grain silos, domes, barges, earth shelters, floating wharves, pontoons, roofing beams, and oil tankers. In a way, ferrocement has become an “all purpose” material for thin products and its potential combination with other materials (such as fibers and prestressing) is a testimony to its versatility. It is believed that increased utilization of ferrocement will continue. In industrialized countries this may take the form of mechanized production of small size elements such as cement sheets and pipes to replace asbestos cement products. The housing market has become most suitable at this time for the use of cladding, roofing and exterior skins made out of ferrocement.

The technology of ferrocement will continue to span a very wide range, from a fairly simple technology requiring only a small investment in tools and equipment, to a very advanced technology where robots will replace labor at a fraction of the cost.

While existing codes and guidelines for reinforced concrete offer a good starting point, there will be increasing needs for improved guidelines and codes specifically geared toward ferrocement and thin cement-based products. In the U.S., for instance, fire resistance requirements regarding minimum concrete cover to the reinforcement do not and cannot apply to ferrocement. Yet such requirements hinder its use. On the “analysis-design” side, in the future, particular attention should be given to simplifying the analytical involvement, providing design aids and reducing the total effort devoted to the “analysis-design” process. As the majority of builders of small ferrocement structures are non-technical, it is not realistic to assume that they will be able and willing to follow complex design methodologies. Some progress toward simplification has already been made through the Ferrocement Model Code published by the International Ferrocement Society. It should facilitate the acceptance and implementation of ferrocement throughout and pave the way for unhindered future developments.

As with many structural materials, increased educational activities and increased applications will move in steps, and will be strongly correlated. It is

likely that the subject of ferrocement will find its way into courses related to reinforced concrete, plates and shells, and brittle matrix composites.

MY STORY WITH FIBER REINFORCED CONCRETE

During my early graduate studies at MIT, I quickly understood the importance of strength versus ductility and stiffness, nominal stress versus stress concentration, and fracture toughness and fracture sensitivity. My doctoral thesis dealt with developing a rational theory to predict the tensile strength of fiber reinforced concrete. At the time only plain circular steel fibers were available and their bond to the concrete matrix was very poor. This led to very poor post-cracking strength and in a way little structural ductility. In a separate research term project for a course, I tried to imagine a method to improve fiber bond and toughness. I came up with a tri-dimensional fiber structure that looks like two footballs placed in line with each others axis and linked along their axis. In one configuration, the structure of the fiber consisted of four wires, one straight positioned along the axis of the football, and 3 surrounding wires forming the football skeleton (Fig. 11a). They were all connected at the link by twisting the wires like a cord. I then built a hand driven machine to fabricate the fibers and I tested them in small mortar beams. The fibers could not be premixed; but they were placed into a mold until its full capacity and then a mortar matrix was poured in. Fiber content varied between 1.35 and 3.1% by volume. The test beams led to superior performance in terms of strength, multiple cracking and deflection. A U.S. Patent (No. 3,852,930) was then filed by MIT and granted.

SIFCON and HPFRCC

About 10 years later, Lankard and co-workers introduced a manufacturing process for fiber reinforced concrete called SIFCON (Slurry Infiltrated Fiber Concrete), a process by which straight steel fibers (single plain or deformed fibers) were placed into a mold up to its full capacity and then infiltrated by a cement-based slurry. This process was similar to the one I used with the 3D fibers, but more complex because the cement slurry was to be fine enough to infiltrate the very dense fiber network of high fiber content, made of straight steel fibers. During the 1980s, with several students, we worked extensively on the mechanical properties of SIFCON, particularly compression and tension. SIFCON provided unique tensile behavior with multiple cracking and strain-hardening after cracking (Fig. 11b), and a record performance among all types of fiber reinforced concretes in terms of strength, ductility and toughness (Fig. 12). However it used a relatively large volume fraction of fibers ranging, at the time, from about 7 to 15% by volume. SIFCON was then extensively used in numerous defensive and protective structures by the Department of Defense. It is based on my better understanding of SIFCON tensile properties that I introduced in 1987 the concept of High Performance Fiber Reinforced Cement Composites (HPFRCC) which were defined as those developing a stress-strain response in tension with a post-cracking strength larger than their cracking strength and with multiple cracking occurring following first cracking. The figure I used is reproduced in

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Fig. 11b. Later on, such behavior was termed “pseudo strain-hardening,” “quasi strain-hardening,” or simply “strain-hardening” behavior and is now a generally accepted term among researchers. A discussion that led to the closure of the 4th International Workshop on High Performance Fiber Reinforced Cement Composites (in Ann Arbor, Michigan, 2003) led to a recommended simple classification of all fiber reinforced cement composites according to their tensile behavior as either strain-softening or strain-hardening; such behavior is illustrated in Fig. 13.

SIFCON demonstrated that strain-hardening in tension with multiple cracking can be achieved by FRC composites, without adding reinforcing bars. Post-cracking tensile strength as high as 30 MPa (4.4 ksi) was achieved, with tensile strain capacity exceeding 1% (Fig. 12b). In compression SIFCON achieved 210 MPa (30.5 ksi) with a strain capacity at peak of about 10%. One drawback of SIFCON was the high volume fraction of fibers needed and their related cost. Again Lankard and co-workers imagined a prefabricated mat made with long steel fibers. The mat would be preplaced in a mold and infiltrated by a cement slurry. SIMCON (or Slurry Infiltrated Mat Concrete) was born. One advantage of SIMCON is that it led to a volume fraction of fibers ranging from about 4 to 7% (significantly smaller than SIFCON) while achieving competitive strength and toughness. Additional work later by Krstulovic-Opara and others showed that while, overall, the performance of SIMCON was not as good as that of SIFCON, it offered an excellent trade-off when performance to cost ratio was considered.

While developments at the material level were progressing during the 1980s, many researchers attempted to use fiber reinforced concrete in structural applications in combination with conventional reinforced concrete. The use of SIFCON for example, in combination with conventional reinforcing bars, led to superior (but also very costly) reinforced concrete structures which were mostly used by the department of defense for protective structures. In 1990 I was awarded a Senior Scientist Award by the Alexander von Humboldt Foundation from Germany to work with H.W. Reinhardt. We initially selected to work on the influence of SIFCON in applications with conventional reinforced concrete beams; this was the beginning of a continuing and productive collaboration which led to the organization of five international workshops on High Performance Fiber Reinforced Cement Composites, between 1991 to 2007. During the late 1980s, Bache from Denmark introduced Compact Reinforced Concrete, a concrete reinforced with a high reinforcement ratio of reinforcing bars in both tension and compression, and simultaneously reinforced with a relatively high volume fraction of short steel fibers. The resulting beams showed an almost elastic-plastic response up to failure, exhibiting both high resistance and high ductility.

Since the first introduction of modern fiber reinforced concrete in the US by Romualdi, Mandel, and Batson, in the early 1960s, the most common method of production was to premix the fibers with the concrete. Such a method, even with today's advances and modern fibers, allowed the proper premixing of steel fibers in amounts not exceeding about 2% by volume, in a conventional concrete with

aggregates. In the lab, we could go up to 4% provided smaller length fibers were used with a fine mortar matrix. However, there were many trade-offs to consider. For instance, a longer deformed steel fiber is more efficient, bond wise, than a shorter plain steel fiber. Yet the longer fiber is more difficult to mix and tends to segregate and form balls more easily than the short fiber. During the late 1980s and beginning of 1990s, it had become clear that “strain-hardening” behavior in tension is essential for the full success of fiber reinforced cement composites, not only in stand alone applications but also in structural applications where fiber reinforced concrete can be used in combination with conventional reinforced and prestressed concrete.

The NSF Center for Advanced Cement Based Materials was founded in 1989 and was a consortium of five institutions comprising Northwestern University, the University of Illinois at Urbana Champaign, Purdue University, the University of Michigan, and the National Institute of Standards and Technology. One of the focus areas of ACBM was fiber reinforced concrete and thus, there were continuous requests by industrial partners for composites with better performance. Extensive studies on bond, toughness, mechanical properties, characterization, and the like were carried out.

TOREX Fiber

Shortly after joining the University of Michigan, I introduced a graduate level course on Fiber Reinforced Cement Composites, first taught in 1985. I believe it may have been the first such complete course in a US university. The course covered ferrocement, laminated cement composites with continuous reinforcement, and fiber reinforced concrete with discontinuous fibers. The course was generally taught once every two years. In it, I covered the mechanics of how to predict cracking and maximum post-cracking strength in fiber reinforced concrete under tension as essentially developed in my PhD thesis, in 1972. The methodology assumed round fibers and the resulting prediction equations used the fiber aspect ratio, L/d , that is, length divided by diameter, as one of the causal variables. One of the questions that was sometimes asked by students was: what would happen to L/d if the fiber is not round? and my answer was that we could derive an equivalent diameter from the fiber cross sectional area and use that as a first approximation. During the Fall of 1991, when teaching the course, I decided to put my teaching notes in typewritten form. I decided then to solve the case of “non-circular” fiber as a special case. However, it became soon evident to me that the “non-circular” fiber should be the general case and the circular fiber a particular case of it. The formulae revealed that a most important factor is the ratio of fiber lateral surface area per unit length divided by the fiber cross-sectional area, which I later termed the FIER (Fiber Intrinsic Efficiency Ratio). For circular fibers, FIER becomes proportional to the fiber aspect ratio. Soon I realized that polygonal fibers (triangular, square, rectangular) have higher values of FIER than circular fibers of same cross section, and thus are theoretically more efficient, bond wise; the formulae were clearly confirming this result; moreover, thin flat fibers would be most efficient, but not practical since they are more difficult to mix and also since they provide too much

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interruption in the continuity of the matrix. So I focused attention on fibers of square and triangular cross section and observed that triangular fibers would be about 28% more efficient than circular fibers of same cross section. I also worked on “substantially square” or “substantially triangular” cross-sectional fiber shapes that showed theoretical FIER values 200% to 300% those of circular fibers.

Extensive prior investigation on the pull-out behavior of fibers from concrete had revealed that frictional and adhesive bonds were much less effective than mechanical bond, such as in the case of hooked ends fibers or deformed fibers. So it was obvious for me to imagine that polygonal fibers could also benefit from a mechanical component of bond. Since twisting is a good way to anchor a screw or a non-circular nail, the idea of twisting the fiber came naturally. Hence, what I called later the Torex fiber for identification was conceptually born (Fig. 14). The theoretical findings on the importance of the FIER and on twisting were then revealed in a confidential seminar in September 1994 to a fiber producing company, with the intent to launch a new family of fibers. But no interest was shown; the main reason given was that it would be difficult to produce the Torex fiber and there were insufficient experimental results to demonstrate its theoretically “predicted” superior performance. It took an additional three to four years before the results could be demonstrated first from pull-out tests and then in small scale fiber reinforced concrete specimens. Experimental tests revealed that the behavior of Torex fiber during pull-out depended on numerous parameters including the fiber twisting ratio, and the fiber and matrix mechanical properties. The most important new observation was that the parameters can be tailored in such a way that the fiber un-twists during pull-out providing enormous amount of energy (three to four times what the best fibers on the market could achieve) while maintaining a high pull-out resistance. The results were so impressive and convincing that a patent was filed by the University of Michigan and granted in 1999 almost seven years after initial conception. An additional provisional patent followed in 2000. Three doctoral students should be credited for the extensive experimental work on the bond properties and performance of Torex fibers: Jamil Alwan, now Chief Engineer at Ford Motor Co., Patricia Guerrero, now Professor at Universidad del Valle in Cali, Colombia, and Chuchai Sujivorakul, now Associate Professor at King Mongkut University in Bangkok, Thailand.

The performance of the Torex fibers opened new opportunities for fiber reinforced concrete. Extensive tensile tests on FRC specimens revealed that strain-hardening behavior in tension using Torex fibers could be achieved with a volume fraction less than 2% and in some cases close to 1%. Strain capacity up to 1% in direct tension was possible at stresses 2 to 3 times the strength of the non fiber reinforced matrix. Moreover, crack widths prior to maximum strain and localization were less than 50 microns, implying a good service and durability performance of the composite whether used alone or in combination with reinforced and prestressed concrete. The above two results have greatly simplified the way to possible large scale application of fiber reinforced concrete

in both structural and not structural applications. With mixing constraints of higher volume fractions of fibers for ready mix trucks taken into consideration, the use of self-consolidating fiber concrete became the next challenge.

Self-consolidating FRC

Since it was possible to obtain strain-hardening FRC composites with less than 2% steel fiber by volume, and since premixing more than 2% steel fiber (of length exceeding 30 mm) by volume was quite difficult, the next objective was to focus on simplifying the mixing and site delivery process. One of the most promising ongoing developments is the production of a self-consolidating fiber reinforced concrete, possibly delivered in ready-mix trucks, with the potential of achieving a strain-hardening behavior in tension after cracking and a fiber content not exceeding 2% by volume. This will provide an enormous impetus for wide scale applications in earthquake, impact, and blast resistant reinforced and prestressed concrete structures. As of this writing, self-consolidating fiber reinforced concrete and fiber reinforced mortar mixtures have been developed and shown to exhibit strain-hardening behavior in tension. Since such mixtures can be used in ready-mix trucks they will become readily available to non-specialized practitioners.

In a nutshell—FRC progress

Over the past few decades, we have learned that fibers affect various properties of concrete at the fresh, hardening, and hardened state. The effect is rather negative (balling, segregation, harsh mixtures, etc.) in the fresh state and positive in the hardening (plastic shrinkage cracking) and hardened state. We have learned that bond between fiber and matrix is paramount for composite action and that the stiffness of the fiber material should be significantly larger than that of the matrix in order to improve hardened properties. Practically all mechanical properties, in the post-cracking state, improve with fiber reinforcement. At the material level, fracture toughness, energy absorption capacity, and fracture energy all improve as well. When added to reinforced and prestressed concrete, the benefits from fiber reinforcement can be predicted using various non-linear analysis models where the constitutive properties of the materials are considered. It is therefore expected to observe improvements in ductility, energy absorption capacity, impact and blast resistance, and resistance of the structure to various solicitations in bending, shear, tension, biaxial loading and the like. Average crack widths at service load are expected to be smaller. There is also clear evidence that numerous intangible benefits result from the use of fibers such as reduced spalling of cover under cyclic loading, reduced fragmentation under impact loading, improved bond of reinforcing bars and prestressing strands, improved durability, and reduced permeability and leakage.

A single sentence used by Parra-Montesinos sums up the intangible contributions of the fiber to structural concrete and that is “*fibers improve the damage tolerance of the structure.*”

In summarizing the path of fiber reinforced concrete over the past century the

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following observations can be made:

- The initial objective of most patents on fiber reinforced concrete was to improve the tensile strength of concrete, that is, by mixing in the reinforcement.
- The real benefit (initially un-recognized) was to improve toughness and fracture toughness, ductility, energy absorption capacity and impact resistance of the material.
- Another key benefit recognized much later (today) is the improved performance of conventional reinforced and prestressed concrete using matrices with fibers (bond, ductility, cyclic response, etc.), that is, leading to a highly “damage tolerant structure.”

At the practical experience level, we have learned that while concrete and fiber reinforced concrete seem very simple at first glance, tailoring and optimizing their properties for a high level of performance, such as high tensile strength and strain-hardening behavior, can be very complex and challenging.

Looking ahead: fiber-reinforced concretes

The previous paragraphs suggest already a number of ongoing developments including the increasing use of fibers in combination with reinforced and prestressed concrete structures, the increasing use of strain-hardening FRC composites, and the increasing availability of self-consolidating fiber reinforced concrete for delivery in ready-mix trucks. Recent ACI code adoption of fibers to replace minimum shear reinforcement is a first but daring step that will allow penetration of fiber reinforcement in many codes, standards and guidelines world-wide, and thus will have great impact on all applications. Simultaneously, at the educational level, either graduate courses on fiber reinforced cement composites will be increasingly offered, or the treatment of fiber reinforcement will find its way into various courses on concrete materials, brittle matrix composites, and reinforced concrete analysis and design.

In one statement summarizing the future impact of prestressed concrete, Freyssinet once said: “Prestressing will renew, from tip to toe, the present day conception of the ship, the port, and the port accessories.” Fiber reinforcement is about at the same stage. We can safely say today that fibers will renew from tip to toe the present day concept of structural concrete in all civil infrastructure applications.

To those who, like me, have worked on fiber reinforced cement composites some time during or over the past four decades, I would like to restate our secret dream which will keep challenging our minds and imagination, and those of our students, that is, *to mix fibers into concrete like sand or aggregate to achieve a moldable, strong, ductile and durable composite for construction applications (with similar strength and ductility as reinforced concrete or steel)*. Nothing less.

ACKNOWLEDGMENTS

However hard I tried, the list of acknowledgments would be too long and surely will not be complete. If I had to work on this enormous task, I would certainly include very

many colleagues not only in the U.S. but also in many countries abroad, friends, family, professional societies, funding agencies, students, visiting scholars, the universities which provided the exciting work environment for my academic career, institutions who hosted my visits during leaves and sabbaticals, and of course the organizers of this symposium and its proceedings.

DEDICATION

Often life is compared to a tree. Significant encounters lead to branches with their own path and fruit carrying ability. Eventually the tree stops growing taller but the branches blossom and the tree gains strength. With my students, I have had intersections in my life which led to new branches and new trees. These have changed my life and gave my work meaning, satisfaction and joy. In hindsight, I look back and wonder how lucky I was and I am humbly grateful.

I would like in particular to cite the following doctoral students whose path and mine crossed and converged to become each a venture, which led to most satisfying professional experience and friendship. These dissertations offer glimpses into their work.

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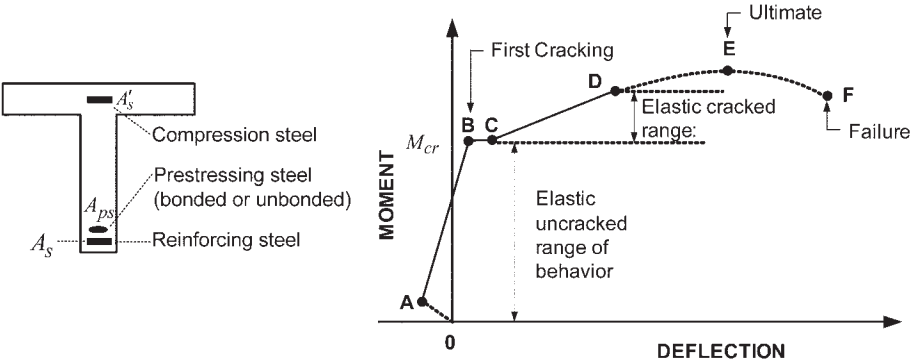


Figure 1—Partially prestressed beams: Analysis in the elastic uncracked, elastic cracked and ultimate range of behavior—work completed in the late 1970s.

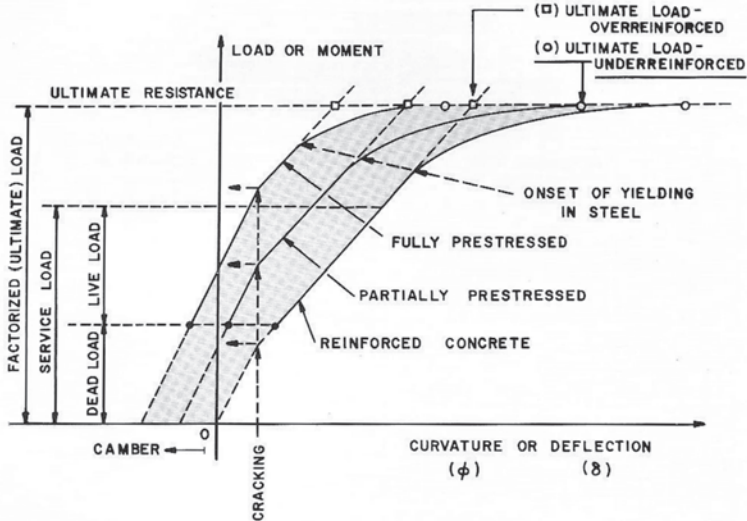


Figure 2—Schematic moment curvature of reinforced, prestressed and partially prestressed beams for same ultimate resistance—work completed in the late 1970s.

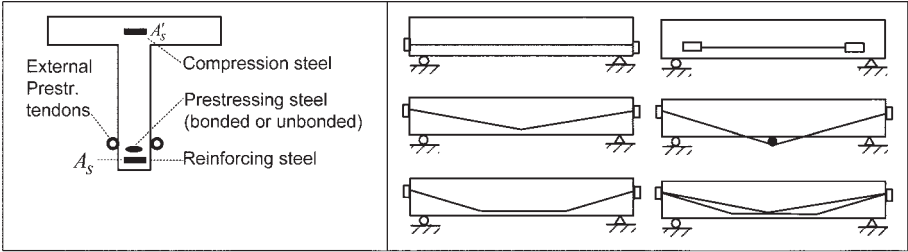


Figure 3—Typical external prestressing applied to reinforced, prestressed or partially prestressed beams with bonded or unbonded tendons—generalized case for unified solution studied in the 1980s.

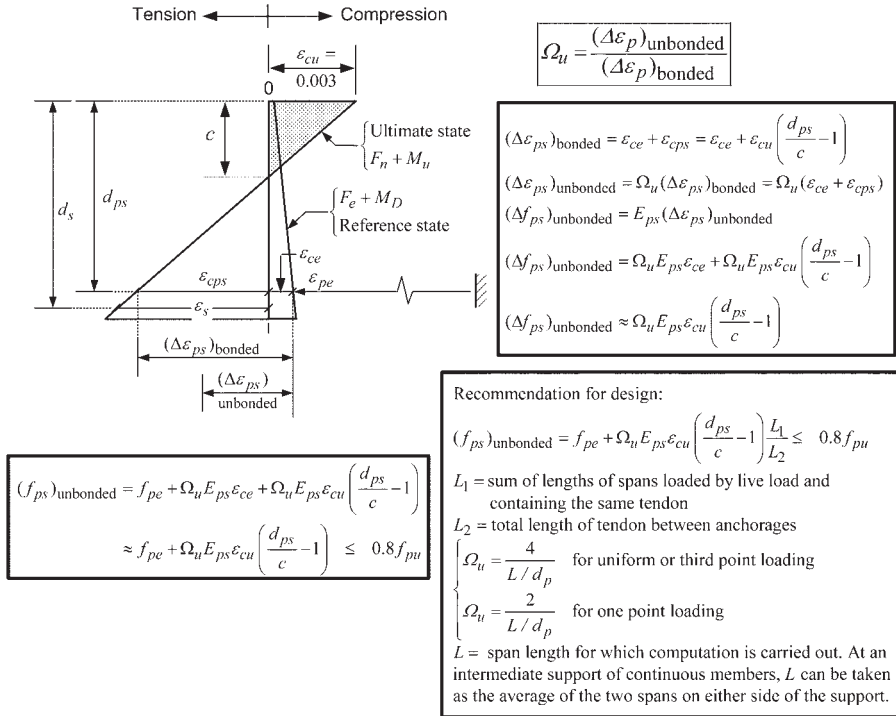


Figure 4—Summary solution to predict the stress in unbonded tendons at ultimate with recommended value of strain reduction coefficient, Ω_u for use in design—late 1980s to date.

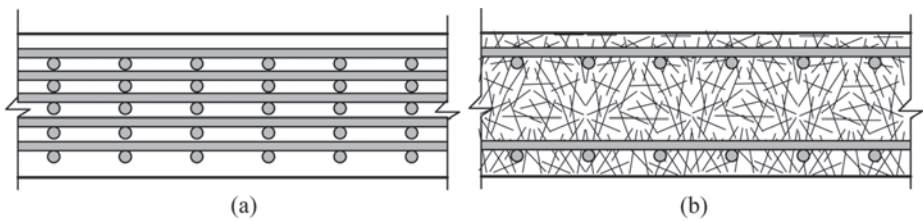


Figure 5—(a) Typical section of ferrocement with several layers of mesh; and (b) Typical section of efficient hybrid composite with only two extreme layers of mesh and fibers.

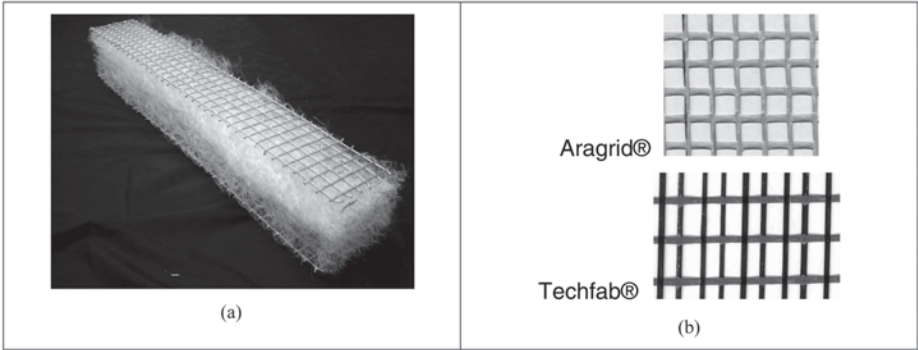


Figure 6—(a) Typical ferrocement sandwich construction where a fiber mat acts as core and spacer of the two extreme layers of mesh reinforcement; and (b) typical aramid (10 x 10 mm [0.394 x 0.394 in.] opening) and carbon (25 x 13 mm [0.984 x 0.512 in.] opening) meshes made with high-performance fibers embedded in a polymeric matrix.

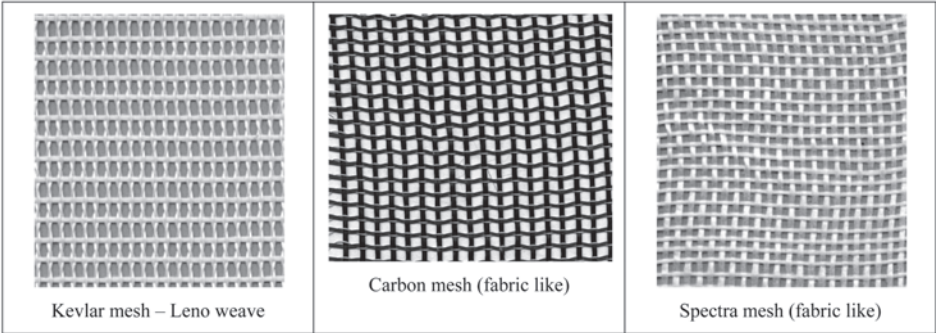


Figure 7—Typical fiber reinforced polymeric meshes (or textiles of fabrics) suitable for use in ferrocement and thin cementitious composites (mesh openings about 3 to 5 mm [0.12 x 0.20 in.]).

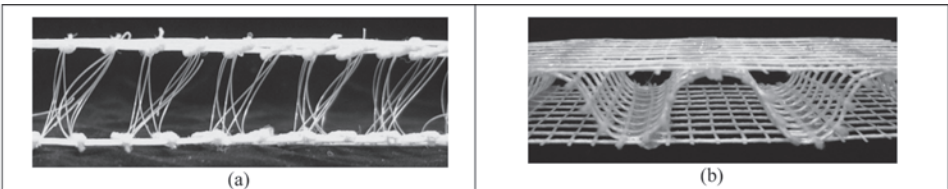


Figure 8—Examples of 3D mesh systems with FRP (textile) reinforcements fabricated at the ITA in Aachen, Germany: (a) 3D spacer-sandwich textile; and (b) 3D spacer stiff textile (about 15 mm [0.59 in.] in thickness).

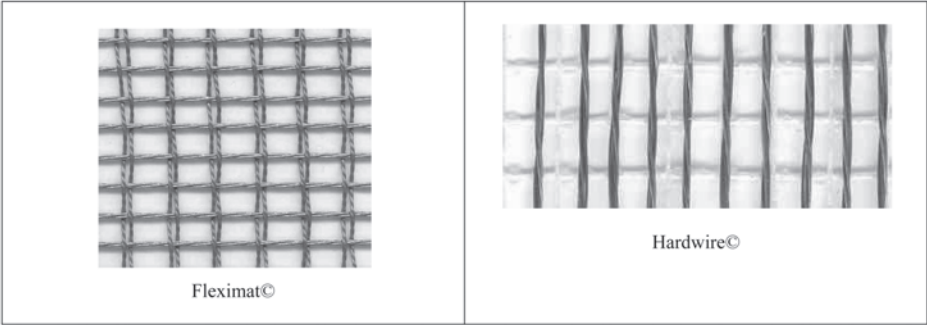


Figure 9—Unidirectional high strength steel meshes using strands with tensile strength exceeding 2400 MPa (350 ksi) for use in ferrocement type applications (strand spacing about 3 mm [0.12 in.]).

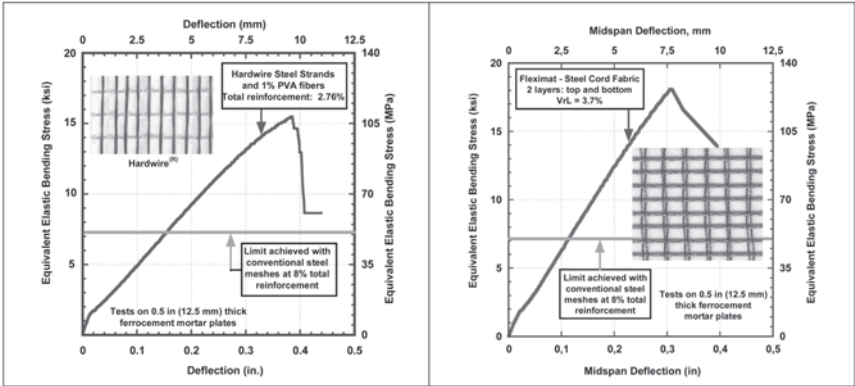


Figure 10—Illustration of bending response of ferrocement plates using high strength steel reinforcement (from Fig. 9) and comparison with limit achieved using conventional steel wire meshes

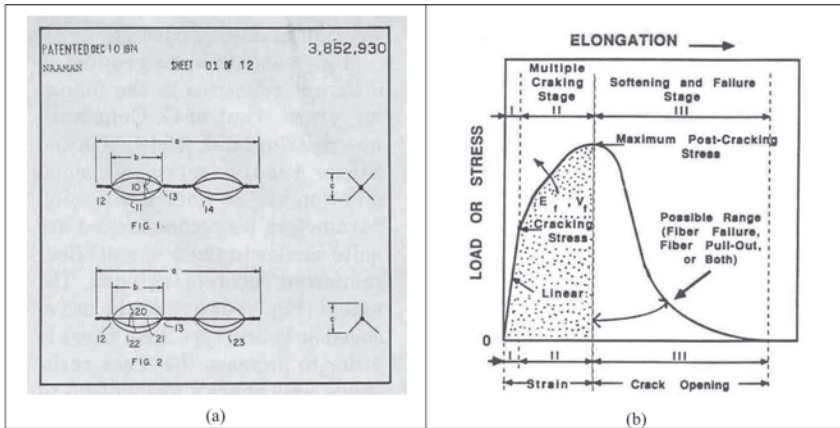


Figure 11—(a) 3D steel fiber (1974 US patent) designed to deform (the wires tend to confine the matrix inside the ball structure) during pull-out in order to improve pull-out resistance and ductility; and (b) Tensile response of high performance fiber reinforced cement composite (HPFRCC) as first defined in 1987.

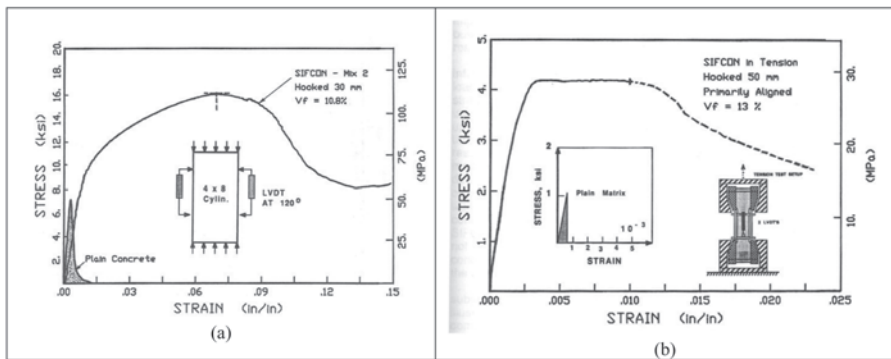


Figure 12—Examples of stress-strain properties of SIFCON as compared to plain concrete in: (a) compression; and (b) tension (research carried out during the 1980s)

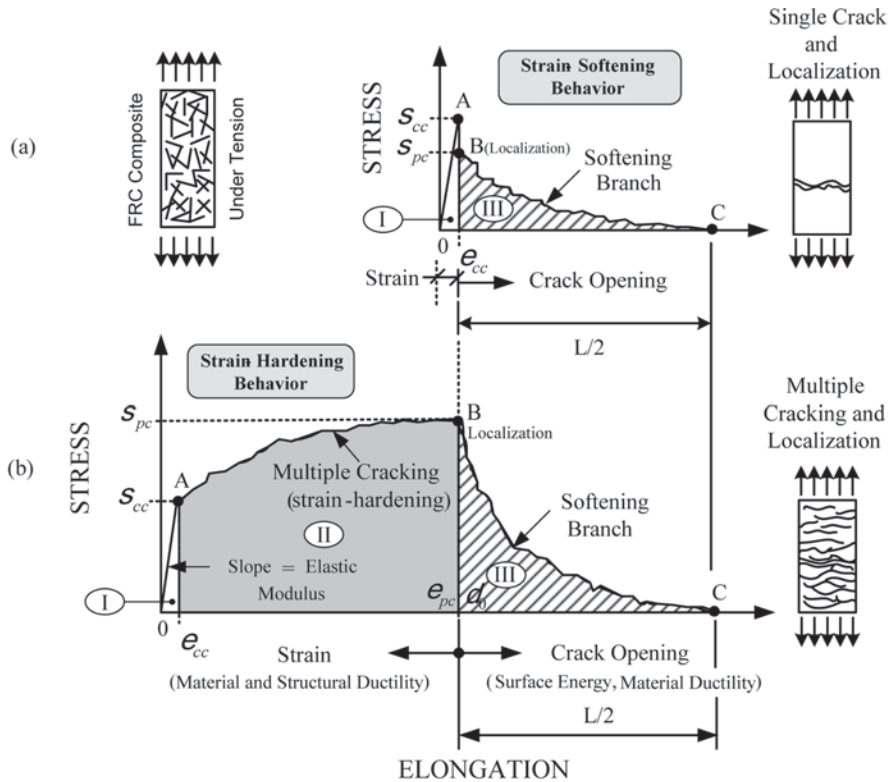


Figure 13—Characterization and classification of tensile response of fiber reinforced cement composites: (a) strain-softening; and (b) strain-hardening (also termed HPFRCC)

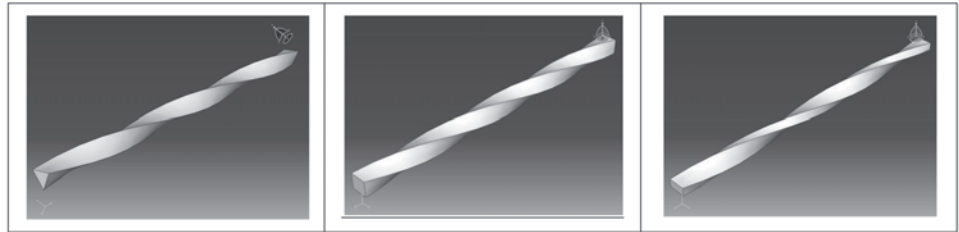


Figure 14—Twisted triangular, square, and rectangular Torex fibers (1999 U.S. patent) designed to untwist during pull-out in order to improve composite ductility while maintaining high tensile stresses in the fiber

Innovative Technologies for Prestressed Concrete Bridges in Japan

by H. Mutsuyoshi and H. Minh

Synopsis: Prestressed (PS) concrete technology is being used all over the world in the construction of a wide range of structures, particularly bridges. However, many PS bridges have been deteriorating even before the end of their design service-life due to corrosion and other environmental effects. In view of this, a number of innovative technologies have been developed in Japan to increase not only the structural performance of PS bridges, but also their long-term durability. These include the development of novel structural systems and the advancement in construction materials. This paper presents an overview of such innovative technologies on PS bridges including a brief discussion of their development and applications in actual construction projects. Some noteworthy structures, which represent the state-of-the-art technologies in the construction of PS bridges in Japan, are also presented.

Keywords: external tendons; extradosed prestressed concrete bridges; grout; high-strength concrete; prestressed concrete bridges, prestressing tendon.

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INTRODUCTION

In Japan, the application of prestressed (PS) concrete was first introduced in the 1950s. Since then, the construction of PS bridges has grown dramatically. Figure 1(a) shows the total number and length of concrete and steel bridges longer than 15 m (590.5 in.) in Japan, as of 1997. The increased interest in the construction of PS bridges can be attributed to the fact that the initial and life-cycle costs of PS bridges, including repair and maintenance, are much lower than those of steel bridges. Moreover, PS bridges are more economically competitive and aesthetically superior due to the employment of high-strength materials compared to reinforced concrete (RC) bridges.

In recent years, many PS bridges have been deteriorating even before the end of their design service-life due to corrosion and other environmental effects. Figure 1(b) shows the number of bridges with age over 50 years until year 2021. As a consequence, long-term durability has become a particular concern and is considered seriously in the design and construction of PS bridges, apart from the viewpoint of structural safety, aesthetics, and cost. In Japan, a number of innovative technologies have been developed to increase not only the structural performance but also the long-term durability of PS bridges. These include the development of novel structural systems (external prestressing, highly eccentric external tendons, and extradosed prestressing) and the advancement in construction materials (transparent sheath, pre-grouted prestressing tendon, ultra-high-strength concrete, and corrugated steel web).

This paper presents an overview of such innovative technologies in PS bridges, including a brief discussion of their development and application in actual construction projects. In addition, some noteworthy structures in Japan, which represent the state-of-the-art technologies in PS bridges, are presented.

DEVELOPMENT OF NOVEL STRUCTURAL SYSTEMS IN PS BRIDGES

External prestressing is widely being used in the construction industry.

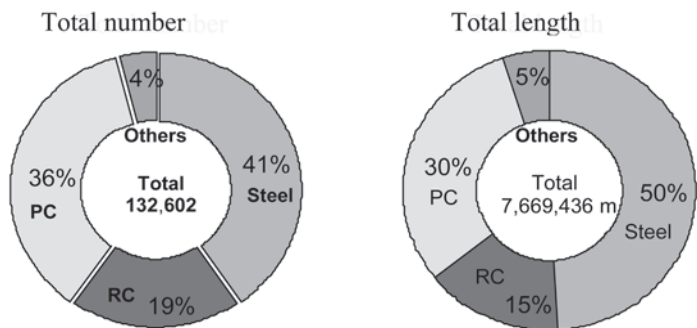


Fig. 1(a)—Total highway bridges with length >15 m as of 1997 (1 m = 39.4 in.).

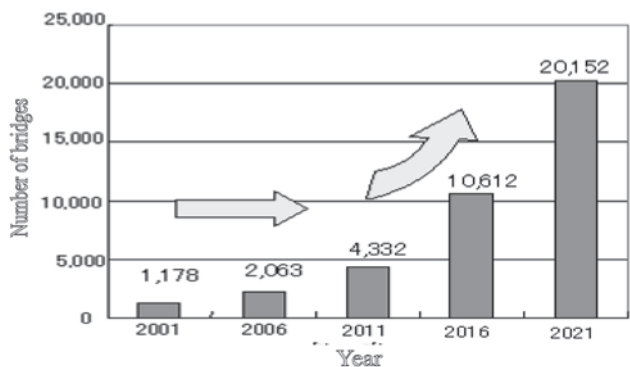


Fig. 1(b)—Number of bridges over 50 years (PWRI 2001).

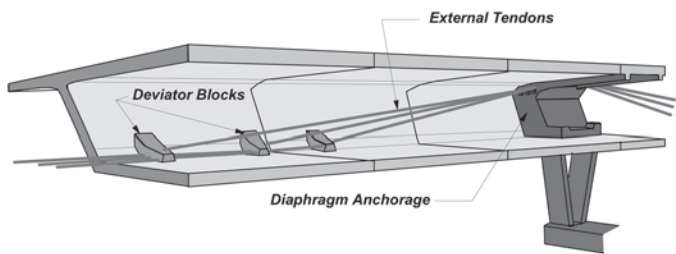


Fig. 2—Typical layout of external PS box girder bridge.

Externally, PS bridges are designed with prestressing tendons placed outside the concrete section, but still remaining within the bounds of the girder cross section (Fig. 2). The concept of external prestressing has become increasingly popular in the construction of medium- and long-span bridges due to several advantages such as reduced web thickness, possibility to control and adjust tendon forces, and ease of inspection of tendons during construction. The Japan Highway Public Corporation (JH; now changed to three highway companies), which governs most of the highway structures in Japan, has actively adopted the concept of fully external tendons for box girder bridges (Fig. 2) since 1999

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due to the improved durability compared to that of internally grouted tendons. It is important to note that, recently, construction of new PS bridges with internally grouted tendons has been forbidden by JH due to the bad quality of grouting of internal tendons in some existing PS bridges (Mutsuyoshi 2001). Consequently, various new technologies have recently been developed in Japan to improve the performance of externally prestressed concrete bridges.

PS bridges with highly eccentric external tendons

Externally, PS bridges are well recognized to have several advantages; however, they have lower flexural strength compared to that of bridges with internally bonded tendons (Virlogeux 1988). This is due to the smaller tendon eccentricity, which is limited by the bounds of the girder concrete section (that is, at the bottom slab in case of box-girder bridges), as well as the reduction in tendon eccentricity at the ultimate flexural failure (so-called second-order effect). One possible method of enhancing the flexural strength of externally PS structures is making the tendons with high eccentricity (Fig. 3). This type of construction is possible when external prestressing is used since this allows the tendons to be placed outside the concrete section. In this concept, the compressive forces are taken by concrete and the tensile forces by external tendons, thus taking advantages of both materials effectively (Mutsuyoshi, 2000).

There has been extensive research conducted at Saitama University both analytically and experimentally to study the fundamental behavior of girders with highly eccentric external tendons (Aravinthan et al. 1999; Aravinthan et al. 2001; Witchukreangkrai et al. 2000). Figure 4 shows details of test specimens and the layout of highly eccentric external tendons (Types A, B, and C). Test variables included tendon profile, casting method (monolithic and segmental), loading arrangement (symmetrical and unsymmetrical), and type of loading (one-point and two-point loading). The tendon profiles in Types A and B were obtained by linear transformation of that of specimen Type C (Fig. 5). The loading setup for a single-span beam with highly eccentric external tendon is illustrated in Fig. 6. Test results have shown that, in continuous beams with linearly transformed tendon profiles, the overall flexural behavior is unaffected by the tendon layout in both linear elastic and ultimate limit state (Fig. 7(a)). This gives considerable flexibility in the design of tendon layouts for beams with large eccentricity, allowing tendon eccentricity to be extended below the beam at the midspan or above the beam at the support, depending on the clearance available at a particular site. In addition, the method by which such beams are cast, either monolithic or precast, has little influence on overall flexural behavior (Fig. 7(b)). Hence, this gives considerable flexibility in selecting the method of construction when designing bridges with highly eccentric external tendons.

The relationship between stress increase in the external tendons with mid-span deflection is shown in Fig. 8. It can be seen that the tendon stress increased in a nearly linear manner up to yielding. The rates of tendon stress increase in continuous beams with symmetrical loading and single-span beams were nearly the same and much higher than that of continuous beams with unsymmetrical

loading. Hence, it can be concluded that the stress increase in highly eccentric external tendons is greatly influenced by the loading arrangement in each span. One of the concerns raised for this type of structure was the shear capacity because the girder height is considerably reduced. It was verified, however, from the experiment on shear characteristic of model specimens that the shear capacity of the girders with highly eccentric external tendons is much higher than that of the conventional girders due to the large increase of tensile force in external tendons.

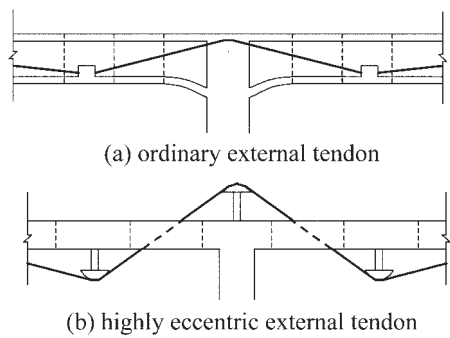


Fig. 3—Ordinary versus high eccentric external tendons.

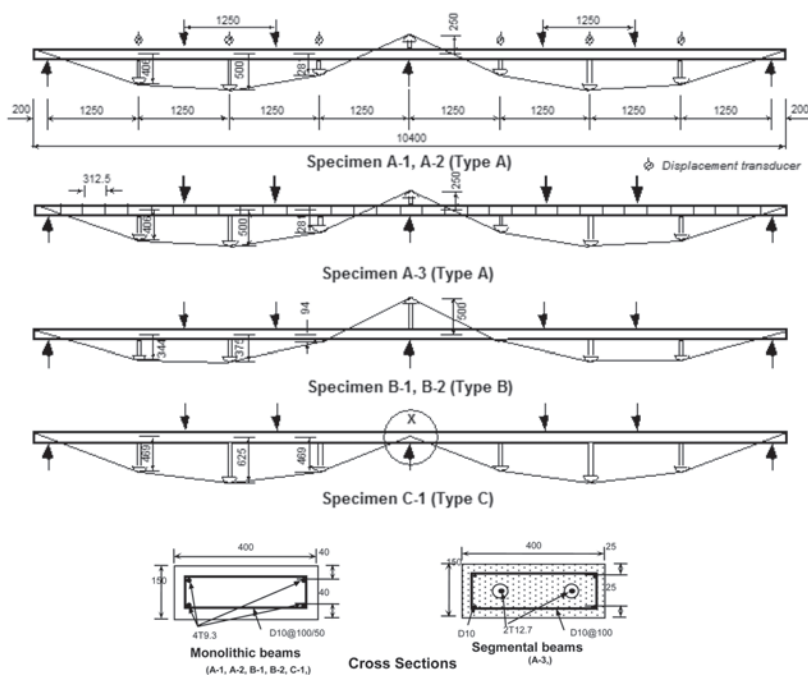


Fig. 4—Details of test specimens and layout of highly eccentric external tendons (1 mm = 0.04 in.)

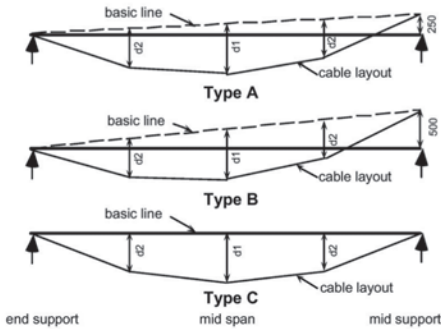


Fig. 5—Linear transformation tendon layout.

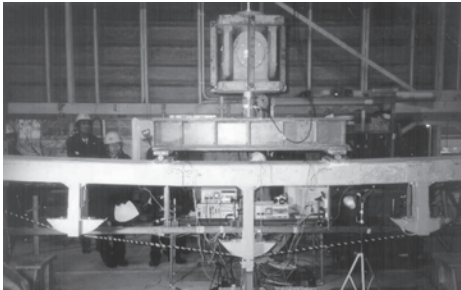
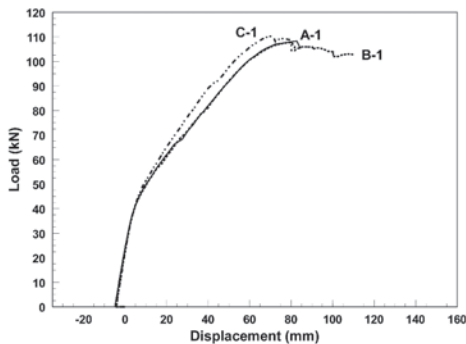
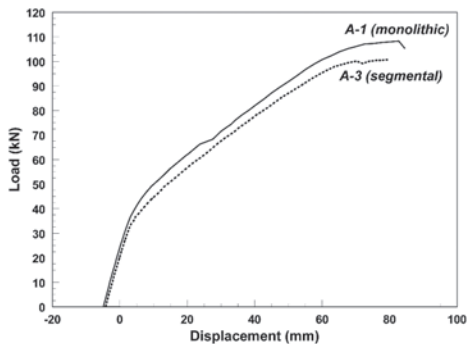


Fig. 6—Loading test of single span girder of with highly eccentric external tendons.



a) Continuous beams (symmetrical loading)



b) Continuous beams (monolithic & segmental)

Fig. 7—Load-displacement characteristics (1 mm = 0.04 in.).

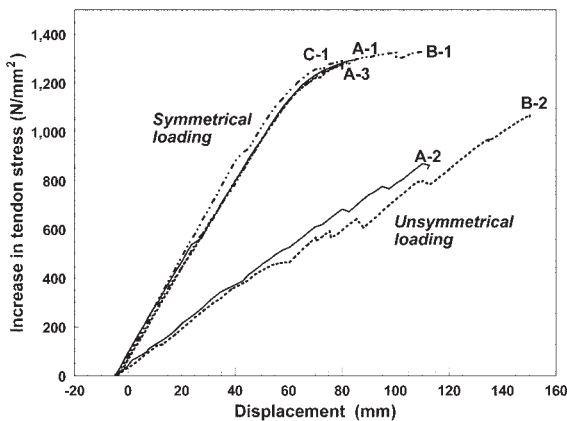


Fig. 8—Stress increase in external tendons with midspan deflection (1 mm = 0.04 in.).

The world's first application of prestressing with highly eccentric external tendons to a continuous-span girder was in the construction of the Boukei Bridge in Hokkaido, Japan. Considering the site conditions, the bridge was designed with a two-span continuous and unsymmetrical girder having a total length of 57 m (6181.1 in.) (Fig. 9). The effective width of the bridge varies from 3.0 m (118.1 in.) at the abutments to 6.0 m (236.2 in.) at the central pier. A view of the completed bridge is shown in Fig. 10.

The characteristic of this innovative bridge is that the external tendon layout takes a shape similar to the bending moment diagram. The structure was designed to form a pseudo truss, with the main girder made of concrete as compression chords, the external tendons as tension chords, and the steel deviators as diagonal members. This allowed the girder height to be reduced significantly, making the bridge lightweight. The external tendons are placed below the girder in the midspan region by means of steel struts, the function of which is similar to a truss. At the intermediate support region, the external tendons are placed above the bridge deck and covered with a fin-shaped concrete web member. The combination of the subtended tendons and the fin-shaped concrete web makes this bridge a unique one with aesthetically pleasing appearance.

Although having several advantages, PS bridges with highly eccentric external tendons should be carefully designed and constructed paying particular attention to following important points. Since the main girder, struts and highly eccentric external tendons form a truss in this type of structure, construction precision of individual members has a significant influence on the structure. For this, it is necessary to give special consideration to the techniques and procedures for constructing the falsework, formwork, and external tendons. Moreover, the vibration characteristics under service load may be of concern due to the smaller stiffness of the main girder caused by the reduction in girder height. Nevertheless, the authors believe that this new concept of prestressing would pave the way to a wider use of external prestressing technology in the construction industry, leading to improved structural performance as well as cost effective PS bridges. Research is in progress regarding the possibility of applying these kinds of structures to highway bridges using precast segmental construction.

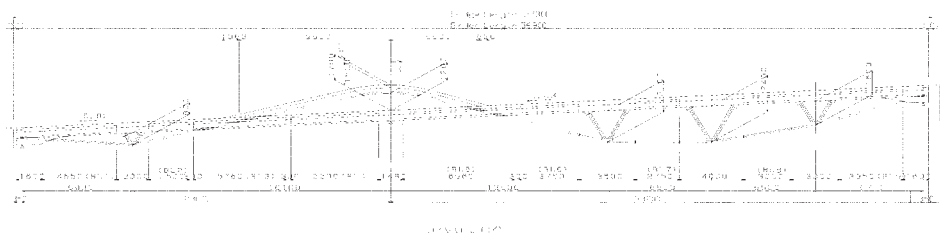


Fig. 9—Layout diagram and dimension of Boukei Bridge (1 mm = 0.04 in.).



Fig. 10—View of completed Boukei Bridge.

Extradosed PS bridges

An extradosed prestressing concept, which was first proposed by Mathivat (Mathivat 1988) in France, is a new type of structural system in which the tendons are installed outside and above the main girder and deviated by short towers located at supports. Considering its definition, this type of bridge is placed between cable-stayed bridges and ordinary girder bridges with internal or external tendons. Extradosed PS bridges have several positive characteristics. The girder height may be lower than that of ordinary girder bridges, thus reducing self-weight of the structure. As shown in Fig. 11, the ratio of the girder height to the span length (H/L) in extradosed bridges ranges between 1/15 and 1/35, while it is approximately 1/15~1/17 for box-girder bridges. The height of the main tower of extradosed bridges is lower than that of cable-stayed bridges (Fig. 11); hence, a reduction in construction labor costs of can be achieved.

Because of a lower main tower in extradosed bridges, vertical loads are partly resisted by main girders and stress variations in stay cables produced by live loads are smaller than those in cable-stayed bridges. This is quite similar to the behavior of box-girder bridges, where the main girder itself has a decisive influence on the structure rigidity and live loads produce only limited stress variations in tendons. Based on these facts, the Japan Road Association (1993) has recommended that the safety factor for the stayed cables in extradosed bridges under design loads shall be taken as 1.67 ($0.6 f_{pu} : f_{pu}$ = tensile strength of tendons), which is the same as that for tendons in ordinary girder bridges. For cable-stayed bridges, this value is specified to be 2.5 ($0.4 f_{pu}$).

The major difference among box-girder, extradosed and cable-stayed bridges can be further revealed by comparing the relationship between materials used with span lengths. In box-girder bridges, as shown in Fig. 12, the average concrete depth increases with the span length, since the girder height is a function of the span length. On the other hand, in cable-stayed bridges, there is almost no increase in the average depth of concrete because the girder height is generally designed to be 2.0~2.5 m (78.7~98.4 in.), regardless of the span length. It is

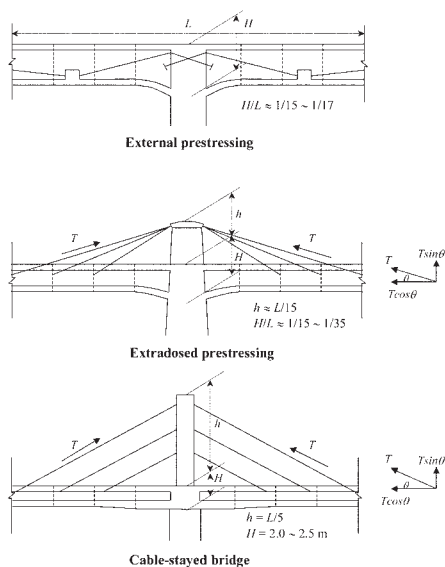


Fig. 11—Comparison among externally box-girder, extradosed, and cable-stayed PS bridges (1 m = 39.4 in.).

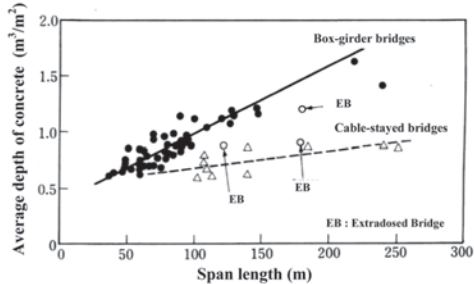


Fig. 12—Relationship between average concrete depth and central span length (1 m = 39.4 in.).

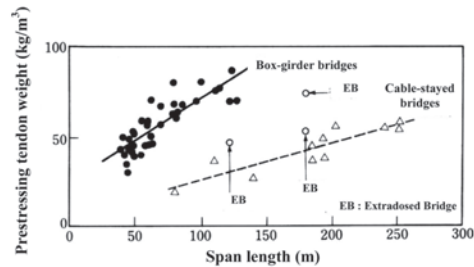


Fig. 13—Relationship between quantity of steel cable and central span length (1 m = 39.4 in.).



Fig. 14—Odawara Blue Way Bridge (three-span continuous extradosed PS bridge).

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interesting to note that extradosed bridges are placed between these two types. Similarly, with increasing span length, the quantity of prestressing tendons in box-girder bridges shows higher increase than that in cable-stayed bridges, whereas extradosed bridges show intermediate values between the other two types, as shown in Fig. 13.

Nowadays, a great number of PS bridges using extradosed prestressing are being constructed in Japan. Attempts are also being made to apply this structural concept to other innovative technologies, such as corrugated steel web, precast segmental construction, and composite structures with steel girders. Figure 14 shows the Odawara Blue-Way Bridge, which was the first extradosed PS box girder bridge in the world and was completed in 1994. This bridge was designed with a three-span continuous box-girder with extradosed prestressing, having a middle span length of 122 m (4803.1 in.), a tower height (h) of 10.5 m (413.4 in.), and a girder height at supports (H) of 3.5 m (137.8 in.).

Corrugated steel web bridges

In PS bridges with corrugated steel webs, lightweight corrugated steel plates are used instead of concrete webs. The corrugated steel plate webs are capable of withstanding shear forces without absorbing unwanted axial stresses due to prestressing, thus enabling efficient prestressing of top and bottom concrete deck slabs, resulting in an “accordion effect” (Fig. 15). Moreover, the corrugated webs also provide high shear buckling resistance. Use of lightweight corrugated steel plates for webs causes a reduction of self weight of about 25% in main girders. Therefore, this enables the use of longer spans and reduction of construction cost. The weight of a segment to be cantilevered during erection can also be reduced, thus longer erection segments can be adopted and construction period can be shortened. This also eliminates assembly of reinforcement, cable arrangement and concrete placement for concrete webs. Thus, saving of construction manpower, quality enhancement and improvement of durability are expected. In addition, replacing damaged deck slabs is easier than that in ordinary PS bridges.

Recently, the use of corrugated steel webs has been applied to a variety of new PS bridges in Japan (Fig. 16). In addition to the rigid or box girder bridges, the concept of corrugated steel web was also successfully adopted in the construction of extradosed and cable-stayed PS bridges.

DEVELOPMENT OF INNOVATIVE MATERIALS FOR PS BRIDGES

Several research projects have been carried out recently for the development of new construction materials to enhance the performance and long-term durability of PS bridges. In this paper, application of transparent sheath, pre-grouted internal tendons, and ultra-high-strength concrete (UHSC) are explained with a brief overview on their applications in actual PS bridge projects.

Pre-grouted prestressing tendon

Pre-grouted prestressing tendon was first developed in 1987. It is made by

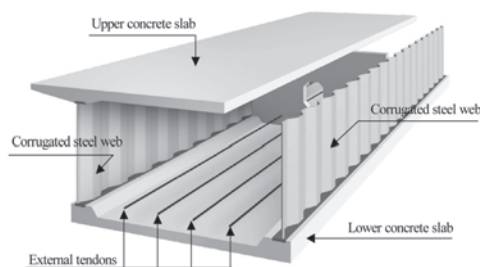


Fig. 15—Typical section of PS bridge with corrugated web.



Fig. 16—Hontani Bridge with corrugated web.

coating prestressing strands with an unhardened epoxy resin and a polyethylene protective tube (Fig. 17), and is embedded directly into concrete with the polyethylene protective tube as a tendon for post-tensioning. Time of hardening is set for the epoxy resin filled in the polyethylene protective sheath so that post-tensioning process can be completed before hardening of the epoxy resin. The resin has viscosity like grease before hardening, and it naturally hardens with time after the completion of tensioning of prestressing steel and bonds with concrete to form an integrated object. As the grout is injected into the polyethylene sheath, complete grouting is ensured in this technique. Furthermore, construction work can be saved as neither in-situ insertion of prestressing tendons nor grouting process is required. Sheath and epoxy resin also provide double layer corrosion protection to the prestressing tendons. This technique also ensures stronger bond with concrete than conventional cement grouting technique. Moreover, smaller diameter of sheath makes concrete placement relatively easier and higher efficiency in prestressing can be achieved as polyethylene sheath and unhardened resin reduce the friction during prestressing.

Pre-grouted prestressing steel was generally applied only to transverse prestressing of deck slabs or other work. Application as main steel has just started in view of the above benefits. When pre-grouted prestressing strands are used for main tendons, more prestressing strands are required than when conventional multiple strands are used. Numerous prestressing strands, therefore, should be anchored in limited anchorage spacing. Special attention should, thus, be given to design for the arrangement of pre-grouted prestressing tendons and detailing of anchorage. Management issues include the identification of the time of hardening of resin and of the coefficient of friction because the resin hardens at various times according to the temperature during construction and the maximum temperature of placed concrete (Fig. 18 and 19).

UHSC in PS bridges

One of the recent developments in PS technology is the use of UHSC. The chief advantage of using UHSC is the possibility of achieving higher prestressing force

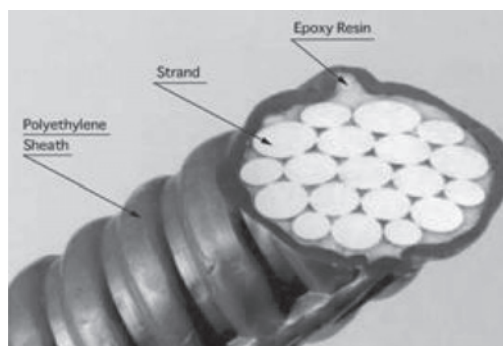


Fig. 17—Pre-grouted prestressing tendons.

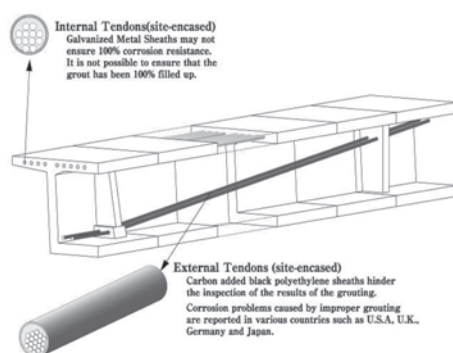


Fig. 18—Conventional prestressing tendon systems,

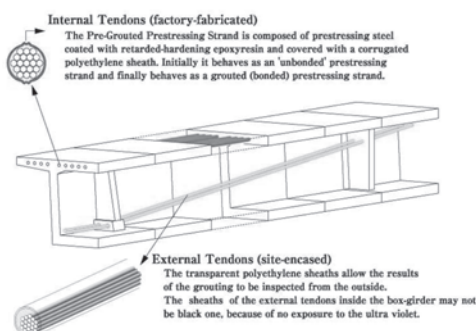


Fig. 19—Innovative prestressing tendons system.

compared to regular strength concrete, which could lead to smaller cross section and reduction in the overall weight of the structure. Hence, the use of UHSC has a good potential in the construction of large structures.

The lower water-binder ratio in UHSC may, however, cause the increase in autogenous shrinkage, which will lead to a decrease in effective prestressing force and cracking due to the restraining caused by the reinforcing steel. A conventional method of reducing autogenous shrinkage is to use expansion-producing admixture and shrinkage-reducing agents. However, these materials are expensive. This problem has been overcome by the development of a new type of artificial lightweight aggregate called as “J-Lite” (Fig. 20). J-lite is made from environment-friendly coal ash and is twice as strong as conventional lightweight aggregate. Low-shrinkage UHSC termed as “Power Crete,” with a compressive strength as high as 120 N/mm² (17,400 psi), has been developed with the use of J-lite together with expansion-producing admixtures and shrinkage-reducing agents. As strength development in low-shrinkage UHSC is independent of the curing condition, it can be used for cast-in-place applications as well.

The two-span continuous girder pedestrian bridge in Akihabara, Tokyo, was

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constructed using low-shrinkage UHSC with design compressive strength of 120 N/mm² (8700 psi). The total span of the bridge is 170 m (6692.9 in.) and the effective width is 8.0 m (315 in.). External prestressing technique was used in this bridge and struts were installed to increase the rigidity. The webs were fabricated in plant and transported to the site whereas the slab, external cable deviators and cross beams on piers were cast in place. A view of the completed bridge is shown in Fig. 21. Figure 22 shows the comparison of girder section using high-strength concrete and ordinary concrete. It is shown that using high strength concrete, the height of the girder was reduced 63% as compared to that when using ordinary concrete. Therefore, using high-strength concrete is a very effective method to reduce the girder's height and weight.

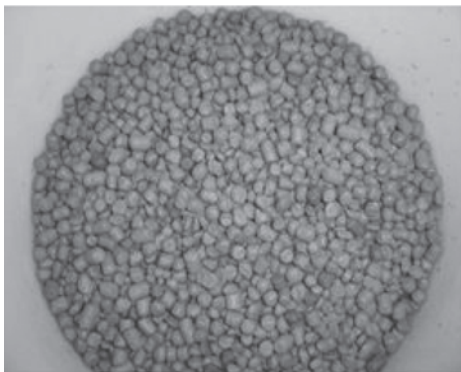


Fig. 20—J-lite.



Fig. 21—Pedestrian bridge in Akihabara constructed with UHSC.

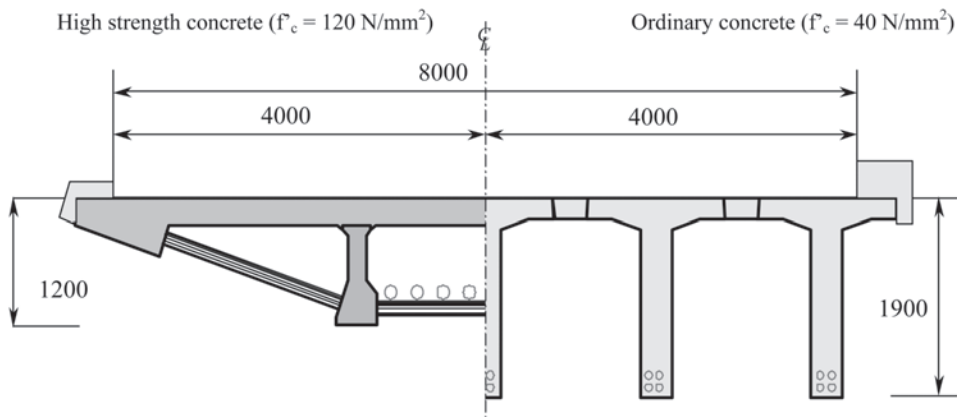


Fig. 22—Comparison of girder section using high strength concrete and ordinary concrete (1 mm = 0.04 in.).

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APPLICATION OF INNOVATIVE TECHNOLOGIES IN PS BRIDGES

The state-of-the-art technologies described in this paper with regard to structural systems and construction materials have already been implemented in PS bridges in Japan. Some of the noteworthy structures, which represent the state-of-the-art technologies in the construction of PS bridges in Japan, are presented here.

Kiso and Ibi River Bridges

The Kiso River and the Ibi River Bridges were constructed as part of the New Meishin Expressway that will connect Nagoya and Kobe. The 1145 m (1252.1 yd) long Kiso River Bridge (Fig. 23) and the 1379 m (1508 yd) long Ibi River Bridge (Fig. 24) are located approximately 1300 m (1421.7 yd) apart, crossing the two major rivers. The total width of both bridges is 33 m (36.1 yd), accommodating six traffic lanes. The center and the side spans are over 270 m (295.3 yd) and 150 m (164 yd), respectively.



Fig. 23—Kiso River Bridge.



Fig. 24—Ibi River Bridge.

An extradosed PS bridge type and steel composite girders were selected to cope with such long spans, taking economy, construction time, and workability into consideration. This is the most unique feature of these bridges, making them the world's first extradosed bridges with a composite structure. Steel girders were used in the central sections of approximately 100 m (109.4 yd) to reduce the weight of the superstructure, while high-strength concrete girders were used for the remaining sections. Further reduction of the dead load was achieved by placing some of the tendons externally inside the box girder section, which made thin webs possible.

A precast segment method was employed for the concrete sections of the girders. Each segment with weight of 300 to 400 ton (272 to 362 tonne) was precast using high strength concrete of 60 N/mm² by the short line match casting method in the fabrication yard located about 10 to 15 km (6.2 to 9.3 mi) away from the project sites. The segments for the main girders were transported to the sites by sea and placed into position by large temporary facilities, such as erection noses and erection trusses, except for the pier head tables, each of which was divided into three and installed by a 600 ton (544 tonne) floating

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crane. The steel girder sections, which weigh about 2000 ton (1814 tonne) each, are being fabricated in a factory and will be transported to the sites and then erected at one time by reaction girders attached to both ends of the concrete girders after completion of the concrete segments installation.

The construction of the Kiso and Ibi River Bridges has demonstrated that the concept of extradosed prestressing can be successfully applied to long-span bridges having composite structures. Based on the creation of standards and experience up to now, the extradosed PS bridges, streamlined in terms of both structural properties and economic considerations, will undoubtedly continue to develop in the future.

Shibakawa Viaduct

The Shibakawa Viaduct was constructed under the New Tomei Expressway project about 160 km (99.4 mi) west of Tokyo. The structure was designed as two separate bridges consisting of the inbound and outbound lines using continuous PS box-girder bridges with struts constructed by the cantilever method (Fig. 25).

The main characteristic of the Shibakawa Viaduct is the use of inclined struts with box-girder bridges (Fig. 26). In the Shibakawa Viaduct, which has piers of over 80 m (3149.6 in.) in height, the construction costs of piers and foundations were relatively large compared to the total construction costs. By adopting the inclined struts to support the cantilever slabs, the dead load of the superstructure was reduced to approximately 80% and the volume of concrete that was used for the bridge piers and foundations was reduced to approximately 50%, thus leading to a significant reduction in the total construction costs.



Fig. 25—Shibakawa Viaduct (continuous PS box girder with inclined struts).



Fig. 26—PS box girder with inclined struts.

Shiosai Bridge

The Shiosai Bridge in Shizuoka, Japan, is a prestressed concrete four-span stress-ribbon bridge with roadway slab decks, which forms a so-called inverted suspension bridge as shown in Fig. 27 (Mochizuki et al 1996). The bridge, for cyclists and pedestrians, was completed in 1995. The superstructure consists of

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roadway slab decks, columns which support the slab decks, and stress ribbons forming the lower cord. The bridge length is 232 m (9133.9 in.), the clear width is 3.0 m (118.1 in.), and the span lengths are 55 m (2165.4 in.), 61 m (2401.6 in.), 61 m (2401.6 in.), and 55 m (2165.4 in.). In this structure, loads are transferred via the slab decks into the columns, which are supported by the stress ribbons that in turn are supported by the piers and abutments. Overall stiffness was increased using elastic connections with horizontal neoprene bearings, which were installed between the ends of the roadway slabs and the abutments. Most of the superstructures were precast using lightweight concrete, in order to minimize the horizontal forces acting on the abutments, thus making the structure more economical. To ensure corrosion protection, epoxycoated prestressing strands were used for bearing cables and prestressing cables for the stress ribbon.



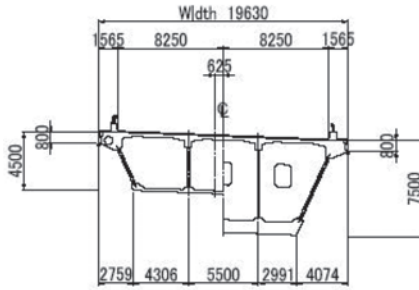
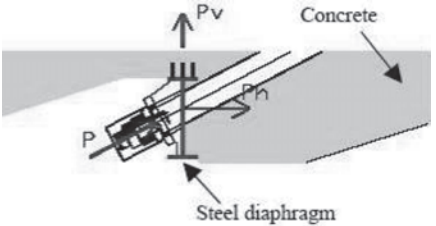
Fig. 27—Shiosai Bridge (four-span continuous stress-ribbon PS bridge).

Rittoh Bridge

The Rittoh Bridge over the southern edge of Lake Biwa, about midway between Otsu Junction and Shigarai Interchange on the New Meishin Expressway, was constructed utilizing several innovative construction technologies. It is the first extradosed PS bridge with corrugated steel web whose main girder has a three-celled cross section (Fig. 28), making it suitable for a bilaterally suspended structure with a wide roadway. Figure 29 shows closer view of the corrugated steel web of the Rittoh Bridge.

The bridge consists of four-span and five-span continuous rigid-frame structures with total span length of 495 and 555 m (541.3 and 607 yd), respectively (Fig. 30). Due to construction restrictions, the erection of supports at the side span portions was impossible. Hence, an innovative technology was used for the construction of side span closures by first installing a corrugated steel web as integrated steel girders, and then applying the load of the main deck concrete to the corrugated steel web girder. This construction method made it possible to construct side spans without the need for large-scale erection equipment, resulting in significant cost savings.

The most significant aspect of this bridge is the use of a composite structure for the cable anchorage. In this bridge, steel diaphragm structure is used for the extradosed cable anchorage at the main girder instead of the conventional concrete diaphragm, which significantly reduced the dead weight. The extradosed cables are directly attached to the steel diaphragm. The horizontal component of the force from the extradosed cables is transmitted to the concrete behind the steel diaphragm, while the vertical component of the force is transmitted directly to the steel diaphragm as shown in Fig. 31.

	
Fig. 28—Rittoh Bridge (extradosed PS bridge with corrugated steel web).	Fig. 29—Closer view of corrugated steel web of Rittoh Bridge.
	
Fig. 30—Cross section of main girder (1 mm = 0.04 in.).	Fig. 31—Transmission of stress from extradosed cable.

CONCLUSIONS

Recent techniques in design and construction of PS bridges in Japan were presented in this paper, with emphasis on their background and development as well as their applications in actual structures. Not only were these technologies developed to improve structural properties in terms of safety, aesthetic, and economical aspects, but also to enhance long-term durability, which is becoming one of the most serious problems in concrete structures nowadays.

The innovated technologies presented herein can be mainly divided into two

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categories: novel structural systems and new construction materials. In light of new structural systems, external prestressing with highly eccentric tendons and extradosed prestressing are excellent examples of a wider use of external prestressing technology to achieve a PS bridge with improved structural performance as well as cost-effective outlook.

Considering the development of new construction materials, applications of transparent sheaths, pre-grouted internal tendons and use of low-shrinkage UHSC were discussed. The application of innovative construction materials can significantly enhance the long-term durability of externally PS bridges. Corrugated steel webs, which take advantages of steel and concrete, have proved to be a promising solution to reduce the self-weight of main girders, thereby enabling the use of longer spans and a reduction of construction cost. Pre-grouted prestressing tendons, which are made of prestressing strands coated with unhardened epoxy resin and polyethylene tube and embedded directly into concrete can provide better protection to and enhance the performance of prestressing tendons. On the other hand, a new lightweight aggregate, referred to as “J-lite”, can be used to produce low-shrinkage UHSC, which can achieve higher prestressing force compared to normal concrete.

Furthermore, several research projects are in progress to develop fiber composites as prestressing materials, which are more durable compared to conventional prestressing tendons. However, implementation of fiber composites in prestressing is limited due to shortcomings such as relaxation. These issues need further investigations. Moreover, use of external post-tensioning for shear strengthening of reinforced concrete beams is also being investigated.

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Direct Method for Strengthening Continuous RC Beams with External Tendons

by K.H. Tan and D. Kong

Synopsis: This paper presents a simple direct method to determine the external tendon configuration required for a desired increase in load-carrying capacity of continuous beams. The tendon layout is selected based on the concept of equivalent loads, but need not be concordant. By considering the collapse mechanism of the beam, the increase in load-carrying capacity can be related directly to the tendon force. It is shown that the increase in load-carrying capacity is partly due to an increase in the force in the compression zone arising from the horizontal component of the prestressing force, and partly due to the upward components of the prestressing force. The method was verified with a test program on six two-span continuous beams, in which the tendon profile and loading pattern were varied. Comparison of the test results and those available in the literature showed that the proposed method gives a reasonably conservative design. A simplified method based on the direct balancing of increased loads is also proposed.

Keywords: collapse mechanism; continuous beam; equivalent loads; external tendons; load-carrying capacity

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INTRODUCTION

Continuous reinforced concrete beams are commonly found in buildings and bridges. Due to a change in usage or upgrading of design codes, there may be a need for structural strengthening. External post-tensioning, in which the tendons are installed on the outside of members using anchorages and deviators, is an efficient method due to the easy installation of tendons and hence minimal interruption in the regular functioning of the structure. As the external tendons are placed outside of concrete sections, they are free to move between the end anchorages. The tendon stress at ultimate state is more or less uniform along the tendon and depends on the overall deformation of the strengthened beam. As a result, the analysis and design of beams with external tendons is different from those with internal bonded tendons, and is member dependent.

For practical design, Naaman and Alkhalairi (1991) have proposed the use of bond reduction coefficients to account for the difference in strain increase in unbonded tendons and concrete, so that such beams can be analyzed based on section analysis. More recently, the authors (Tan and Kong 2009) have proposed a direct method to determine the tendon area required for a specified increase in load-carrying capacity of simple-span beams. The method is based on the concept of equivalent loads and moments and found to provide a conservative design. In this paper, the method is explored for use in continuous beams, which are commonly found in buildings and bridges.

RESEARCH SIGNIFICANCE

This paper presents a direct method by which the required tendon configuration in terms of profile, area, and eccentricities to provide an increase in load-carrying capacity of continuous beams can be determined. The method was found to provide a conservative design for strengthening beams using external tendons. Based on the formulation, a simplified method independent of the material properties of the beam is also proposed to predict the increase in load-carrying capacity.

DIRECT METHOD FOR STRENGTHENING DESIGN

In the proposed method, the tendon profile is first selected based on the concept of equivalent loads. The tendon need not be concordant, but it would

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provide a moment equal to and opposite to that due to the applied load, which may include the moment redistribution from the interior support section. The tendon area is then determined from the equations that are established by considering the collapse mechanism of the beam assuming plastic hinges formed at section of maximum positive moment in the span and at the interior support section.

Tendon profile

Consider a two-span continuous beam subjected to symmetrical two-point loads on each span, as shown in Fig. 1. The tendon can be selected such as to balance the applied load based on the concept of equivalent loads. That is, the tendon is draped at points directly under the point loads and over the interior support. Thus, equating the moment diagram due to the prestressing force F_{ps} (assumed constant throughout the length of tendon) to that due to the applied loads, one obtains

$$F_{ps}(e_{m1} - \alpha e_s) = F_{ps}[e_{m2} - (1 - \alpha)e_s] = \frac{\alpha \Delta P L}{2} \quad (1)$$

where F_{ps} is the tendon force, e_{m1} , e_{m2} , and e_s are the tendon eccentricities at the loading points and interior support, respectively, measured positive (+) downward from the centroidal axis of the beam, α is a factor less than 1, ΔP is the increase in applied load, and L is the span of the beam.

From Eq. (1), the eccentricities e_{m1} and e_{m2} can be determined for given values of ΔP , F_{ps} , and e_s . It should be noted that the value of e_s should be selected such as not to compromise the serviceability of beams in terms of cracking at the interior support. Also, Eq. (1) can be apply to individual spans of a multiple-span beam, for which e_s should be replaced by $(e_{s2} - e_{s1})$, where e_{s1} and e_{s2} are the tendon eccentricities at the left and right supports, respectively.

For beams under uniform distributed loads, the required tendon profile can be similarly obtained, that is, the tendon has a parabolic shape within each span and is draped over the interior supports.

Tendon area

The required tendon area is obtained from equations established by considering the collapse mechanism of the beam as follows. For the two-span continuous beam shown in Fig. 1, the collapse mechanism comprises the formation of plastic hinges at the section of maximum positive moment under the outer point load and interior support section as shown in Fig. 2(a).

Figure 2(b) shows the stress diagram for the critical sections, in which the subscript i refers to the section of maximum positive moment or the interior support section. Assuming that internal steel bars yield at ultimate limit state and ignoring the presence of compressive reinforcement, if any, the moment capacity of the critical section of the unstrengthened beam is given by

$$M_{n0i} = T_{0i} (d_{si} - \frac{a_{0i}}{2}) \quad (2)$$

where T_{0i} and d_{si} are the force in and effective depth of the tensile reinforcement and a_{0i} is the height of equivalent rectangular stress block, given by

$$a_{0i} = \frac{T_{0i}}{0.85bf'_c} = \frac{A_{si}f_y}{0.85bf'_c} \quad (3)$$

in which A_{si} and f_y are the area and yield strength of internal reinforcement, b is the beam width, and f'_c is the concrete cylinder compressive strength.

After the beam has been strengthened using external tendons, the flexural capacities of the critical sections will increase from M_{n0i} to M_{nsi} ; again, assume yielding of the internal reinforcement at ultimate limit state and no friction between the tendons and deviators, the value of M_{nsi} , referring to Fig. 2(c), can be expressed as

$$M_{nsi} = T_{0i} (d_{si} - \frac{a_i}{2}) + F_{ps} (d_{pi} - \frac{a_i}{2}) = T_{0i} (d_{si} - \frac{a_{0i}}{2}) + F_{ps} (d_{pi} - \frac{a_{0i} + a_i}{2}) \quad (4)$$

where F_{ps} is the tendon force, d_{pi} is the effective depth of tendon at the critical sections and a_i is the height of equivalent rectangular stress block in compression zone given by

$$a_i = \frac{T_{0i} + F_{ps}}{0.85bf'_c} = \frac{A_{si}f_y + A_{ps}f_{ps}}{0.85bf'_c} \quad (5)$$

in which A_{ps} is the external tendon area and f_{ps} is the tendon stress.

In view of Eq. (2) and (4), the increase in moment capacity of each critical section is

$$\Delta M_{ni} = M_{nsi} - M_{n0i} = F_{ps} (d_{pi} - \frac{a_{0i} + a_i}{2}) \quad (6)$$

Introducing the ratio of depths of rectangular compression stress blocks as

$$K_i = \frac{a_i}{a_{0i}} = \frac{c_i}{c_{0i}} \quad (7)$$

where c_{0i} and c_i are the depth of neutral axis before and after strengthening, respectively, Eq. (6) can be written as

$$\Delta M_{ni} = F_{ps} [d_{pi} - \frac{a_{0i}}{2} (1 + K_i)] \quad (8)$$

An upper limit can be imposed on the value of K_i by requiring the strengthened section to remain tension controlled. For this to occur, ACI code specifies that the net tensile strain in the extreme tension steel shall be equal to or greater

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than 0.005, which requires $\frac{c_i}{d_{si}} \leq 0.375$. Substituting $ci = 0.375d_{si}$ into Eq. (7) gives an upper limit of K_i

$$K_i = \frac{0.375d_{si}}{c_{0i}} \quad (9)$$

On the other hand, substituting Eq. (3) and (5) into (7) gives the value of K_i as

$$K_i = \frac{A_{si}f_y + A_{ps}f_{ps}}{A_{si}f_y} = 1 + \frac{A_{ps}f_{ps}}{A_{si}f_y} = 1 + \frac{\rho_{pi}f_{ps}d_{pi}}{\rho_{si}f_yd_{si}} \quad (10)$$

where ρ_{pi} is the prestressing steel ratio ($= A_{ps}/bd_{pi}$) and ρ_{si} is the internal reinforcement ratio ($= A_{st}/bd_{si}$). As $f_{ps} \leq f_{py}$, where f_{py} is the yield strength of prestressing steel (which corresponds to the stress at one percent strain for wires and strands, and 0.7% strain for bars [ASTM A421, A416, and A722]), another limiting value of K_i is obtained by substituting $f_{ps} = f_{py}$ into Eq. (10), giving

$$K_i = 1 + \chi_i \frac{d_{pi}}{d_{si}} \quad (11)$$

where χ_i is termed the relative prestressing index, defined by

$$\chi_i = \frac{\rho_{pi}f_{py}}{\rho_{si}f_y} \quad (12)$$

STRENGTH ENHANCEMENT BASED ON COLLAPSE MECHANISM

Once the increase in moment capacity of the critical sections has been determined from Eq. (8) to (11), the increase in load-carrying capacity of the beam, ΔP_n , can be calculated based on the collapse mechanism of the beam. For the two-span continuous beam carrying two point loads at αL from the supports on each span, as shown in Fig. 2(a), the flexural capacities for the section of maximum positive moment and interior support section before strengthening are denoted by M_{n01} and M_{n02} , respectively. Assuming tension-controlled sections thereby ensuring sufficient rotation capacity, the load-carrying capacity of the beam (for one span) before strengthening can be determined as

$$P_{n0} = \frac{2}{\alpha L} (M_{n01} + \alpha M_{n02}) \quad (13)$$

After strengthening, the flexural capacities of the strengthened beam at section of maximum positive moment and interior support are M_{ns1} and M_{ns2} , respectively. The load-carrying capacity of the strengthened beam (for one span), based on the same mechanism, is given by

$$P_{n0} + \Delta P_n = \frac{2}{\alpha L} (M_{ns1} + \alpha M_{ns2}) \quad (14)$$

where ΔP_n is the increase in load-carrying capacity.

In view of Eq. (13) and (14), the increase in load-carrying capacity of the beam is given by

$$\Delta P_n = \frac{2}{\alpha L} (\Delta M_{n1} + \alpha \Delta M_{n2}) \quad (15)$$

Substituting the values of ΔM_{n1} and ΔM_{n2} from Eq. (8) into (15) gives

$$\Delta P_n = \frac{2}{\alpha L} F_{ps} \left\{ [d_{p1} - \frac{a_{01}}{2} (1 + K_1)] + \alpha [d_{p2} - \frac{a_{02}}{2} (1 + K_2)] \right\} \quad (16)$$

Introducing the effective tendon depth at critical sections as

$$d_{p1} = y_t + e_{m1} \quad (17)$$

$$d_{p2} = y_b - e_s \quad (18)$$

where y_t and y_b are the distances from the centroid of the section to the top and bottom faces of the beam, respectively, Eq. (16) can be written as

$$\Delta P_n = 2F_{ps} \frac{(e_{m1} - \alpha e_s)}{\alpha L} + 2F_{ps} \sum_{i=1}^2 \left\{ \frac{\alpha_i [y_i - \frac{a_{0i}}{2} (1 + K_i)]}{\alpha L} \right\} \quad (19)$$

in which $\alpha_1 = 1$, $y_1 = y_t$ for the critical intra-span section, and $\alpha_2 = \alpha$, $y_2 = y_b$ for interior support section.

Equation (19) includes two terms: the first term being a direct contribution to the load-carrying capacity due to the vertical components of the prestressing force at deviator points. The second term is due to an increased concrete compression zone at the critical sections, which is needed to balance the horizontal components of the tendon forces. Equation (19) is termed the “Refined Equation” in this study.

Omitting the second term in Eq. (19) leads to

$$\Delta P_n = 2F_{ps} \frac{(e_{m1} - \alpha e_s)}{\alpha L} \quad (20)$$

which gives a conservative estimate of the increase in load-carrying capacity of the beam. This equation is independent of the material property of the beam and is termed the “Simplified Equation” in this study.

Tendon stress at ultimate flexural limit state

Equations (19) and (20) can be used to determine the required tendon area A_{ps} for a desired increase in load-carrying capacity provided that the tendon stress f_{ps} at ultimate is known. Four equations for the value of f_{ps} are selected for