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THÈME 4

Innovation in the Field of Concrete Structures

Innovations dans le domaine des structures en béton

Neuerungen auf dem Gebiet von Betonbauwerken

Challenge of Innovation in Materials for Structural Concrete

Défi des nouveaux matériaux dans les structures en béton

Herausforderung von neuen Materialien in Betonbauwerken

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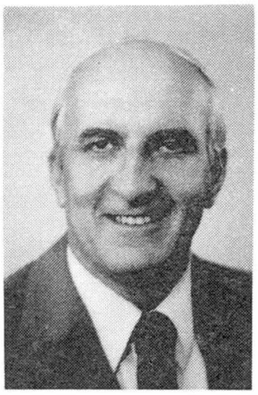

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SUMMARY
This introductory statement defines and illustrates the nature of innovation in structural concrete materials. The challenges due to the rapid innovations in this field are shown to be more related to overcoming institutional barriers than to spurring scientific development. Examples are cited to illustrate that the dramatic changes taking place in the space travel and electronic computer fields have direct parallels in structural concrete. Recommendations are made to help both individuals and the profession in adjusting to these rapid changes.

RÉSUMÉ
Cet exposé définit et illustre les possibilités d'innovation dans le domaine des matériaux pour les structures en béton. Ces innovations rapides font apparaître les problèmes à surmonter les obstacles institutionnels plutôt que d'encourager le développement scientifique. Des exemples présentés montrent que les progrès spectaculaires réalisés dans les domaines des vols spatiaux et de l'électronique ont leurs équivalents dans le domaine de la construction en béton. Des recommandations sont faites pour permettre aux individus et à la profession de s'adapter à ces changements rapides.

ZUSAMMMENFASSUNG
Dieser einführende Bericht erläutert die Möglichkeiten von neuen Baumaterialien auf dem Gebiet des Betonbaus. Es zeigt sich, dass die Schwierigkeiten eher mit institutionnellen Hindernissen verbunden sind als mit der Förderung von wissenschaftlichen Entwicklungen. Anhand von Beispielen wird aufgezeigt, dass den spektakulären Fortschritten auf den Gebieten der Raumschiffahrt und der Elektronik auch ähnliche Entwicklungen auf dem Gebiet des Betonbaus gegenüberstehen. Es werden Empfehlungen gegeben, die dem Einzelnen und dem gesamten Bauwesen erlauben sollen, sich diesen raschen Entwicklungen anzupassen.

1. INTRODUCTION

1.1 Innovation

It is a great pleasure and honor to share in the opening of this session: Innovation in the field of materials. In the papers which follow, we will learn details of the incredible variety of new developments in the various materials and processes incorporated in structural concrete. In contrast, I will highlight the basic nature of innovation and describe some of the challenges which recent innovations are posing to the vast structural concrete industry.

A typical dictionary [1] defines innovation as "that which is newly introduced: a change." This gives little insight into the process of innovation. A much more useful distinction has been made by Strassman [2], who suggested that "The word invention may therefore be used as the contrivance of a new device with certain technical features, and the word innovation as all activities of a business enterprise in developing a product and production method to the point at which it gives reliable service and allows sales at a price greater than the cost." I prefer this latter definition because it not only addresses the technological or scientific domain, but clearly indicates the need for serviceability, reliability, and economy.

Toffler [3] suggests that techological innovation consists of three stages which are interactively linked to a self-reinforcing cycle:

First, there is the creative, feasible idea
Second, there is its practical application
Third, there is its diffusion through society

The first of these stages is that of invention while the second and third stages are what distinguishes an innovation from an invention.

1.2 Rate of Innovation

We live in an age of almost frighteningly fast change. In Science and the Crisis in Society, George [4] reminds us ". . .that in the next thirty years we shall achieve, scientifically, more than in the last million years." Evidence of this accelerated development abounds in our daily lives. Many flew to this Symposium from far-flung continents, traveling to Paris in a matter of hours. Fig. 1 shows the development of the speed of human travel. The speed records of the camel caravans of antiquity (13 km/hr) gave way to chariots (30 km/hr) [3]. The speed of human travel remained basically unchanged until the development of the steam locomotives, followed by the automobile. In this century, man learned to fly, to conquer the sound barrier, to ride the rockets, and to orbit in space at speeds over 30,000 km/hr. The rate of innovation during the last half of this century is difficult to comprehend.

While few of us were shot in rockets to Paris, the more affluent could ride the Concorde at supersonic speed across the ever-shrinking oceans. This suggests that the almost asymptotic speed curve has practical limits for mass travel slightly below the speed of sound. These limits are both economic (design, manufacturing, and operation costs for supersonic aircraft tend to be very high) and environmental (jealous neighbors will not permit the shock caused by supersonic flight over their land masses).

A similar but much more recent phenomenon can be seen in the development of the electronic circuits that have made the computer feasible. Fig. 2 shows the dramatic change in the density of electronic circuits as the mechanical switches and relays of the nineteenth century gave way first to the vacuum tubes and then rapidly to the transistors of the 50's, the integrated circuits of the 60's, and today's third generation very-large scale integrated circuits. We can see that in this century the density of electronic switches has evolved through a number of orders of magnitude, resulting in almost zero length paths for electrons to travel.

1.3 Participants in Innovation

If we examine the role players in the development of electronic circuits from the perspective of the three stages of an innovation suggested by Toffler, we can draw an interesting analogy for innovations in structural concrete materials.

The first stage is the creation of a feasible idea. In electronic circuits this involves physicists and electrical engineers. In concrete materials development we see the roles of physicists, metallurgists, chemists, and some materials engineers.

The second stage is the practical application of the idea. None of us buy electronic circuits as such. We want a personal computer, a compact disc player, or some other convenience of the digital revolution. The practical utilization of electronic circuits involves product development by some physicists but many more electrical engineers, mechanical engineers, and computer scientists, who transform the basic circuits into a machine with a useful function and a reasonable cost. In concrete materials development, we see the roles of the materials engineer, the industrial and chemical engineers, the sales and distribution specialists who must transform the "test-tube" product into a viable field product.

The third stage is the diffusion through society. The ways of utilizing the electronic circuitry are brought about by a combination of computer programmers, systems analysts, and engineers. They know very little of the basic manufacture of silicon wafers, but they do know how to utilize the resultant hardware and software combinations to make life easier, faster, more efficient, or more entertaining. Without the knowledge, skill, and imagination of these programmers who are able to use the basic computer for a myriad of tasks, the products would never be sold. A direct parallel in concrete materials technology are the roles of the structural engineer and the constructor. The structural engineer must make the technical decision to use a new material in some structural application. The constructor must correctly place the new material in service. Thus, innovation requires communication between multiple disciplines. Any of the participants can destroy the effectiveness of the new development if they do not clearly understand its potential and its limitations. The consequences of a structural concrete system failure can be catastrophic as shown by the Ronan Point collapse.

1.4 Time Lags in Innovation

Careful study [3,5] has shown that while the lead time between invention and first practical application has shortened somewhat, it is still usually measured in decades. However, what has accelerated appreciably is the diffusion process, the time between introduction of a new product or process into the market and its general adoption [5]. For example, a study of the diffusion rate of electrical appliances introduced in the first half of this century as compared to those introduced in the second half indicated that the lag time between first commercial introduction and peak production had shrunk by more than 76% from 34 years to only 8 years. In our structural concrete industry one sees similar phenomena. In a comparatively short time the high range water reducing admixtures (superplasticizers) went from a demonstration novelty to a major market force. Our traditional codes and standards regulating structural concrete are often revised using volunteers who work slowly but steadily with schedules which often span a decade. These were acceptable periods when innovations required decades for diffusion, but are far too long in today's world.

2. IMPORTANCE AND ACCEPTANCE OF INNOVATION

2.1 Attitudes Towards Innovation

In Future Shock, Toffler tells us that change is the process by which the future invades our lives [3]. Like all invasions, change threatens those who have become comfortable with the status quo. Managers often fear and oppose change. It disrupts continuity and forces them to new decisions. It destroys reliance on past experience and increases risk of error and failure. Generally, firms dealing with an established product are not inclined to introduce an innovation unless the prospective profits are large enough to not only cover the development and marketing costs of the new product but also to write off the capital costs invested in the manufacture of the old product [6]. In a specific purpose material like concrete, it is difficult for the new material sciences to dramatically change the fundamental nature of the material. Many of the basic applications of concrete are as bulky surfaces--i.e. pavements, floors, roofs, and walls. The exceedingly low unit cost of the present material in bulk form makes it very difficult for new materials to supplant it. Concrete has generally fine performance characteristics within its normal range of application. Basic innovations tend to target specialty product areas where these ranges can be profitably extended. High strength compression materials for building columns and prefabricated sections, fiber concretes for strengthening complex joints, polymer concretes for toughening surfaces subject to abrasion and chemical attack such as bridge decks and chemical tanks, are all examples of important specialty innovations. The use of chemical retarders to improve placement in hot climates, air entrainment to improve freezing and thawing resistance, and pumping as a placement tool are examples of more general innovations.

In society in general and in our little world of structural concrete, change is here. In Figs. 1 and 2, the accelerating nature of a few typical changes in society were traced. An almost direct parallel can be seen in the development of the two basic ingredients of structural concrete. Fig. 3 indicates the general development of metals over the last six centuries. The coppers and bronzes of the Middle Ages slowly gave way to the cast irons and low grade steels. The lowly reinforcing of Lambot and Monier gave way to the more sophisticated reinforcing bars and prestressing steels of today. Again, almost an order of magnitude of improvement in strength has taken place in our century. The improvement in concrete strength shown in Fig. 4 is even more dramatic. The 10 to 15 MPa concretes one reads about in the pioneering research works of Ritter and Morsch have become the 100 MPa concrete columns widely used in Chicago and the 140 MPa concrete columns now under construction in three buildings in Seattle [7].

Many in the industry are completely unaware that these changes have occurred. We tend to be blinded by the everyday surroundings in which we find ourselves. We become what Drucker calls "prisoners of the familiar" [5]. Designers become used to certain material ranges which their experience indicates work well. Specifications are repeated successfully from job to job. Mix designs work well. Concrete is made with familiar ingredients from the same sources. Regardless of real cause, any difficulty on a jobsite is always blamed on any

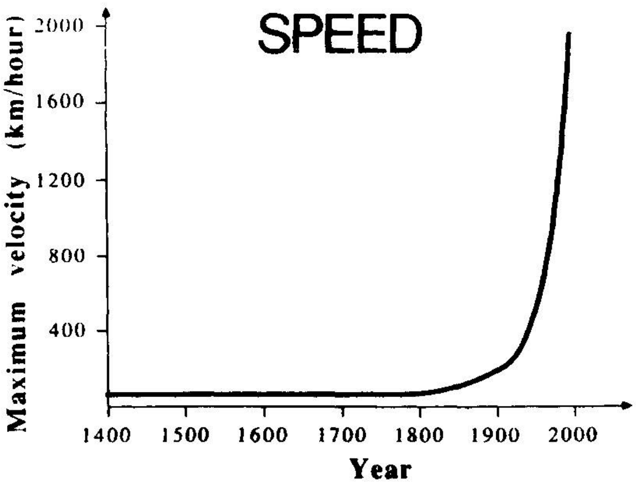

Fig. 1 Development of speed of human travel

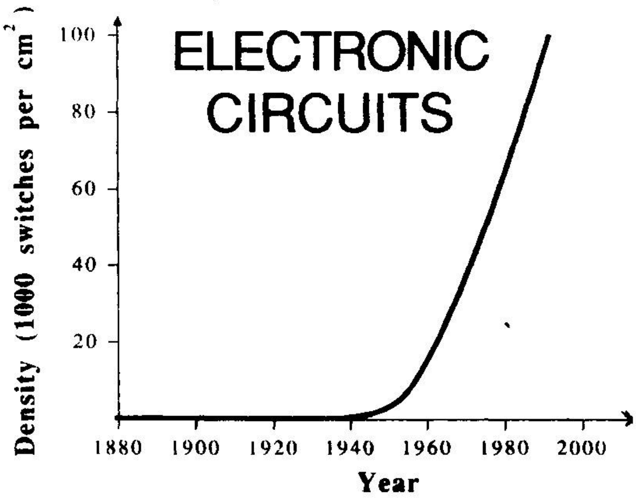

Fig. 2 Development of density of electronic circuits

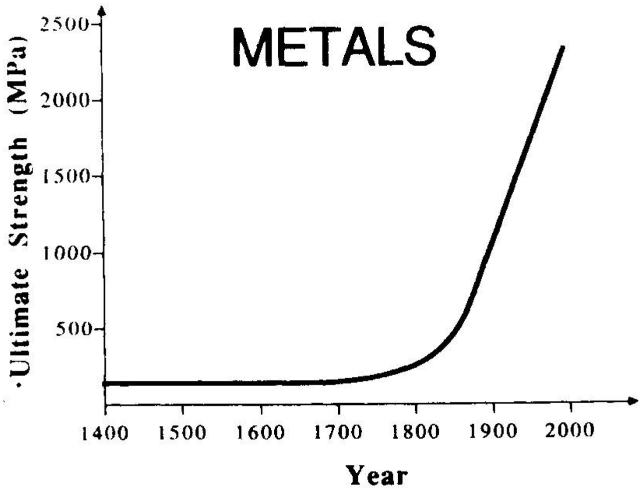

Fig. 3 Development of metal tensile strength

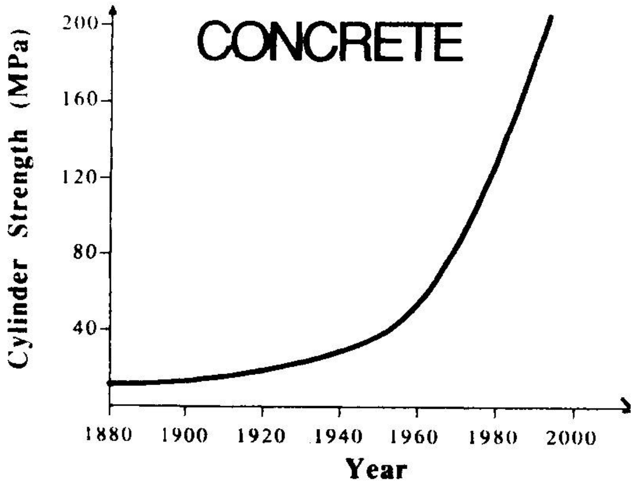

Fig. 4 Development of concrete compressive strength

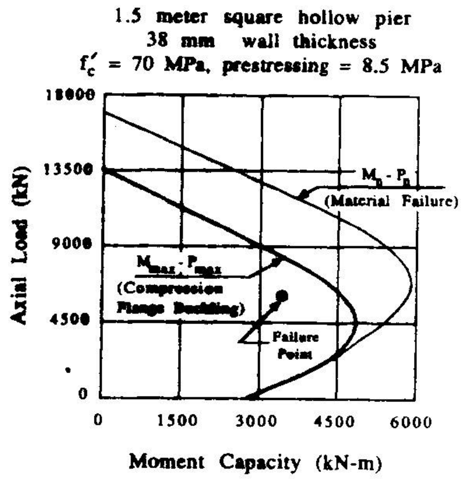

Fig. 5 Local stability effects on a thin wall pier section

innovation present. Often when a major innovation is introduced in a traditional industry such as construction, it must come from outside. Often it comes from a chemical firm, or an equipment manufacturer. Seldom does it come from within the concrete industry, which is fragmented and penurious in its research and development expenditures. This innovation by invaders tends to be a characteristic of traditional industries. Synthetic fibers were developed by the chemical industry and not by the cotton and wool-based textile industry.

Technological change often depends on the decisions of senior management. In the United States we see important structural concrete usage changes as almost generational changes. Technology changes as senior management changes. New managers are less tied to old experiences and are more open to experimentation. We must constantly improve our lifelong educational systems to bring about an openness to new ideas in our managers. Fig. 4 dramatically indicates a rapid development rate which can no longer be tied to the experience of a generation.

Many feel that they should not interrupt present practice, retrain personnel or change equipment in order to take advantage of technological innovations in the construction industry. Based on a comprehensive study of modern industries, Crowther [6] has concluded that "In the short term, higher productivity is more important than technical innovation, but in the long term, the contrary is true." It is a challenge for each of us to determine what are the appropriate short term cycles and what are the effective longer term cycles in which we are engaged. New materials, new processes, new designs should continually evolve with these longer term cycles so that we continue to make effective use of our resources. Gilbreath [8] tells us that "Change seldom sneaks up on our companies or attacks without warning. Astute managers constantly peer into the future, over the horizon of their immediate needs, in order to sense impending change and better prepare for it."

2.2 Resistance to Innovation

A detailed study of techological innovations by Crowther [6] indicates that economic and sociological factors were most important in stimulating or obstructing innovations. In the Western World, the basic criterion for filtering out certain technical innovations, and applying others remains economic profitability [3]. In the highly fragmented concrete industry this poses a difficult problem. The chemical company which introduces a new chemical admixture must make a series of multiple sales in order to actually sell its product. It must convince the architect and engineer representing the owner that the product will improve the concrete performance and that the product will have no detrimental side effects. It must convince the ready mix concrete supplier that it is dependable, repeatable, easy to dispense, and will produce a positive economic return. It must convince the constructor that he will profit by its use, that it will not delay placement or finishing operations and that it is thoroughly dependable as far as setting and strength characteristics. In order to sell the innovation, the proponent must develop a win-win-win-win situation, where everyone involved sees an advantage from their point of interest. Few industries have such diverse and fragmented multiple decision-makers.

Concrete construction often involves strongly organized labor forces. Innovation tends to threaten the uninformed within an industry. In Great Britain during the period from 1811-1816, the "Luddites" were a group of textile workers who rioted and destroyed textile machinery because they believed its introduction into textile mills would diminish employment. In _The Challenge of Hidden Profits_, Green and Berry [9] describe "Contemporary

Luddites" who "include balky workers and union leaders, as well as white-collar managers who ignore the need to innovate." Thus, one of the great challenges to innovation is to inform and educate the labor and managerial forces involved with the innovation.

Education is needed to overcome industry's resistance to innovation. Gilbreath [8] indicates that to come to grips with change, we must come to grips with those whose business fantasy is a world stopped and standing still for them. The world of structural concrete today is a far cry from that of the 1950's. The vastly higher strength concretes, the dramatic impact of prestressing, the heightened awareness of the importance of durability and corrosion resistance, the more efficient forming and placing techniques, all mean that professional and managerial leaders, originally educated in the 50's and 60's, must be continually "retooled" to be aware of today's problems and opportunities. Particularly in some public sector positions, continuing education has been neglected and decision-makers have become outdated. It is particularly important that our national and international regulatory standards stay abreast of innovation to place pressure on these laggards to allow proven technological developments.

Many charge our regulatory standards as being the "front line" of the resistance to change. It might be more realistic to think of them as a "Maginot Line," since the ingenious can penetrate them with relative ease. Regulatory standards must defend the public against ill-considered innovations. However, if the standards are well-developed and are basically performance-oriented, they should not be serious barriers to innovation. The fundamental conflict arises in that most structural concrete regulatory standards have been developed on a largely empirical basis from observations of both laboratory experiments and performance of actual construction. Elaborate theories have been developed to extend these empirical theories to general cases. The theories are only as good as the data bank on which they are based. Ritter, Morsch, Talbot and Richart never thought of 100 MPa concretes and 500 MPa reinforcing bars when they laid the foundation for reinforced concrete shear theory. Most tests of reinforced concrete beams have been run with concrete compressive strengths below 60 MPa. Our data bank is thin when we must extrapolate poorly defined, empirical theories to material strengths 200% and 300% higher than the preponderance of tests. The lack of comprehensive structural concrete tests on members with high strength materials is one of the great obstacles to liberalization of regulatory standards to permit full utilization of recent innovations.

3. CHALLENGES OF INNOVATION

3.1 Organizational

The concrete industry is a veritable beehive of activity. A glance at the papers to follow in this session will show that concrete materials and processes are evolving at a dramatic rate. Fly ash, silica fumes, pulverized slags, and improved mix techniques have revolutionized concrete compressive strengths. Latex modifiers, fibers, and polymers have dramatically improved tensile strengths. Stage post-tensioning is becoming commonplace. Glass and carbon fiber reinforcement can outperform steel reinforcement in some specialized applications. Coatings for reinforcement and for concrete slabs have made substantial inroads on corrosion and durability concerns. Our professional organizations must take a lead role in disseminating information about these innovations in a balanced, objective way. Conferences, symposia, proceedings, manuals, and committee reports must probe, evaluate, and synthesize this mushrooming experience. The rapid growth of information means it is no longer sufficient to publish only unrelated descriptions of innovations. Our professional organizations must stimulate development of fair, balanced synthesis reports that encapsulate the experience of many for the information of those considering use of a new innovation.

3.2 Individual

The rapid changes in structural concrete technology as we adjust to limit states design utilizing the full potential of the rapid changes in materials shown in Figs. 3 and 4 offer a personal challenge to each of us whether manager, designer, constructor, material supplier, teacher, writer, or researcher. The ability to effectively cope with these changes requires that we individually become more flexible and adaptable. We must change our outlook towards innovation from that of a mindset which discourages change to that of a mindset which looks and listens for change, correctly evaluates the opportunities in change, and chooses promising innovations for implementation.

3.3 Professional

The rapidity of change in our industry brings awesome new burdens. Drucker [5] clearly stated the challenge to our professionals as:

> "Knowledge, during the last few decades, has become the central capital, the cost center, and the crucial resource of the economy. This changes labor forces and work, teaching and learning, and the meaning of knowledge and its politics. But it also raises the problem of the responsibilities of the new men of power, the men of knowledge."

The essence of professionalism is that one has great skill or experience in a particular field or activity. These skills and experiences are built slowly and laboriously. As our materials change, so must the knowledge of our professionals. The rapid innovations in our industry means that serious and continuous self-study, experimentation and involvement are required of all of us who wish to remain "professionals" in structural concrete. To do less is to betray the public trust.

3.4 Side Effects

The most important challenge in my mind and the one which is frequently most neglected is the tendency to rush new innovations into use without carefully questioning their side effects on structural performance. Any new technology should be required to demonstrate in advance of its use the potential side effects on basic structural members and actions as well as the long term impact on durability. Seldom is such broad and comprehensive testing performed before the new innovation is brought to the market place.

Several recent examples come immediately to mind. Epoxy-coated reinforcement has been put in wide usage in bridge deck construction in the USA because of the dramatic reduction in corrosion when exposed to deicing salts. In the pilot program research, the effects of the coating on bond strength were evaluated by tests in which the bars were pulled out of concentrically loaded cylinders. It was concluded that the bond strength decrease was slight and no changes in development length were required. The coated bars were put in wide usage with no tests in structural members. Literally thousands of metric tons are being placed monthly. Recent comparative tests in lapped splice beam tests indicate that the bond strength is severely effected when splitting type failures can occur, as is usual in beam applications. The splice and development lengths must now be dramatically lengthened. The rush to market with insufficient structural member tests results in a situation where we have an appreciable number of structures in place with somewhat suspect connection details.

Short term testing is also suspect. Sometimes accelerated durability testing gives mistaken indications. Often little such testing is done. The U.S. Army Corps of Engineers Station at Treat Island on the Bay of Fundy subjects concretes to daily salt water immersion and hundreds of cycles of freezing and thawing annually. Twenty-five years of exposure tests on post-tensioned beams have provided much information on proper protection of prestressing components. However, by the time we know what works, the systems tested have disappeared from the market. We were not visionary enough in formulating programs. Of even more concern is the behavior of some epoxy composite protection systems cast as end blocks. In the first ten years, this system gave the best protection and was highly recommended. Over the second decade, it broke down badly at the material interface. It is now obvious that it is a poor solution. We cannot always accelerate nature. Durability studies are expensive, time-consuming, and fallible. This is a major concern in bridge applications where we need to think more in terms of centuries than decades. We must do much more to improve our ability to foresee the side effects on durability.

A final example indicates that there are indirect limits on the development of concrete compressive strength which parallel the previously discussed practical limit of the sound barrier on commerical aviation travel. In an exploratory study of the utilization of very high strength concretes in bridge construction, Jobse [10] tested very thin wall hollow pier sections made from high strength concretes. The vastly improved concrete compressive strength allowed use of very thin cross sections However, as shown in Fig. 5, a very unwelcome side effect occurred. The outer interaction curve is the expected strength of the section based on our normal assumptions, measured material properties, and known dimensions. If the compressive strength of the thin plates is limited using plate stability theory, the column strength would be reduced to the interaction curve shown by the inner solid line. The actual test specimen failed at an even lower load after suffering a local buckling failure of the thin wall member. One of our greatest challenges is that of using the new material strength possibilities wisely and safely. As we go to thinner members, stability problems may prove to be concretes' "sound barrier." Our Codes and Standards must meet these challenges which were relatively unimportant when 20 MPa or 30 MPa concrete resulted in "automatically" stiff members.

3.5 Regulatory Standards

Perhaps the most difficult challenge to meet is that of adequate regulatory standards such as the code of practice or materials specification. On the one hand we need to encourage and reward innovation. On the other hand we need to protect the public safety and public interests. We need to make sure the innovations are safe, durable and serviceable. We must refine our systems of building regulations and standards to satisfy two conflicting purposes. We need to have comprehensive, performance-oriented regulations which will require adequate demonstration that new technology is reasonably safe and durable before allowing public usage. In fairness to the entrepreneur who must shoulder much of the cost and burden of demonstrating the suitability of this new technology, there should be clear incentives which encourage and reward those technological innovations which are safe and durable. There must be appropriate and timely government and private machinery developed to assist in the review of such new products. Once again, France has taken a leadership role in showing how such machinery can be made possible as in the agrément system.

4. CONCLUSIONS

The gaps which will give the structural engineer the most challenge in our coming century are not the highways, the rivers, or the seas which must be bridged. The gaps with which we must be concerned are the human ones. Our technology is changing at a rate that is outrunning our ability to fathom and control. Innovations in materials are being introduced with little demonstration of their effect on structural behavior. Codes and standards lag product development significantly. Our human shortcomings should not stifle creativity and innovation. We must change our control systems to be in harmony with development. We must not become the barriers to innovation. Through a conference such as this, we hope to learn of the new innovations. With the type of free and open technical interchange and discussion to be expected, with the type of excellent papers submitted for the proceedings, and with the participation of all registered for this symposium, I know that innovation will be advanced.

ACKNOWLEDGMENTS

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Innovations in Construction Equipment, Machinery and System

Innovations dans les machines, équipements et systèmes de construction

Neuerungen von Bauausrüstung, Maschinen und Verfahren

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Sukenobu Momoshima, born 1925, received B.S. and D. Eng. degrees from the University of Tokyo. Dr. Momoshima is a pioneering engineer in prestressed concrete construction in Japan, and responsible for design and construction of many prestressed concrete bridges. He has played important roles in IABSE, FIP as well as JSCE.

SUMMARY
This paper deals with the recent progress made in concrete construction technology in Japan. An intensified utilization of electronics and other leading technologies has led to the development of automated construction equipment/machinery, intelligent construction systems and sophisticated inspection techniques. Driving forces for the development are the needs in construction work to save labor, to enhance precision, to reduce time and cost, and to improve the working environment.

RÉSUMÉ
Cet article traite des récents progrès dans la technique de construction en béton au Japon. L'utilisation intense de l'électronique et d'autres technologies de pointe ont conduit au développement d'appareillages et de procédés de construction automatisés, de systèmes avancés de construction et de techniques de vérification sophistiquées. La nécessité d'économiser la main d'œuvre, d'augmenter la précision, de réduire la durée et le coût de construction, d'améliorer le milieu de travail, produit une force motrice favorable en vue de ce progrès.

ZUSAMMENFASSUNG
Dieser Bericht zeigt die neueste Entwicklung der Bautechnik in Japan. Die intensive Anwendung von Elektronik und anderen Spitzentechnologien führte zur Entwicklung von automatisierten Bauausrüstungen und zu computergesteuerten Bauverfahren mit einer hochentwickelten Qualitätskontrolle. Diese Entwicklung wurde gefördert durch die Notwendigkeit, Arbeitsplätze im Bauwesen zu erhalten, die Präzision zu erhöhen, Zeit und Kosten zu sparen sowie das Arbeitsklima zu verbessern.

1. INTRODUCTION

Recent advancement in electronics and robotics engineerings has enabled the manufacturers to realize extensive automation and robotization of industrial equipment and machinery. Following this advancement, vigorous and intensive efforts have been made in Japan to utilize the same frontier technologies in the field of construction.

Motivation of the efforts toward the construction innovation stems from the needs to save labor, to enhance quality and precision, to reduce time and cost, and to improve work environment. Labor saving in construction work becomes more and more important for the labor-intensive construction industry so as to cope with the shortage of skilled labor and the increase in number of workers at advanced ages. Recognition of importance to improve work environment also necessitates the development of construction robots, to which the hazardous tasks in harsh environment are to be assigned.

In the robotic applications to construction, peculiarities of construction operations such as fragmentation of projects into separate phases and uncertainty in weather and environmental factors make innovation more difficult than in other industries.

With the above technical and social background, several innovations in concrete construction have been made in Japan.

This paper gives an outline of the recent development in Japan's concrete construction technology regarding automation and robotization of construction equipment, machinery and systems. The paper will also refer to the possible further advancement of the concrete construction technology in the near future, to be expected to follow with progress in the computer hardware and software technology.

2. AUTOMATION/ROBOTIZATION OF CONSTRUCTION EQUIPMENT AND MACHINERY

2.1 General

In the field of construction, many types of automated equipment and machinery are successfully developed, some of which are called "robots." Basic technological issues concerning the development are sensing, mobility, control and system integration. The critical attributes of a robot are considered to be mobility, autonomy, capability to deal with large forces, and cognitive ability to cope with harsh environments.

Several successful examples are presented below.

2.2 Computerized Concrete Batching Plant

In order to provide high production rate, to meet diversified demand, and to efficiently control the quality of ready-mixed concrete, an effort to establish a computerized concrete batching plant is being made. Systematic operation of delivery trucks, batching, loading, testing and accounting is expected to be included in the plant system.

The new quality control system of ready-mixed concrete is linked with advanced instrumentations such as sensors for measuring surface water of aggregates, and water content and slump of fresh concrete.

2.3 Shotcreting Robot

Shotcreting has been widely used in Japan since 1975 corresponding to an increase in tunnel construction by the New Austrian Tunnelling Method (NATM).

Environmental condition of shotcreting in tunnelling is very poor mainly due to dust caused by rebounded materials and noise. To cope with this environmental

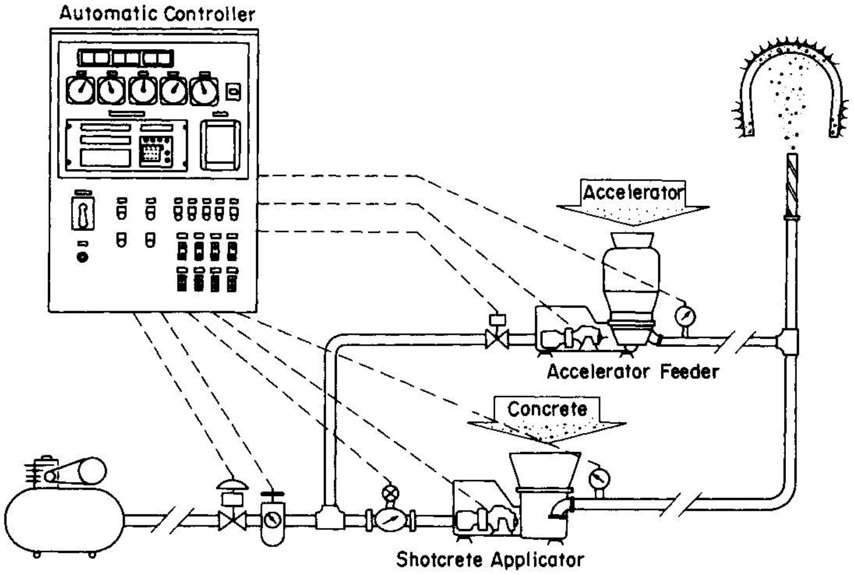

Fig. 1 Shotcreting control system

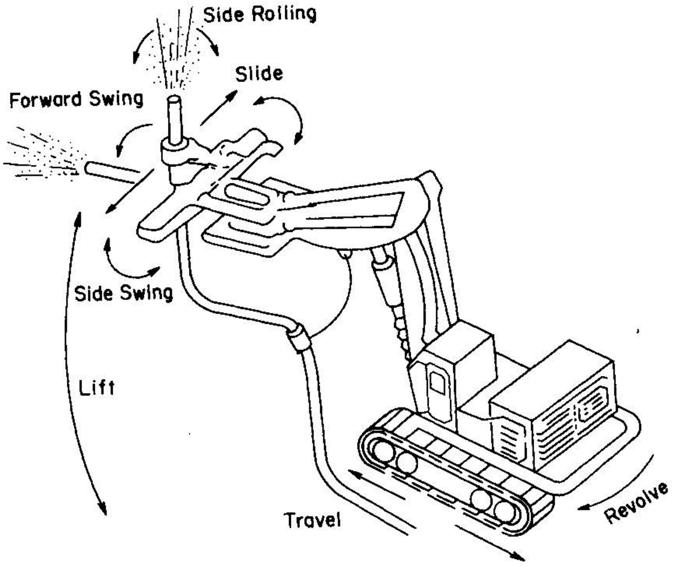

Fig. 2 Movement of shotcreting robot

problem and to secure construction efficiency, various types of shotcreting robots, or automated machines, have been developed and utilized in the NATM tunnelling or underground rock cavern construction.

Figs.1 and 2 illustrate an example of the automated shotcreting system for controlling quality and amount of concrete. Fig.3 shows a picture of the robotized shotcreting equipment of semi-automatic type. In Japan, more than 160 shotcreting robots of these kinds have been successfully put into practical use.

The requirements for securing quality and precision of shotcreting are as follows:

- nozzle should be positioned normal to the surface to be shotcreted,
- distance between nozzle and shotcreted surface should be maintained constant,
- moving rate of nozzle should also be as constant as possible.

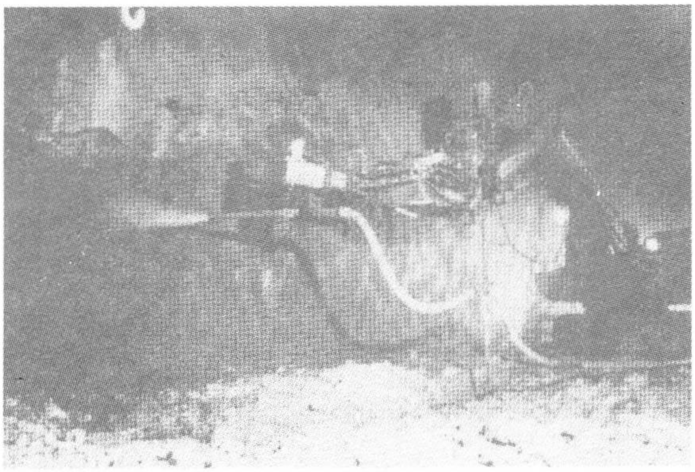

Fig. 3 Shotcreting robot

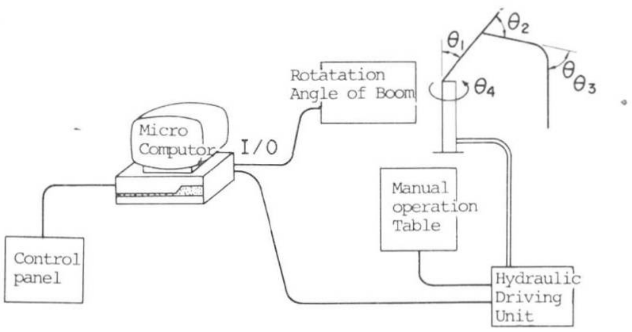

Fig. 4 Control system of concrete distributor

2.4 Concrete Distributor

Operation to move the flexible hose at the tip of piping in concrtete pumping and to change the piping layout is cumbersome and may impair the well-arranged reinforcing bars, resulting in degradation of quality of reinforced concrete construction. To cope with this inherent drawback of the concrete pumping method, a new concrete distributor has been developed.

Fig.4 shows an example of the automated concrete distributing system which is controlled by a microcomputor. With the system, concrete distribution to an adequate place is optimized at the most suitable intervals and rate. Movement of the articulated hose is controllrd by the microcomputer, and remotely-controlled vibrators are attached to the system. Use of the concrete distributor would impose minimal disturbance on the reinforcement.

2.5 Concrete Floor Finishing Robot

An optimum time for the finishing work differs depending on quality of concrete used and on an ambient temperature conditions. In some cases, the work has to be conducted at midnight. Furthermore, when finishing surface of concrete floors over a wide area, it may be difficult to secure the high flatness unless a number of very skilled trowelling workers are put in.

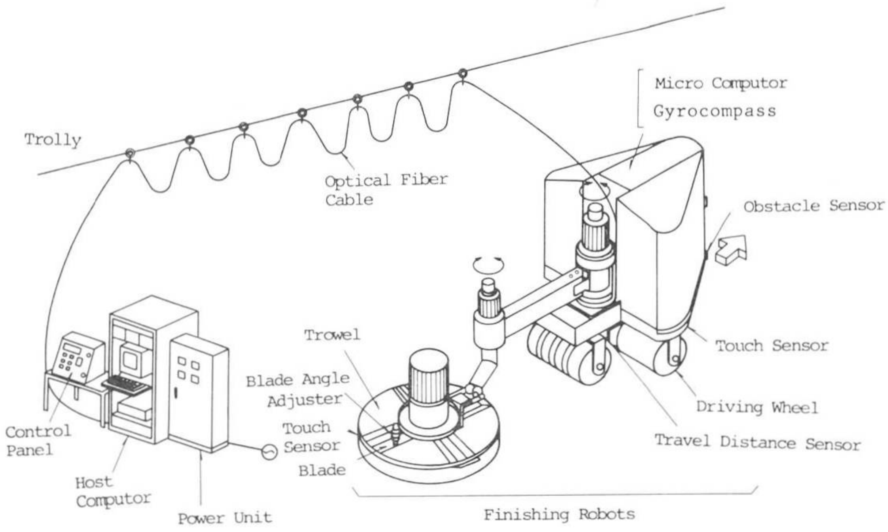

Fig. 5 System of floor surface finishing robot

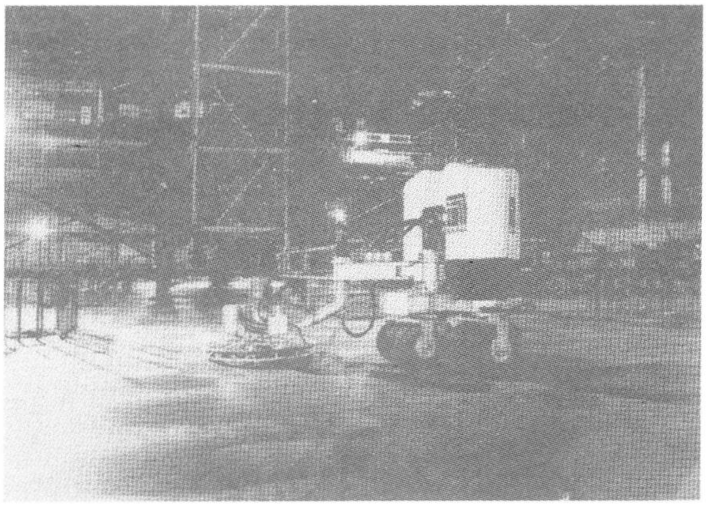

Fig. 6 Floor surface finishing robot

The necessity of development of concrete floor finishing robot thus stems from the needs to obtain a stable and high quality finishing, to save labor and to eliminate overnight work of the workers as much as possible.

Several floor finishing robots have been developed and successfully utilized in actual constructions. Fig.5 illustrates the system, and Fig.6 shows an example of surface finishing with the robot.

The robot is equipped with a travel distance sensor and a gyrocompass which enable the robot to perceive its own position and the flatness of finished surface. The robot is designed to be lightweight and provided with roller-type wheels for the work on soft concrete at a very early age. In the course of developing the robot, work standard on the surface finishing method and required flatness of finished surface has been established experimentally, upon which the operation criterion is based.

Experience obtained to date indicates that the quality of finished surface by the robot is as good as that by skilled labor, that operation is speeded up by

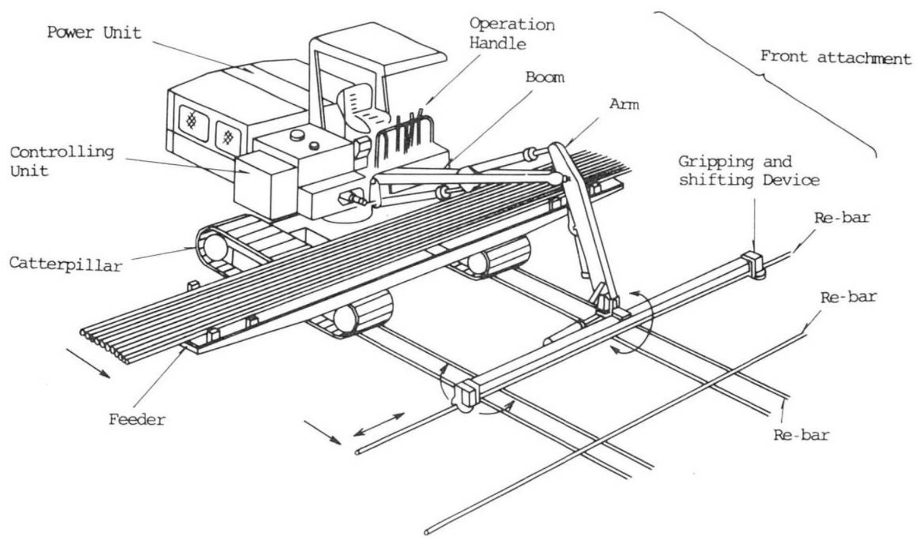

Fig. 7 Scheme of rebar arranging robot system

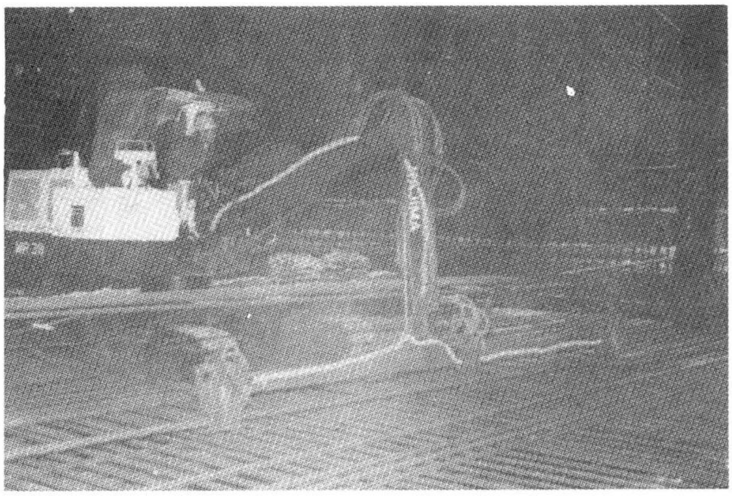

Fig. 8 Rebar arranging robot

50 % compared to troweling by manpower, and that manpower can be saved substantially.

2.6 Heavy Rebars Arranging Robot

In a construction project of the large scale structures such as nuclear power plant facilities, large-sized reinforcing bars up to D51 (nominal diameter: 51 mm) are frequently used. Arrangement work of such heavy reinforcing bars in a limited working space is very hard in terms of weight and quantity, and, in most cases, very time-consuming.

To cope with the above inconveniences, a rebar arranging robot has been developed. The schematic drawing of the robot is shown in Fig.7, and Fig.8 shows a picture of the robot in operation.

The robot, once installed in a predetermined position, puts reinforcing bars in

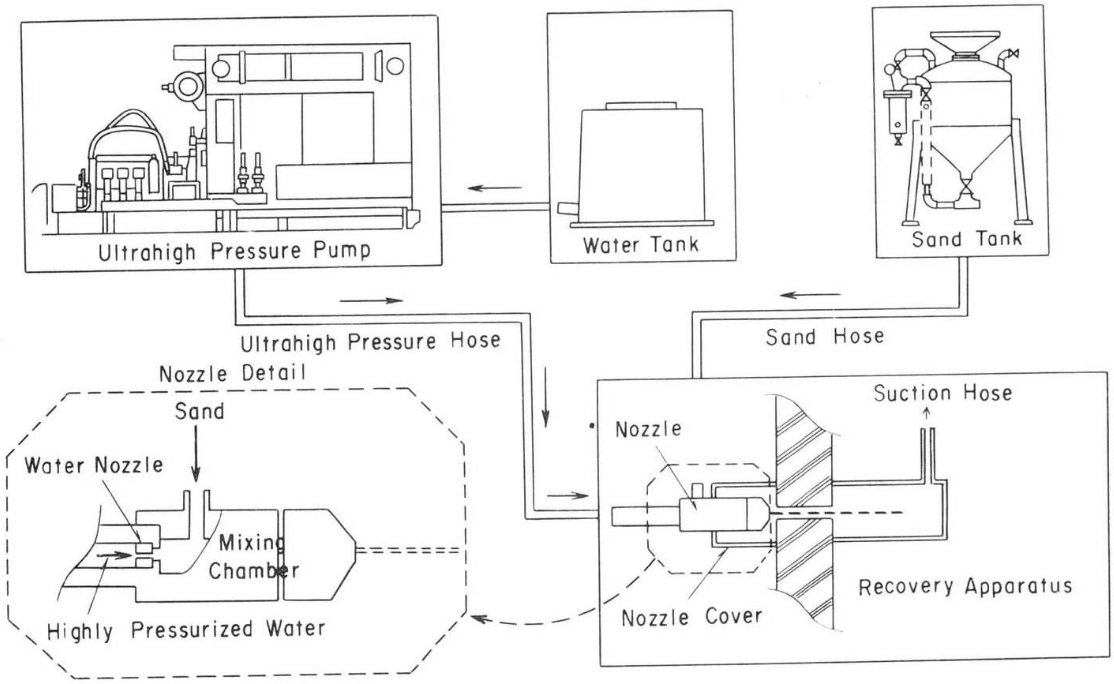

Fig. 9 Scheme of abrasive jet cutting system

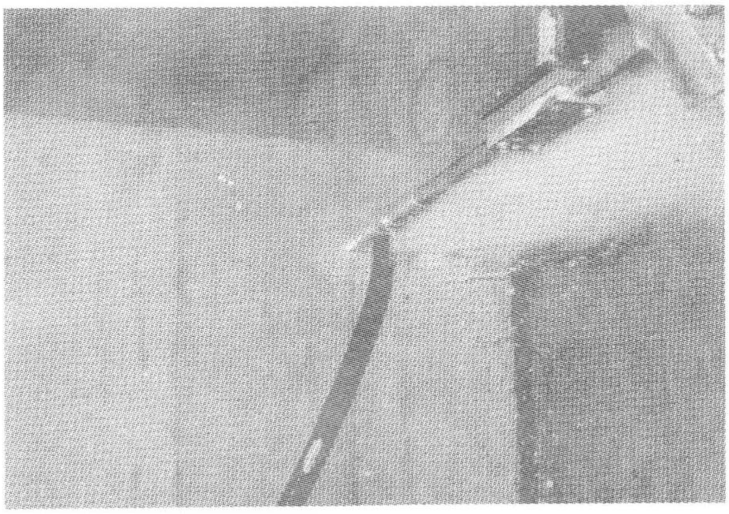

Fig. 10 Abrasive jet cutting system

a specified position at proper spacings. The spacing can be arbitrarily selected between 1 cm and 99 cm. Main body of the robot consists of engine, crawler, rebar feeder, and rebar handling device with controlling unit for positioning. In the actual application, either of manned operation or remotely-controlled operation can be chosen.

From an experience in a construction project of nuclear power plant facilities, in which D38 rebars with a length of 12 m each (100 kg/piece) were used, the following have been indicated.

- rebar arranging work can be carried out by the robot with good precision,
- labor-saving and speed-up of the work can be acomplished compared to the conventional method with a team of five to seven workers,

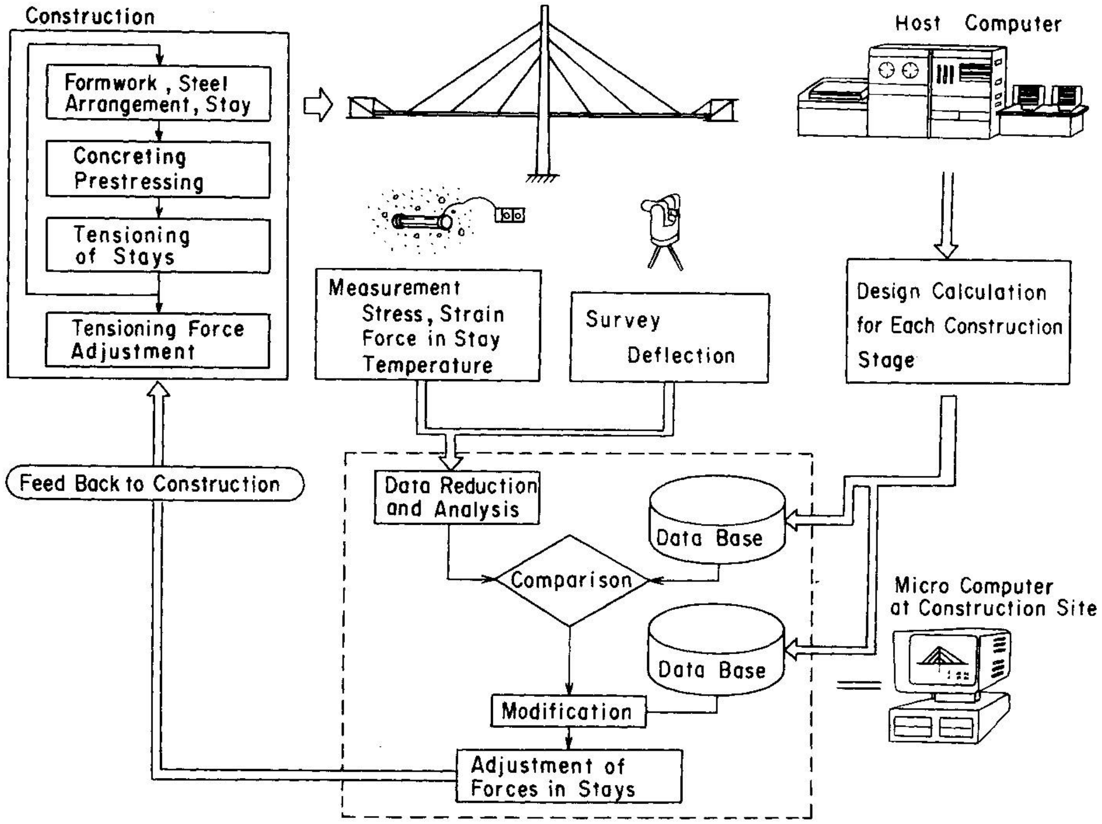

Fig. 11 Deflection control system for prestressed concrete cable-stayed bridges

- dangerous and hard work, especially for elderly workers, is eliminated.

2.7 Ultrahigh-Pressure Abrasive Jet Cutting Machine

As a tool of construction, reform and demolition of reinforced concrete structures, an advanced application of the conventional sand-blasting system to concrete cutting techniques has been realized. The schematic drawing of the ultrahigh-pressure abrasive jet cutting machine developed is shown in Fig.9. Fig.10 shows a picture of the machine in operation.

With the machine, concrete cutting operation can be done efficiently and speedily, and reinforced concrete members with depths of up to 150 cm can be cut without vibration problem and with noise being minimized. The pressure can be increased up to 250 MPa.

3. INTELLIGENT CONSTRUCTION SYSTEM

The basic idea of the intelligent construction system is to acquire data on construction timely, to reduce and analyze them promptly, and to reflect the obtained information, including results of the analysis, to the ongoing construction. Such system will rationalize the control and management of construction practice and will also pay off in the economics of construction. Needless to say, the intelligent systems of such kind are realized by the use of computer hardware and software technology.

Examples of the intelligent construction system are as follows:

- slip forming with online data desplay,

- high-precision ultradeep slurry wall excavation system for the construction of inground reinforced concrete diaphragm walls,
- sinking operation of large-diameter concrete caissons for the side walls of LNG inground reinforced concrete tanks,
- surveying and geometry-controlling system of the prestressed concrete bridge during construction, which is illustrated in Fig.11.

4. FUTURE DEVELOPMENT

4.1 Further Advancement of Robotization

The attempts made to date in Japan to utilize robotics engineering in concrete construction technology represent the first step of developing the various construction robots into more mature technology. In applying robotics to concrete construction, rather simple problems have been chosen in expectation that increased sophistication of the prototypes will follow as experience is gained. In other words, the current construction robotics are regarded to be in an embryonic state.

Many aspects of robotic-based construction shall differ from conventional construction technology, and there will be a fundamental rethinking of the entire design process of concrete construction as robots move into the workplace more and more. Economic feasibility is also important because of the amount of money involved to conduct the research and development.

4.2 Direction of Future Development

Robotics-related needs for the future development include: continuous sophistication of indicators and sensors; and integration of project data bases on design and construction. Research on artificial intelligence and laser beam will also be a matter of great interest.

Robotization of transportation, placement and compaction of concrete is foreseen, noticing that placing and compacting of concrete in the present concrete practice depend largely on experience of skilled labor.

It is also safe to say that robotics engineering will certainly play a key role in dismantling of aged nuclear power plants. Thus, the more sophisticated autonomous robots will be developed and extensively used in a harsh work environment.

4.3 Impact of New Materials

Development of new materials often results in realization of the new types of structures and construction methods, and will probably enhance innovation in concrete construction technology.

Among such new materials are:

- carbon fiber and aramid fiber which have been used in a form of fiber reinforced concrete, and now are considered to be used in a form of rods as a promising reinforcement for the concrete structures that never rusts.
- anti-washout concrete, or a specially developed underwater concrete named "Hydrocrete," which is to replace the conventional preplaced-aggregate concrete, and on which a great deal of research and development have been carried out.
- roller compacted concrete (RCC), or "Rollcrete," which greatly facilitated the construction of the gravity-type dams, and on which the innovative development is still actively being under way, especially for its application to concrete pavement.

5. CONCLUSIONS

The present status of innovation in concrete technology in Japan has been presented with respect to utilization of robotics and automated equipment.

Robotics utilization in construction has just begun, and a number of more successful applications are expected to come, bringing about a great advance in construction which results in higher quality and better performance of construction activities.

The Japanese construction companies as well as the public sector are significantly involved in the cultivation of further developments in robotic technology, and extensive corporate-funded R&D efforts have been made to devise innovative equipment. A Japanese government-sponsored research project has also been underway for these few years to develop robotics equipment for the most difficult of tasks such as maintenance and inspection chores in nuclear-power reactors and high-pressure work undersea.

With the experience in applied utilization of the automated machines already developed, the robotics technology in the construction field will be continuously fostered, noting the following:

- technological advancement is an important means for the construction industry to meet challenges of increased cost-effectiveness and improved competitiveness.
- technological advancement will provide significant new incentives for innovation and technologically based competition in the construction industry.

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Évolution récente des ponts en béton précontraint

Die jüngste Evolution der vorgespannten Betonbrücken

Latest Developments in Prestressed Concrete Bridges

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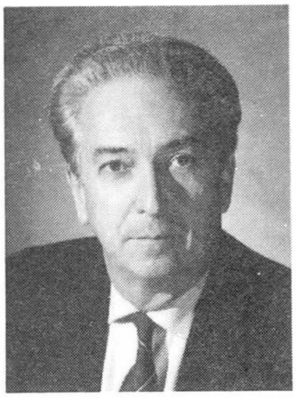


Jacques Mathivat, né en 1932, a été jusqu'en 1977 Directeur des Études des Entreprises Campenon Bernard. Il est actuellement Ingénieur-Conseil de SECOA (Société d'Études et de calculs en Ouvrages d'Art) qu'il a créée en 1978. Jacques Mathivat, Directeur Technique au SNBATI, est Membre du Conseil Exécutif de l'AIPC et Vice-Président de l'AFPC.

RÉSUMÉ
L'évolution des ponts en béton précontraint a été marquée, au cours de la dernière décennie, par le développement spectaculaire, dans une large gamme de portées, des structures haubanées, ainsi que par l'apparition de nouvelles méthodes de construction, comme par exemple la réalisation des ponts en arc par rotation verticale de chaque demi-arc. Mais deux autres tendances importantes, l'allègement des structures transversales des tabliers et le recours à la précontrainte extérieures au béton, sont apparues récemment, plus particulièrement sous l'impulsion des ingénieurs français grâce auxquels elles ont connu des applications intéressantes.

ZUSAMMENFASSUNG
Die Evolution im vorgespannten Betonbrückenbau war während der letzten 10 Jahren gezeichnet durch die spektakuläre Entwicklung auf dem Gebiet der seilverspannten Brücken und durch die Anwendung neuer Bauverfahren, wie zum Beispiel die Errichtung des Bogens durch die vertikale Rotation der beiden Halbbogen, beim Bau von Bogenbrücken. Aber auch zwei andere wichtige Tendenzen, die leichtere Ausführung des Ueberbaus in Querrichtung und die Anwendung der äusseren Vorspannung auf den Beton, sind ersichtlich. Diese haben dank französischen Ingenieuren interessante Anwendungen gefunden.

SUMMARY
Progress made in the construction of prestressed concrete bridges over the past ten years has been marked by a spectacular development in the area of cable-stayed bridges and by the application of new building methods as e.g. , erecting an arch bridge by vertically rotating both semi-arches. However, two other important tendencies have arisen recently , namely, employing lighter transverse structures for bridge decks and the use of external prestressing of the concrete. These techniques have found interesting application thanks to French engineers.

L'évolution des ponts en béton précontraint a été marquée, au cours de la dernière décennie, par le développement spectaculaire, dans une large gamme de portées, des **structures haubanées**, ainsi que par l'apparition de **nouvelles méthodes de construction.**

Mais deux autres tendances importantes, **l'allègement des structures transversales des tabliers** et le recours à **la précontrainte extérieure au béton,** sont apparues récemment, plus particulièrement sous l'impulsion des ingénieurs français grâce auxquels elles ont connu des applications intéressantes.

1 - DEVELOPPEMENT DES PONTS HAUBANES

Dans les les dernières années de nombreux ponts haubanés en béton ont été construits un peu partout dans le monde. Pour des **portées supérieures à 250 m,** ces ouvrages remplacent généralement les ponts traditionnels construits par encorbellements successifs.

Les **structures transversales** les mieux adaptées aux tabliers de largeur moyenne sont aujourd'hui (Fig. 1) :
- dans le cas d'une **suspension centrale,** une poutre tubulaire à deux âmes inclinées et à encorbellements latéraux, raidie par une triangulation intérieure ;
- dans le cas d'une **suspension latérale,** une poutre tubulaire en forme d'aile d'avion, sans encorbellements latéraux et à deux âmes fortement inclinées, dont le hourdis sous-chaussée est supporté par des poteaux.

Bien que la plus grande travée haubanée en béton - celle du pont de Barrios de Luna, en Espagne - ne franchisse que 440 m, des portées de **600** m à **700** m peuvent être atteintes avec les matériaux et la technologie actuels. Mais, pour de telles portées, une **structure mixte acier-béton** est certainement plus économique, surtout si elle associe des nervures longitudinales fortement comprimées en béton avec des entretoises transversales fléchies en acier. La même conception peut d'ailleurs s'appliquer à des ouvrages de portées modérées.

Une solution de ce type a été adoptée pour le pont du Boulevard urbain de Gennevilliers, dont la construction a malheureusement été différée (Fig. 2).
Une autre particularité de cet ouvrage réside dans la simplicité de sa suspension qui comporte des mâts uniques centraux et un **"haubanage en parapluie",** constitué de haubans latéraux traversant la tête des mâts et assurant efficacement leur stabilité élastique. La bonne tenue à la fatigue des ancrages par courbure des haubans à travers les mâts a été confirmée par des essais effectués au Laboratoire Fédéral de l'EMPA, en Suisse, à l'occasion de la construction du pont de Coatzacoalcos.

Dans le domaine des **portées inférieures à 250 m,** une nouvelle famille de structures, dites à **"précontrainte extradossée"** - qui sont décrites en 4 - constitue une transition économique entre les ponts traditionnels par encorbellements successifs et les ponts à haubans (Fig. 18).

2 - EVOLUTION DES METHODES DE CONSTRUCTION

Les tabliers en béton précontraint peuvent être classés en deux catégories, ceux qui sont construits à leur emplacement définitif et ceux qui sont construits à un emplacement différent - où leur exécution est plus facile - et mis en place par **déplacement.** Cette dernière catégorie a connu récemment quelques développements nouveaux.

Les tabliers peuvent être déplacés par **translation** ou par **rotation.**
La **translation** peut avoir lieu dans un plan horizontal et être transversale (ripage) ou longitudinale (poussage). La translation peut aussi être verticale

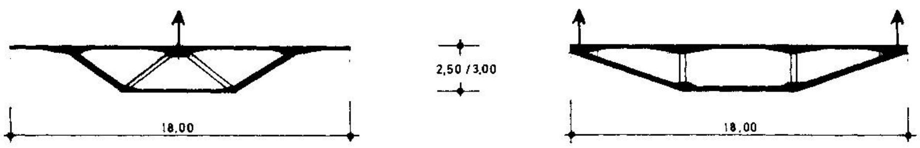


Fig. 1 Structures transversales des ponts à haubans modernes

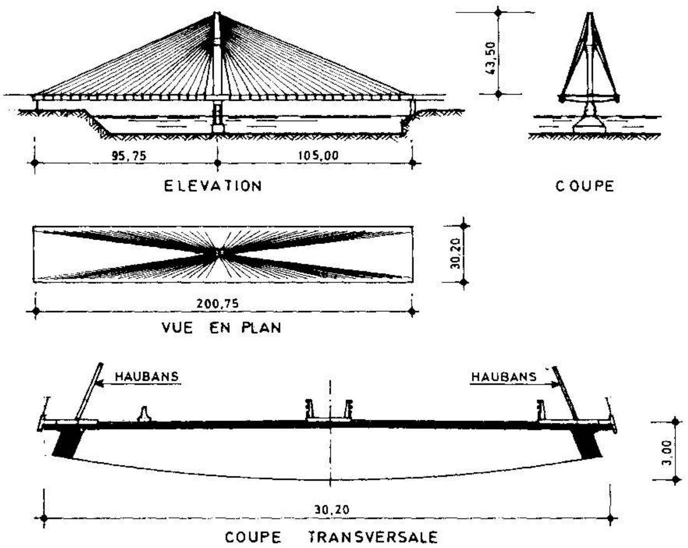


Fig. 2 Pont du Bd urbain de Gennevilliers (projet SECOA/SETRA)

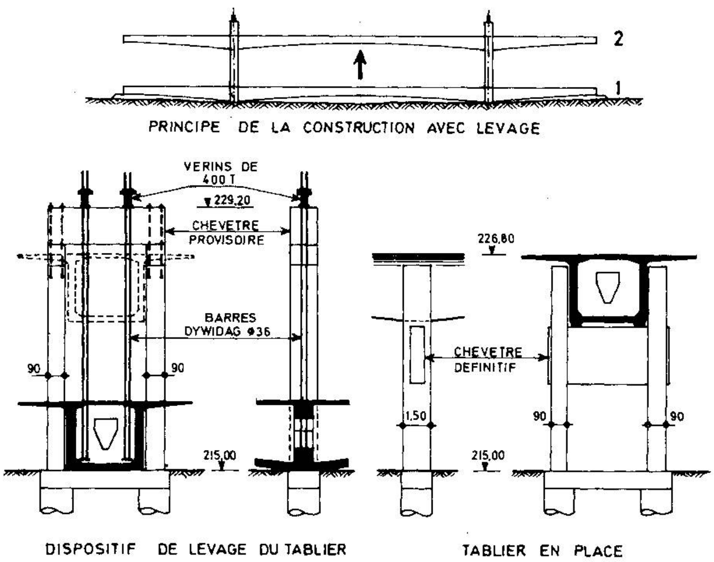


Fig. 3 Levage d'un tablier (projet Dragages et Travaux Publics)

et consister alors dans le **levage d'un tablier** construit à un niveau inférieur.

Cette méthode de construction a été envisagée par l'Entreprise Dragages et Travaux Publics pour un projet, qu'il nous parait intéressant de mentionner et dans lequel le tablier, exécuté au sol, était ensuite levé à partir des piles définitives de l'ouvrage conçues de manière à servir d'appuis pendant cette opération (Fig. 3). Le levage terminé et le tablier maintenu en position définitive, il était ensuite procédé à l'exécution des chevêtres des piles sur lesquels venait enfin reposer le tablier.

Dans le cas d'une **rotation**, celle-ci s'effectue généralement dans un plan horizontal autour d'un des appuis de l'ouvrage et d'assez nombreux ouvrages ont été construits suivant ce procédé, chaque fois qu'il était difficile ou coûteux de les construire directement au-dessus de la brêche à franchir.

Mais la rotation peut aussi avoir lieu dans un **plan vertical**. C'est le cas du pont en **arc** d'Argentobel, en République Fédérale d'Allemagne, qui a été construit par bétonnage des demi-arcs en position quasi verticale, et rotation de ceux-ci autour des culées (Fig. 4). Ce mode de construction peut également s'appliquer à la réalisation de **tabliers rectilignes de portées moyennes**, mis en place à l'avancement par travées entières lancées comme **un pont-levis** (Fig. 5). Le tablier, bétonné verticalement dans des coffrages grimpants au moyen d'une grue à tour située sur le tablier, est ensuite basculé par rotation verticale autour d'articulations provisoires disposées en tête de pile. Des haubans de retenue assurent la stabilité du tablier pendant son déplacement.

3 - ALLEGEMENT DES STRUCTURES TRANSVERSALES DES TABLIERS

Depuis de nombreuses années les projeteurs ont cherché à allèger la structure transversale des tabliers des ponts en béton précontraint en réduisant la section des **âmes**. Ces dernières, quand elles sont en béton, représentent en effet une part importante du poids propre du tablier (couramment entre 30 % et 40 % pour un tablier de hauteur constante) qui correspond à une répartition inefficace de la matière diminuant le rendement géométrique de la section (rendement géométrique passant de 1 pour une section idéale sans âmes aux environs de 0,6 pour une section tubulaire à âmes pleines).

La limitation de l'importance des âmes dans la section transversale entraîne donc une double économie au niveau de la précontrainte longitudinale du tablier, par suite d'une part de la réduction du poids propre et d'autre part de l'amélioration du rendement géométrique de la section, économie à laquelle vient s'ajouter le gain sur les quantités de béton à mettre en oeuvre.

Cet allègement de la structure transversale a été obtenu de différentes façons, notamment par :

3.1 - L'amincissement des âmes (Fig. 6)

3.1.1 en faisant **varier leur épaisseur** sur la hauteur du tablier de manière que cette épaisseur, à l'encastrement des âmes sur les membrures supérieure et inférieure soit proportionnelle au moment statique de la membrure adjacente.

3.1.2 en ayant recours de façon systématique à la **précontrainte verticale des âmes,** associée généralement à une précontrainte longitudinale horizontale logée dans les membrures supérieure et inférieure. Cette solution, qui a probablement été délaissée à tort en France depuis quelques années, devrait trouver dans l'avenir un nouveau développement avec l'utilisation de torons gainés graissés.

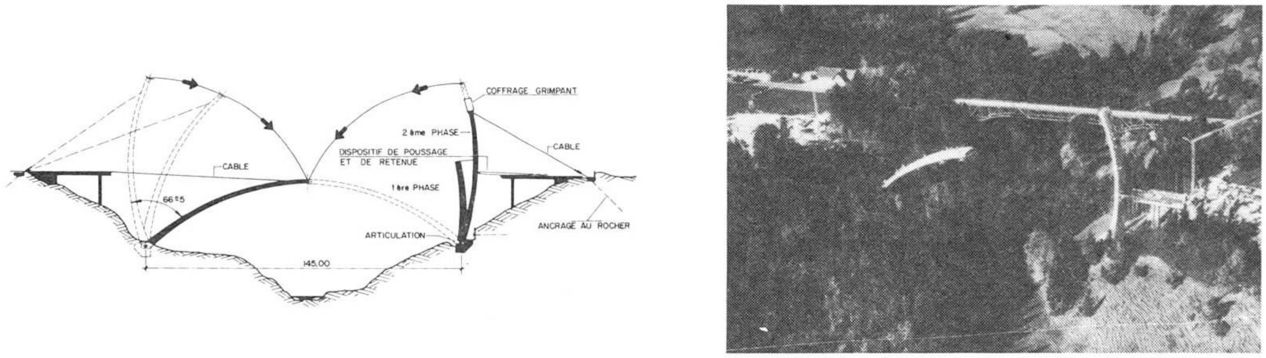


Fig. 4 *Construction d'un arc par rotation dans un plan vertical Pont d'Argentobel*

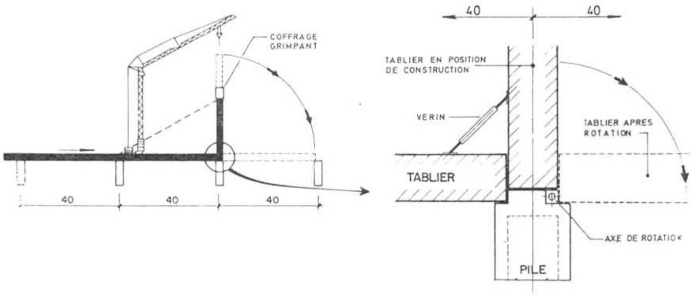


Fig. 5 *Construction d'un tablier à l'avancement par rotation dans un plan vertical*

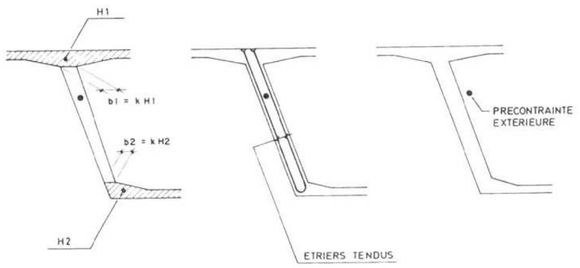


Fig. 6 *Amincissement des âmes traditionnelles*

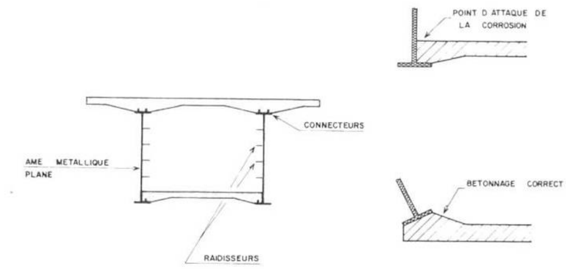
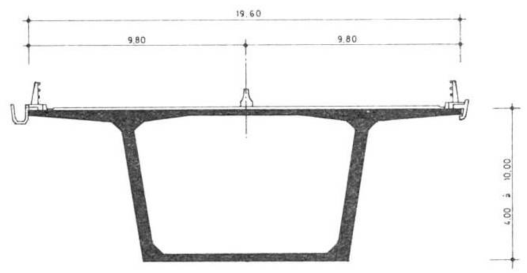
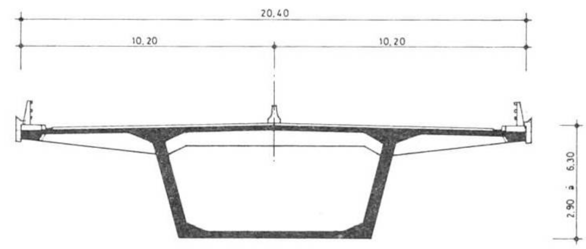


Fig. 8 *Position de la membrure inférieure en béton*





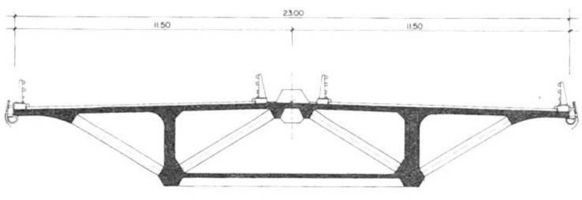


Fig. 7 *Coupes transversales des tabliers larges à deux âmes :*
Viaduc de Poncin (projet Spie Batignolles & Dragages et Travaux Publics)
Viaduc de l'Arrêt Darré (projet Spie Batignolles)
Pont d'Elbeuf (projet SETRA/SECOA)

3.1.3 en disposant la **précontrainte longitudinale du tablier à l'extérieur des âmes**, ce qui évite deux sujétions souvent prépondérantes pour leur dimensionnement, relatives l'une au bétonnage de l'âme et l'autre à l'ancrage des câbles de précontrainte longitudinaux. La **précontrainte extérieure** permet également de profiter de l'épaisseur totale des âmes pour la résistance à l'effort tranchant (suppression de la déduction d'un demi- diamètre de gaine conformément aux prescriptions du règlement français de béton précontraint - BPEL). Mais cet avantage disparaît dans certains pays étrangers où aucune diminution de l'épaisseur de calcul n'est exigée en présence de câbles de précontrainte à l'intérieur des âmes. La **précontrainte extérieure** fait l'objet du paragraphe 4.

Les trois moyens précédents peuvent être utilisés simultanément.

3.2 - La réduction du nombre des âmes en augmentant les portées transversales des hourdis sous-chaussée.

Il est habituel aujourd'hui de réaliser des poutres tubulaires à deux âmes de plus de 20 m de largeur, en constituant la dalle sous chaussée soit d'un hourdis épais (épaisseur de 0,30m), soit d'un hourdis nervuré, soit d'un hourdis traditionnel supporté par une triangulation ou des poteaux intérieurs ou par des bracons inclinés extérieurs.

Deux ouvrages récemment construits, les viaducs de Poncin et de l'Arrêt Darré, ainsi qu'un projet, celui du Pont d'Elbeuf, donnent des exemples de tabliers larges à deux âmes (Fig. 7).

3.3 -La substitution de structures plus légères aux âmes traditionnelles en béton

Ces structures peuvent être de différents types :

3.3.1.**âmes métalliques planes raidies**

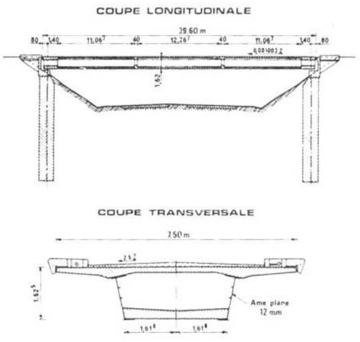
Cette solution est la plus simple car inspirée directement des ossatures mixtes acier-béton classiques. Elle pose toutefois deux problèmes spécifiques.

Le premier concerne la répartition des contraintes normales entre l'acier et le béton par suite du fluage qui, sous l'effet de la précontrainte longitudinale, transfère une part importante des efforts de compression des membrures en béton aux âmes métalliques. Ces compressions rendent nécessaire un coûteux raidissage des âmes, tant longitudinal que vertical.

Le second est relatif à la position de la membrure inférieure en béton par rapport aux semelles des âmes métalliques (Fig. 8). Il est préférable du point de vue constructif de placer la membrure inférieure au-dessus des semelles, ce qui permet un bétonnage aisé et éventuellement le lancement des âmes métalliques avant toute autre opération

Mais il en résulte un point de contact triple : air, acier, béton, le long des âmes, qui constitue une zone d'attaque privilégiée de la corrosion. Il est donc meilleur de placer les semelles au-dessus de la membrure en inclinant les âmes suffisamment de manière à rendre possible la mise en place du béton dans de bonnes conditions.

Un ouvrage expérimental, comportant une travée isostatique de 40 m de portée, a été réalisé sur l'Autoroute A 71, à Salbris, par l'Entreprise Fougerolle (Fig. 9).



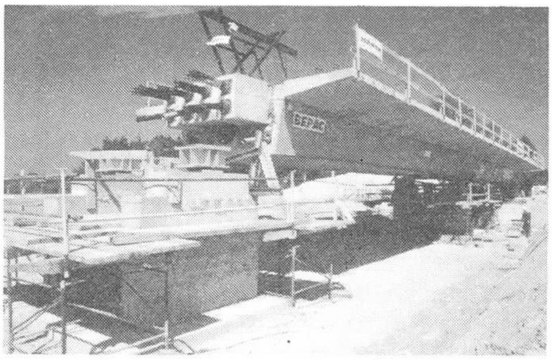
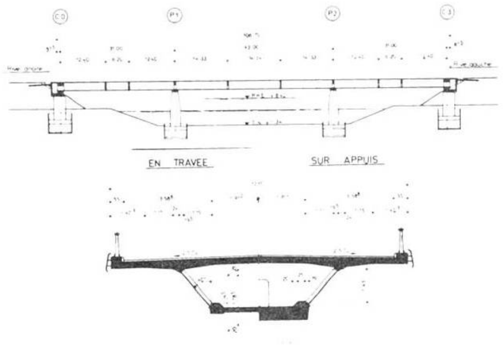


Fig. 9 Pont sur l'Autoroute A71 à Salbris (projet Fougerolle)



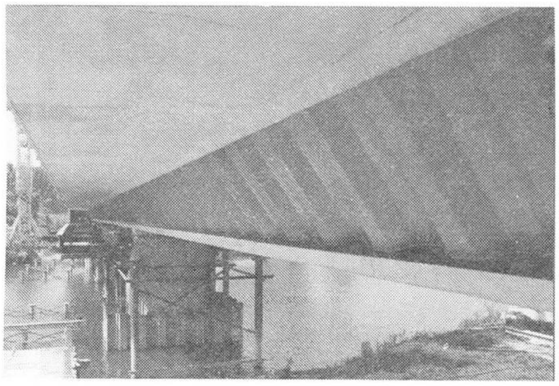


Fig. 10 Pont de Cognac (projet Campenon Bernard)

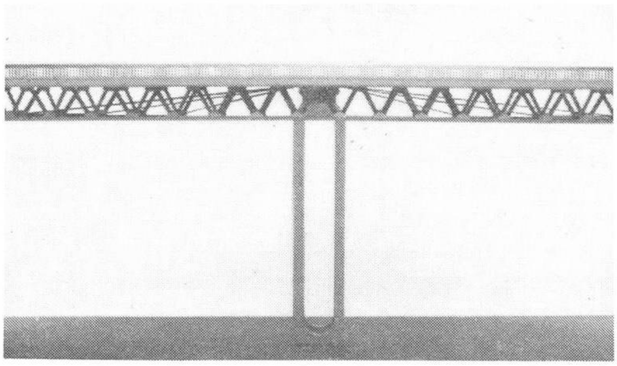


Fig. 11 Maquette du Viaduc de Charolles (projet SECOA)

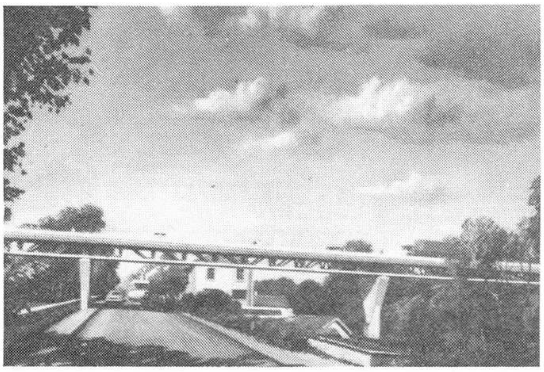
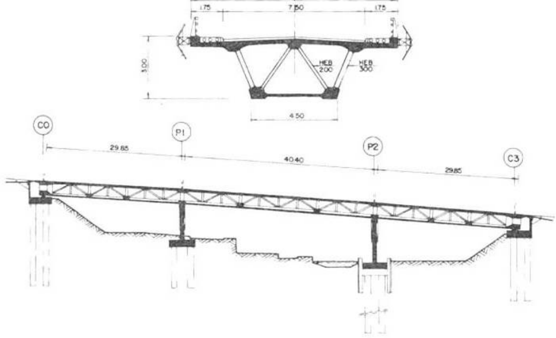



Fig. 12 Pont d'Arbois (projet Dragages et Travaux Publics et Société Générale d'Entreprises

3.3.2. âmes métalliques en tôle plissée

Une solution, imaginée par l'Entreprise Campenon Bernard, consiste à utiliser des âmes métalliques plissées que leur grande déformabilité longitudinale soustrait à l'effet des compressions longitudinales induites par la précontrainte.

Un choix judicieux des dimensions des ondulations permet d'obtenir une rigidité de flexion transversale comparable à celle d'âmes traditionnelles en béton et d'assurer ainsi la stabilité de l'âme au voilement et sa résistance à l'effort tranchant et à la torsion. Les âmes plissées posent toutefois le même problème que les âmes planes raidies vis-à-vis de la corrosion à la jonction de la membrure inférieure et des âmes. Des contraintes locales apparaissent également à l'enracinement des âmes dans les membrures par suite des raccourcissements du béton sous l'effet du retrait et du fluage.

Cette solution a fait l'objet d'une première réalisation expérimentale à l'occasion de la construction du pont de Cognac par l'Entreprise Campenon Bernard (Fig. 10).

3.3.3. âmes triangulées ou triangulation spatiale reliant les membrures supérieure et inférieure

Les âmes de la poutre peuvent être remplacées par un treillis plan ou par une triangulation spatiale, reliant les deux membrures et constitués d'éléments en béton ou en métal. Dans le cas d'éléments métalliques, ceux-ci sont généralement réalisés au moyen de profilés en H ou de tubes circulaires ou carrés.

Les problèmes principaux posés par ce type de structure concernent la forme de la triangulation et la conception des noeuds d'assemblage avec les membrures, qui doivent pouvoir reprendre les efforts de glissement et les efforts de flexion locaux.

La figure 11 montre le projet que nous avions établi pour le Viaduc de Charolles et qui comportait des âmes en treillis métallique avec des profilés en H. Le premier ouvrage réalisé suivant ce principe est le pont d'Arbois, construit par Dragages et Travaux Publics et par la Société Générale d'Entreprises (Fig. 12).

L'idée d'une triangulation spatiale a été développée pour la première fois par l'Entreprise Bouygues lors de la construction du Pont de Bubiyan au Koweit au moyen de voussoirs préfabriqués à joints conjugués constitués par l'assemblage en cellule des membrures supérieure et inférieure avec des triangles préfabriqués en béton (Fig. 13). Un principe analogue a été repris par la même entreprise pour les Viaducs de Sylans et des Glacières avec une triangulation constituée d'X préfabriqués en béton disposés transversalement selon quatre plans formant un W (Fig. 14). Dans ces deux derniers ouvrages les éléments de la triangulation sont de section pseudo-carrée.

3.3.4. âmes évidées en béton à montants trapézoidaux

Les solutions développées précédemment présentent certains inconvénients qui ont été indiqués en 331, 332 et 333.

D'autre part, en ce qui concerne les structures mixtes acier-béton, leur intérêt économique n'a pas encore été prouvé, tout au moins dans les portées moyennes, la plupart des réalisations à ce jour étant des ouvrages expérimentaux confiés directement à des entreprises ou le résultat d'adjudications lancées dans un cadre étroit n'autorisant pas d'autres alternatives. Le choix par l'Entreprise Bouygues pour ses ouvrages à triangulation spatiale d'éléments en béton au lieu d'éléments métalliques semble confirmer ce fait.

Dans le domaine des structures en béton, la triangulation spatiale est une solution assurément économique au niveau de l'allègement de la structure et des quantités de matériaux à mettre en oeuvre. Mais le grand nombre d'assemblages, qui constituent toujours les points faibles d'une construction, ainsi que la faible dimension des éléments de la triangulation, laissent peser quelques incertitudes sur le comportement dans le temps de ce type de structure.

Enfin, toutes les solutions précédentes s'accompagnent généralement d'une précontrainte longitudinale partiellement extérieure au béton dont le tracé nécessite la mise en oeuvre de bossages, diaphragmes ou entretoises, assurant la déviation ou l'ancrage des armatures. Ces éléments en saillie par rapport à la section transversale courante alourdissent le tablier et leur exécution, souvent difficile, en renchérit le prix.

C'est la raison pour laquelle nous avons imaginé de remplacer les âmes traditionnelles par des âmes évidées en béton constituées de montants trapézoidaux, qui, tout en diminuant le poids propre du tablier et en améliorant son rendement géométrique, facilitent le tracé de la précontrainte extérieure longitudinale. Un dessin de principe de ces âmes est donné à la figure 15.

Les montants trapézoidaux sont disposés "pointe en bas", la grande base des montants réalisant ainsi un encastrement continu de la membrure supérieure sous l'effet des charges d'exploitation, tandis que la pointe inférieure assure la déviation des câbles de précontrainte extérieure filants, sans bossages ni entretoises supplémentaires.

Cette disposition permet d'autre part le bétonnage aisé des montants et l'ancrage sur leur tranche de câbles de précontrainte dont la mise en tension peut ainsi s'effectuer à n'importe quel moment.

Du point de vue des quantités de béton à mettre en oeuvre, les âmes évidées à montants trapézoidaux se situent entre les âmes pleines et les âmes à triangulation plane. Elles représentent un gain d'environ 50 % sur le volume de béton d'âmes pleines de même épaisseur.

Elles offrent également plusieurs autres avantages :

- utilisation d'éléments plans (plaques) plus robustes et moins exposés à la corrosion des armatures que les éléments linéaires d'une triangulation (barres), qui ont une plus grande surface de contact avec l'atmosphère extérieure.

- suppression par rapport aux structures triangulées des noeuds de jonction entre éléments, de conception et d'exécution difficile.

Nous avons étudié, pour les Entreprises Quillery, un projet de ce type, à l'occasion de l'appel d'offre des Viaducs de Sylans et des Glacières. Cet appel d'offre comportait deux niveaux :
- un premier niveau , dit de préconsultation technique, destiné à sélectionner, en dehors de toute considération de prix, les variantes proposées par les entreprises. A ce stade de la consultation notre projet a été une des deux solutions retenues.
- un second niveau, de consultation proprement dite, dans lequel les entreprises devaient soumissionner à la fois le projet de base élaboré par le maître d'oeuvre Scetauroute et leur projet variante qui avait été accepté, en y incluant les modifications techniques demandées.

A l'issue de cette consultation et, bien que le projet présenté par les entreprises Quillery se soit révélé le plus économique (Fig. 16), la Société des Auto-

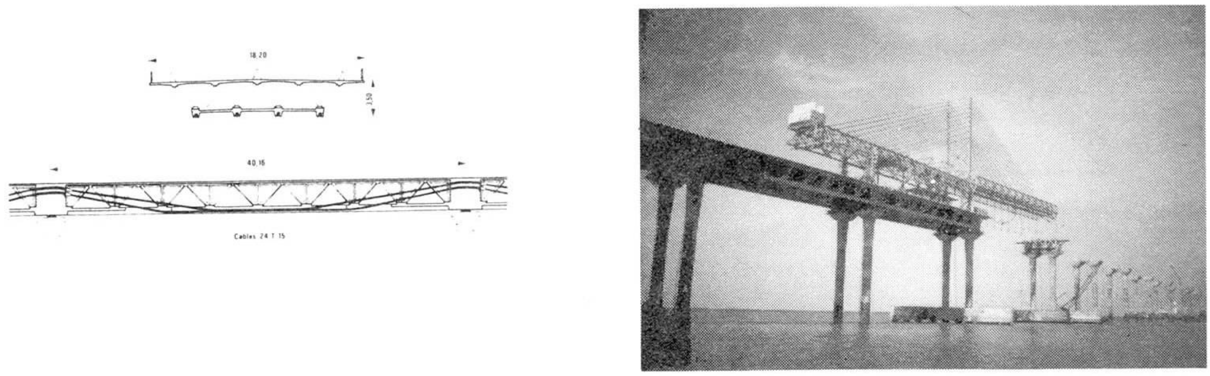


Fig. 13 Pont de Bubiyan (projet Bouygues)

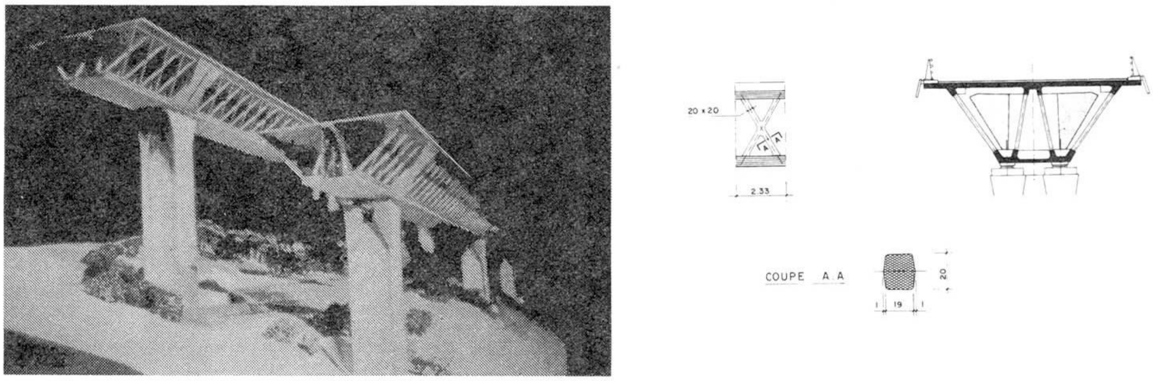


Fig. 14 Viaducs de Sylans et des Glacières (projet Bouygues)
Maquette et voussoirs type

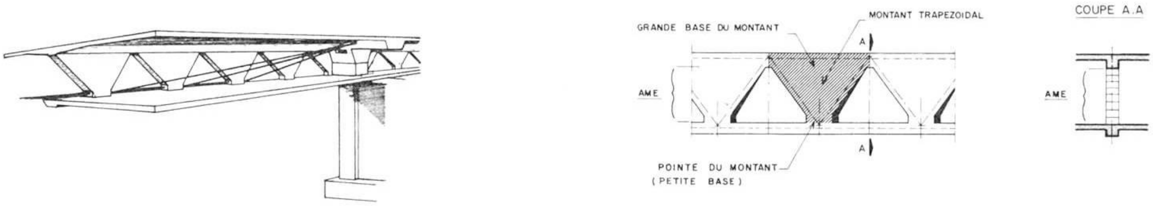


Fig. 15 Principe des âmes évidées à montants trapézoidaux

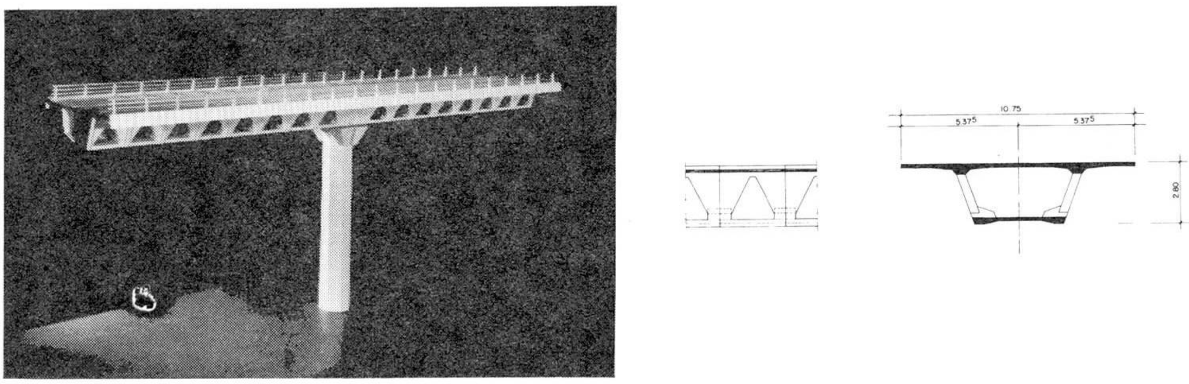


Fig. 16 Viaducs de Sylans et des Glacières (projet SECOA)
Maquette et voussoirs type

routes Paris-Rhin- Rhône a finalement choisi le projet de l'entreprise Bouygues, en raison de son caractère plus innovant.

4 - LA PRECONTRAINTE EXTERIEURE AU BETON

Une autre tendance marquante dans la conception des ponts en béton précontraint est le recours à des câbles de précontrainte extérieurs au béton.

Depuis quelques années, sous l'impulsion du SETRA, en la personne de Michel Virlogeux, et de plusieurs entreprises, **la précontrainte extérieure** a connu en France un développement important.

Cette technique présente de nombreux avantages liés à la facilité d'exécution et à la qualité de la structure finie, sans toutefois qu'il en résulte généralement une économie significative dans les constructions courantes de portées moyennes.

Mais son principal avantage réside dans la possibilité de remplacement éventuel des armatures corrodées ou rompues, moyennant une conception appropriée du câblage. Aussi nous semble-til souhaitable de prévoir systématiquement une précontrainte extérieure **démontable**, qui constitue une garantie supplémentaire pour la durabilité des ouvrages.

Les ouvrages à précontrainte totalement extérieure et intégralement remplaçable sont d'autre part particulièrement bien adaptés à l'utilisation d'une **précontrainte partielle**, la fissuration du béton tendu n'ayant aucune conséquence sur la conservation des aciers de précontrainte.

Les premières applications systématiques de la précontrainte extérieure ont eu lieu aux Etats-Unis avec les projets de "Figg and Muller", comme "Long Key Bridge" (Fig. 17).Il s'agissait d'ouvrages de portées moyennes, réalisés par **travées entières** entre appuis et constitués de voussoirs préfabriqués assemblés sur cintres. La précontrainte, totalement extérieure et filante sur chaque travée, est ancrée dans les entretoises sur pile. Les câbles sont déviés au droit de petits bossages en béton armé, situés à la jonction des âmes avec la membrure inférieure. Cette solution est simple car ces déviateurs, même s'ils sont en saillie par rapport à la section courante, sont de faible poids et d'exécution facile, contrairement aux nervures et diaphragmes utilisés dans d'autres projets.

Comme nous l'avons dit précédemment l'utilisation **d'âmes évidées à montants trapézoidaux** apporte dans ce cas une solution élégante et économique à la déviation des câbles de précontrainte extérieure.

La précontrainte extérieure a également permis d'obtenir un allègement des structures dans le domaine des **ponts poussés** où les âmes des sections tubulaires ont été réduites de façon sensible (2 âmes de 0,30 m pour un tablier de 10 m de largeur et de 50 m de portée).

Les ponts construits par **encorbellements successifs** ont enfin fourni un nouveau champ d'application à la précontrainte extérieure mais n'ont pas permis, jusqu'à ce jour, la réalisation d'ouvrages à précontrainte totalement extérieure et remplaçable.

Les projets les plus satisfaisants construits en France comportent généralement trois familles de câbles :

- des câbles de fléaux, semi-horizontaux, et intérieurs au béton, disposés au voisinage de la membrure supérieure du tablier et reprenant son poids propre en console.

- des câbles extérieurs au béton, mis en place après clavage des fléaux, filants d'une entretoise sur pile à l'autre et déviés à l'aide de dispositifs spéciaux.

- quelques câbles de continuité, horizontaux et intérieurs au béton, disposés au voisinage de la membrure inférieure du tablier dans la zone de clé des travées. Ces câbles complètent la précontrainte engendrée à la clé des travées par les câbles filants.

Ce type de câblage, appelé **câblage mixte**, présente l'inconvénient de comporter un certain nombre de câbles intérieurs au béton, dont le remplacement est impossible.

Il est alors préférable de substituer aux câbles de fléaux précédents des câbles extérieurs au béton disposés au- dessus de la dalle sous-chaussée et déviés à l'aplomb des piles par des voiles verticaux de faible hauteur. Nous avons donné à cette famille de câbles le nom de **précontrainte extradossée.**

Ces câbles sont différents de haubans car leur rôle essentiel est d'assurer une précontrainte horizontale du tablier et non de développer des réactions élastiques verticales. D'autre part, contrairement aux haubans, ils sont peu sollicités à la fatique (leur variation de tension sous charges d'exploitation demeurant faible) ce qui permet de les tendre à des valeurs proches de celles des armatures de précontrainte traditionnelles. Enfin, leur faible excentricité au droit des piles (de l'ordre de 1/15 de la portée) nécessite des déviateurs moins sollicités et de construction beaucoup plus facile que les mâts des ponts haubanés.

L'efficacité de ces câbles, qui équilibrent une fraction importante du poids propre, permet enfin de donner au tablier une **section constante**, ce qui facilite grandement son exécution.

La précontrainte extradossée est complétée par des câbles filants, mis en place après clavage des fléaux, déviés en travée et ancrés dans les entretoises sur piles. Tous les câbles constituant la précontrainte longitudinale de ce type d'ouvrage sont alors extérieurs au béton et remplaçables.

Nous avons développé cette idée pour la première fois à l'occasion du projet du pont de l'Arrêt Darré pour les Entreprises Quillery. Les deux idées exposées précédemment, les âmes évidées à montants trapézoidaux et la précontrainte extérieure extradossée, sont associées dans ce projet et ont conduit à une économie de matériaux de l'ordre de 30 % par rapport à la solution à deux caissons parallèles à deux âmes (Fig. 18).

Dans le cas de tabliers larges à deux sens de circulation séparés, la précontrainte extradossée est située dans l'axe de l'ouvrage comme au pont de l'Arrêt Darré. Dans le cas de tabliers de faible largeur, la précontrainte extradossée est disposée latéralement selon deux plans verticaux.

On peut également envisager d'enrober la nappe de câbles constituant la précontrainte extradossée dans un voile en béton (Fig. 19). Si le voile est exécuté après mise en tension des câbles, comme au pont du Ganter, en Suisse, son béton est tendu sous l'effet des charges d'exploitation et devient un élément passif qui alourdit inutilement la structure.Si le voile est bétonné au fur et à mesure de la mise en oeuvre des câbles et forme ainsi une épine dorsale qui complète la section résistante du tablier, on obtient un autre type de structure, appelé **"Fin-back bridge"**, dont la première réalisation est le pont de Barton Creek, aux Etats-Unis, à partir des idées du Professeur Lee.

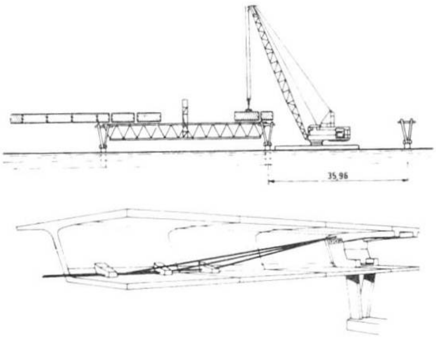
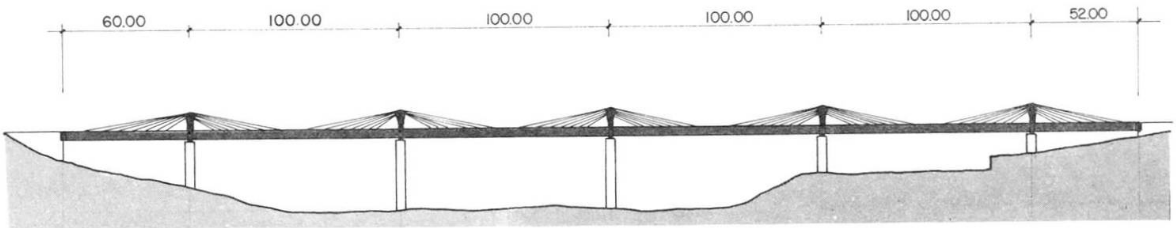


Fig. 17 Pont de Long Key
(projet Figg and Muller)

VIADUC de l'ARRET-DARRE
COUPE LONGITUDINALE 1/2000



VIADUC de l'ARRET-DARRE
COUPE TRANSVERSALE 1/100

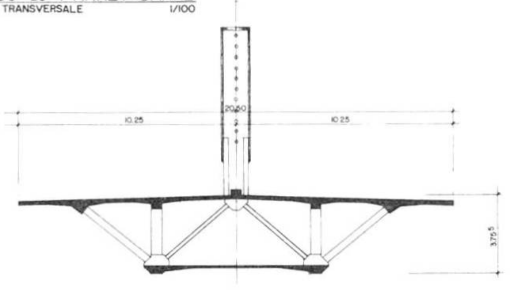
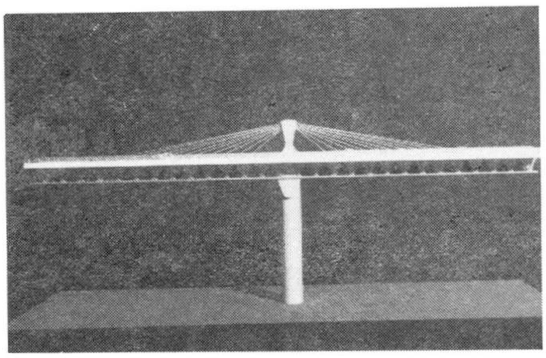
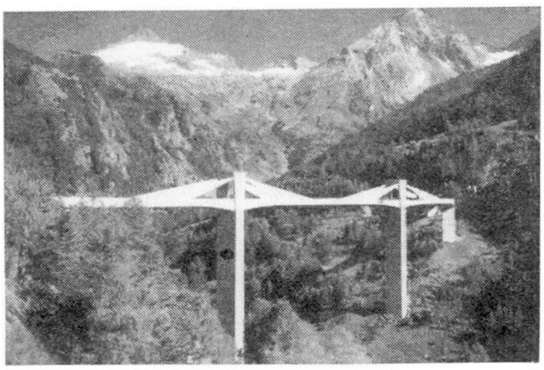


Fig. 18 Viaduc de l'Arrêt Darré
(projet SECOA)
Coupes longitudinale et transversale
Maquette





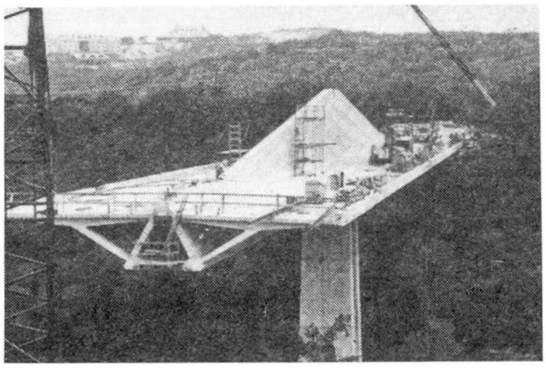


Fig. 19 Pont du Ganter et Pont de Barton Creek (Fin back bridge)

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SOUS-THÈME 4.1

Innovation in the Field of Materials

Innovation dans le domaine des matériaux

Neuerungen auf dem Gebiet von Baumaterialien

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Improvement in Quality of Concrete Structures by Two-Stage Mixing Method

Amélioration de la qualité des structures en béton par la méthode du mélange en deux étapes

Qualitätssteigerung bei Betonbauteilen durch zweistufiges Beton-Mischverfahren

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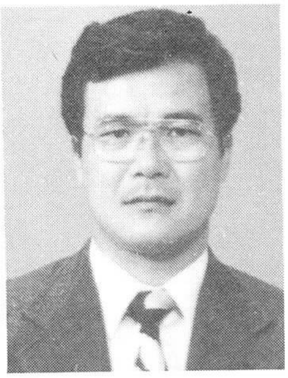

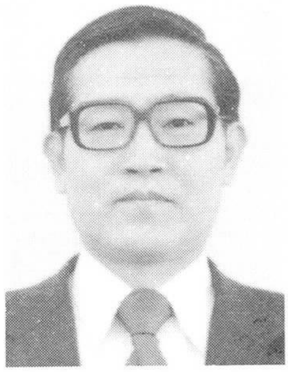

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Yasuro Ito, born in 1926, graduated from Nagano Technical High School, was mainly engaged in manufacturing concrete products for more than 30 years. He received his Dr. Eng. from the University of Tokyo in 1983.

Yasuhiro Yamamoto, born in 1935, graduated from the University of Tokyo in 1959, has been mainly involved in research and development of constructional materials. He received his Dr. Eng. from the University of Tokyo in 1973.

SUMMARY
A two-stage mixing method is proposed for manufacturing concrete in which the surface condition of aggregate is first improved by a process of coating with cement paste of a suitable quality, following which the remaining water is added and mixing is done once more. This paper deals with not only the improvement in quality of the concrete itself, but also the results of experiments on quality improvement of structures, cases of practical use, as well as the actual method of manufacturing.

RÉSUMÉ
Une méthode de mélange en deux étapes est proposée pour la fabrication du béton. L'état de surface de l'agrégat est d'abord amélioré par un procédé de revêtement à l'aide d'une pâte de ciment de qualité appropriée, puis le restant d'eau est ajouté et le mélange effectué de nouveau. L'amélioration dans la qualité du béton lui-même, les résultats des expériences sur l'amélioration des structures, des cas d'utilisation pratique et la méthode de fabrication sont expliqués.

ZUSAMMENFASSUNG
Ein zweistufiges Mischverfahren wird für die Betonherstellung vorgeschlagen. In einer ersten Mischphase werden die Zuschlagsstoffe benetzt und durch Hinzufügen des Zementes mit einer Zementpaste überzogen. Anschliessend wird das Restwasser zugefügt und erneut gemischt. Es wird mit diesem Mischverfahren nicht nur eine bessere Betonqualität erhalten. Auch der Bauteil erfährt eine Qualitätsteigerung, wie anhand von Versuchsergebnissen gezeigt wird. Das Mischverfahren sowie praktische Anwendungen werden besprochen.

1. TWO-STAGE MIXING METHOD AND OPTIMUM W_1/C RATIO

In manufacturing concrete, instead of introducing the materials simultaneously and mixing, it is conceivable to use a two-stage mixing method in which the materials are mixed divided into stages as shown in Fig. 1. This is a manufacturing technique in which primary water is first added to set up a suitable surface moisture content of aggregates, and primary mixing is performed together with cement, followed by introduction of the remaining secondary water for secondary mixing. The essential point of this method lies in coating the surface of aggregates, especially of fine aggregate of large total surface area, with cement paste of low water-cement ratio which is in a capillary state[1].

Fig. 2 shows the influence of the ratio by weight W_1/C of primary water and cement on the rate of bleeding of mortar indicated with the fine aggregate-cement ratio by weight S/C as the parameter. The bleeding ratio of mortar mixed in two stages with W_1/C made extremely low becomes higher compared with the conventional simultaneous mixing method. However, the bleeding ratio declines with increase in W_1/C and becomes extremely low compared with the case of conventional simultaneous mixing. And, when a certain value of W_1/C is exceeded, the bleeding ratio increases again. In this way, there exists an optimum W_1/C at which bleeding ratio becomes a minimum. Such a condition in which the bleeding ratio becomes a minimum is prominent with a rich mortar of low S/C. This condition is alleviated with a lean mortar of high S/C, while the optimum W_1/C is in a wide range, and moreover, the value of W_1/C itself becomes large as shown in Fig. 2. In this way, establishment of W_1/C, the ratio by weight of primary water to cement in primary mixing, is important when adopting the two-stage mixing method, and the quality of concrete differs greatly depending on the value set.

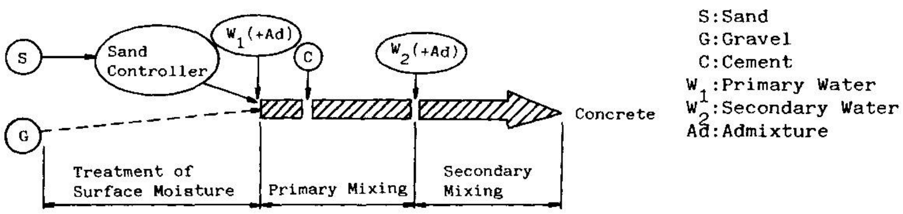

Fig. 1—Example of two-stage mixing method.

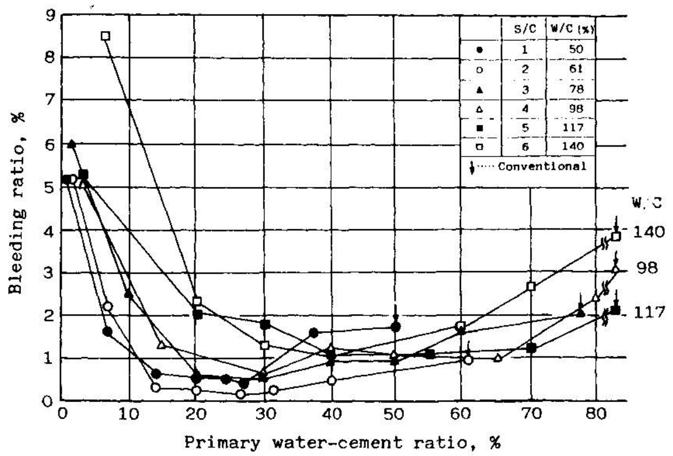

Fig. 2—Primary water-cement ratio and bleeding ratio of mortar.

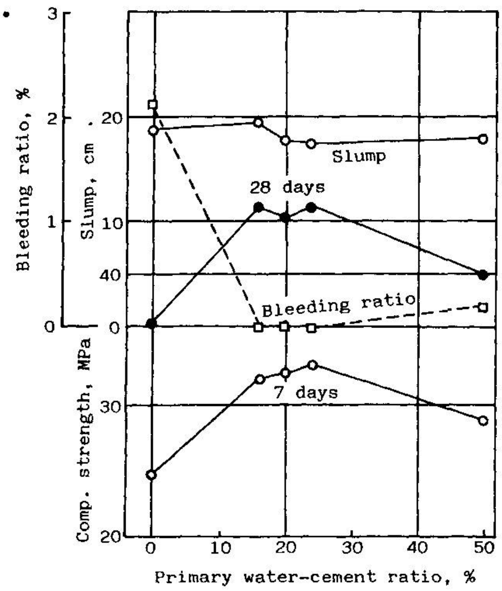

Fig. 3—Primary water-cement ratio and qualities of concrete.

2. QUALITY OF CONCRETE

The results of an experiment on concrete are shown in Fig. 3. This was a case of the final water-cement ratio W/C being 0.50 and slump 18 cm. Two-stage mixing was performed and especially in the range of W_1/C of 0.16 to 0.24, bleeding was reduced drastically to an extent that hardly any bleeding water could be detected. Practically no change was seen in slump even when two-stage mixing was performed.

As a result of testing the amount of dewatering when a pressure of 3.43 MPa was applied to investigate the pumpability of concrete, it was found that concrete made by two-stage mixing was 20 to 60 percent smaller in the amount of dewatering for both the initial and final stages of pressurizing. This trend was more prominent the lower the slump and the lower the water-cement ratio.

It is clear from Fig. 3 that compressive strength of concrete is increased by two-stage mixing. Fig. 4 shows cases of slump maintained constant at approximately 18 cm and with water-cement ratio varied between 0.30 and 0.60. When using the two-stage mixing method compressive strengths and splitting tensile strengths are 10 to 20 percent higher compared with concrete made by the conventional simultaneous introduction method.

3. IMPROVEMENT IN QUALITY OF STRUCTURE

It can hardly be said that concrete structures have always been entirely of uniform quality in the paste, qualities differing between upper and lower positions and parts of the structure, and depending on the conditions when executing work, Particularly, with walls and columns taller than 3 m, wet-consistency concrete of slump higher than 15 cm is often used, and with such members the strength of concrete and bond strength with reinforcing steel are lower at the upper parts and these become weak points of the structure.

Concretes made by the two-stage mixing method and by the conventional simultaneous mixing method were placed in reinforced concrete wall panels of 3-m height, 0.8-m width, and 15-cm thickness, and the compressive strengths of the concretes in the vertical directions of the panels and bond strength distributions of reinforcing bars were compared. The results are given in Fig. 5. The compressive

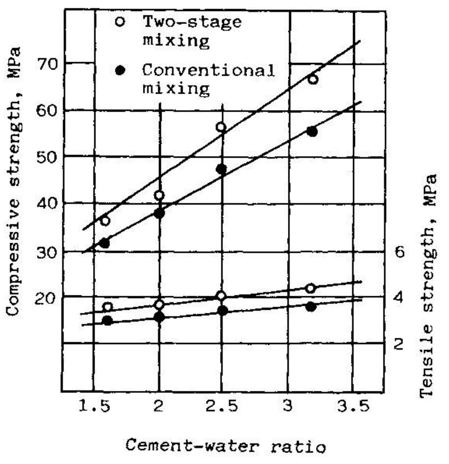

Fig. 4—Compressive and tensile strengths of concrete.

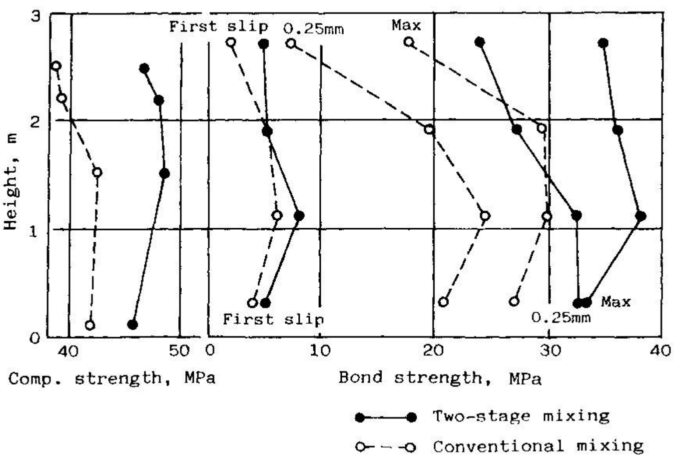

Fig. 5—Distribution of compressive strengths of cocrete and bond strengths of reinforcing steel in wall panels.

strengths and bond strengths according to core samples from various heights are shown. Concrete slump was 18 cm, and water-cement ratio 0.50.

Whereas bond strengths at a height of 2.7 m were 40 to 60 percent lower compared with the bottom parts of the wall panels when using concrete mixed by the conventional method, the decrease in case of the two-stage mixing method was limited to a maximum of 25 percent. Bond strengths per se were higher with the two-stage mixing, and the degree of increase was greater the higher the location in the wall panel. Although not as prominent as with bond strengths, the distributions of compressive strengths showed the effectiveness of two-stage mixing. That is, compressive strengths at various locations in the wall panels were increased 10 to 20 percent over the conventionally-mixed method, and strength reductions did not occur even at a height of 2.5 m.

Such an effect of the two-stage mixing method was confirmed with a reinforced concrete wall panel 8 m in height, 1 m in width, and 40 cm in thickness. In essence, compared with the bottom part of the wall panel, the reductions at a height of 7.5 m were held to 25 percent for bond strength of reinforcing steel and 15 percent for compressive strength, so that the strength reductions were smaller.

That it is possible to reduce variation in quality at various locations in a concrete structure in this way is because with the two-stage mixing method a concrete with extremely little segregation in the forms of bleeding and settling of aggregates is successfully made.

4. CASES OF PRACTICAL USE

Concrete made by the two-stage mixing method was used in large quantities of tunnels such as Seikan Tunnel[2]. Subsequently, it was also used in offshore concrete. Application to buildings[3] and dams[4] lagged behind slightly, but this concrete came to be adopted as the excellent uniformity and stability of quality and the good workability received high regard.

In particular, 1,900 m^3 of concrete by the two-stage mixing method was adopted in 1981 in the administration building annex project of Igata Nuclear Power Station of Shikoku Electric Power Co.. The specified concrete strength was 23.5 MPa, and compared with 610 m^3 of simultaneously-mixed con-

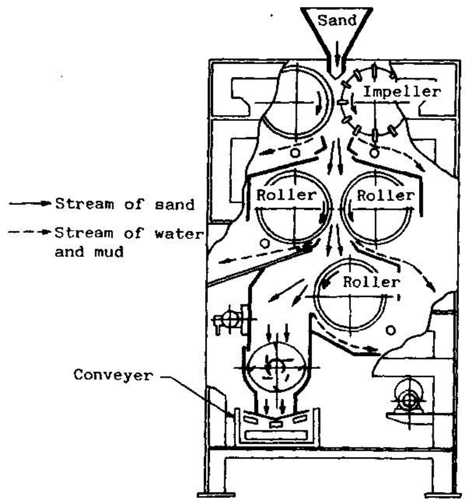

Fig. 6—Sand Controller.

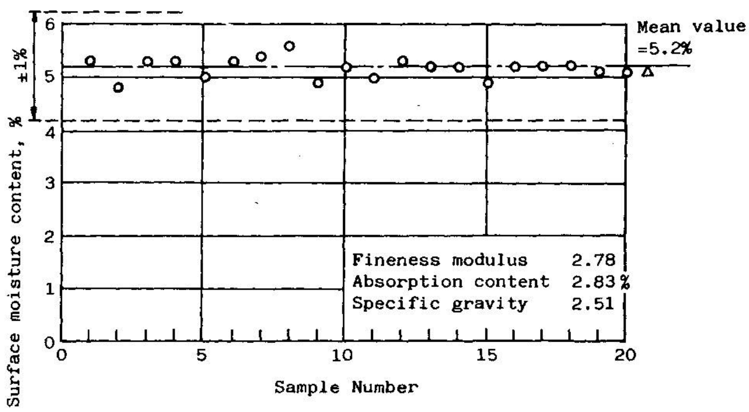

Fig. 7—Control chart of Surface moisture content of sand after adjustment by Sand Controller.

crete used in the same project, compressive strength was 32.2 MPa, 16 percent higher, and bleeding 0.14 cm^3/cm^2, 44 percent less, while the standard deviation in compressive strength of concrete during construction was small at 1.81 MPa, and favorable results were obtained in the aspects of quality and constructability.

The concrete mixer used in this project was a forced-mixing type mixer of capacity 1.5 m^3, while a Sand Controller of fine aggregate-processing capacity 20 m^3/h was newly installed. The Sand Controller, as shown in the diagram of its principles in Fig. 6, is a machine for adjusting fine aggregate to the required surface moisture content. The surface moisture content of pit sand after adjustment, as shown in Fig. 7, was held to a range of roughly 5 ± 0.5 percent.

In 1985, the two-stage mixing method was adopted throughout for concrete work of a total volume of 500,000 m^3 at Tomari Nuclear Power Station of Hokkaido Electric Power Co., and at present, approximately 60 percent of placement of this concrete has been completed. Forced-mixing type mixers, one of 2-m^3 capacity and another of 3-m^3 capacity, and three Sand Controllers, each of 40-m^3/h capacity, are being used for the project. Use of concrete made by the two-stage mixing method is also contemplated for other nuclear power stations to be newly constructed.

5. METHOD OF MANUFACTURING CONCRETE

Concrete by the two-stage mixing method, as shown in Fig. 1, is manufactured by the processes of surface moisture adjustment of aggregates, primary mixing, and secondary mixing. For this purpose, apparatus for adjusting the surface moisture of aggregates, especially fine aggregate which is of large total surface area and a mixer capable of primary mixing of mixes of low water-cement ratio are required. It is amply possible for this primary mixing to be done with an ordinary forced-mixing type mixer.

With only one mixer, however, mixing time will be longer by just the amount of time required for primary mixing, and in an actual concrete plant, efficiency will be lowered to result in poor economy. Hence, two-stage mixers are used for the purpose of shortening mixing time. With the two-stage mixers, as shown in the flow chart of Fig. 8, primary mixing of fine aggregate, cement, and primary mixing water is done using the upper stage mixer to make mortar, while at the

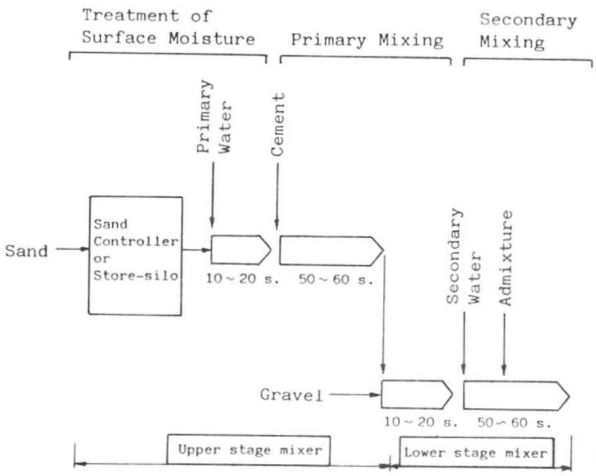

Fig. 8—Flow chart of two-stage mixing method.

Photo. 1—Tomari Nuclear Power Station.

lower stage mixer, the mortar and coarse aggregate are introduced to coat the coarse aggregate surfaces with cement paste of a capillary state, following which secondary mixing water is added to perform secondary mixing. Since primary mixing and secondary mixing are performed simultaneously with this arrangement, mixing time is approximately halved compared with the case of using a single mixer. It is a trend recently for the number of concrete plants equipped with two-stage mixers to increase due to the larger mixing capacity and more stable quality obtained even when the two-stage mixing method is not adopted.

The Sand Controller, as shown in Fig. 6, has been developed for adjusting surface moisture of fine aggregate and is in general use. Recently, in addition, a procedure has begun to be adopted where numerous aggregate hoppers made of concrete are provided and fine aggregate sprinkled with water beforehand is put in these hoppers and adjusted to a thoroughly moist condition for use then in sequence.

ACKNOWLEDGEMENTS

The authors received invaluable advice and guidance from Professor Yoshiro Higuchi, Science University of Tokyo, and Professors Koichi Kishitani and Hajime Okamura, University of Tokyo, in carrying out the study. A tremendous amount of cooperation was also received from the Technical Research Institute of Taisei Corporation. The sincerest gratitude is hereby extended.

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A New Concrete Production Method Using Small Pieces of Ice

Nouvelle méthode de confection de béton par utilisation de petits morceaux de glace

Ein neues Betonherstellungsverfahren unter Verwendung von Eisstücken

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SUMMARY
A new unique method has been developed for the production of concrete. This new method takes advantage of small pieces of ice and their melting process. The concrete is obtained by substituting small pieces of ice for the mixing water. Using this method, the various properties required of concrete during production are greatly improved.

RÉSUMÉ
Une nouvelle méthode unique a été mise au point pour la confection de béton. La nouvelle méthode tire avantage de petits morceaux de glace et de leur processus de fonte. Le béton est obtenu en substituant les petits morceaux de glace à l'eau de mélange. Les différentes propriétés de béton requises durant la confection sont considérablement améliorées.

ZUSAMMENFASSUNG
Es wurde eine neue Methode zur Betonherstellung entwickelt, welche den Schmelzprozess kleiner Eisstücke ausnützt. Bei der Betonherstellung wird das Mischwasser durch Eisstücke ersetzt. Die verschiedenen Eigenschaften des Betons während der Herstellung werden durch diese Methode wesentlich verbessert.

1. INTRODUCTION

This paper introduces the unique concrete production method based on a new concept. The new concept means that small pieces of ice and their melting process are effectively utilized for production of concrete. The concrete is obtained by substituting small ice pieces for the mixing water. The various required properties of concrete during production, including mixing efficiency, placement performance, consolidation ability and curing stability are greatly improved.

The concept of using small pieces of ice and fundamental characteristics of the concrete produced by this method are described in the previous papers [3] and [4], in detail. In the papers, the essential differences between this proposed method and former techniques using ice pieces described in [1] and [2] are also discussed.

This method of producing concrete was originally conceived by T. SUZUKI, primary author, and various experiments and examinations were undertaken jointly by T. SUZUKI and K. TAKIGUCHI.

2. METHOD OF PRODUCING CONCRETE USING SMALL ICE PIECES

The proposed concrete production method is distinguished by using small ice pieces substituted for the mixing water at the start of mixing. The small ice pieces should be perfectly melted at the finish of placing. The characteristics of this concrete production method are shown in Chart 1.

The advantages of this method are as follows.

(1) Mixing can be conveniently carried out almost irrespective of the mix proportions.

The difference between solid-liquid phase mixing and solid-solid phase mixing is shown in Photos. 1 and 2. These two photographs indicate the sections of wheat

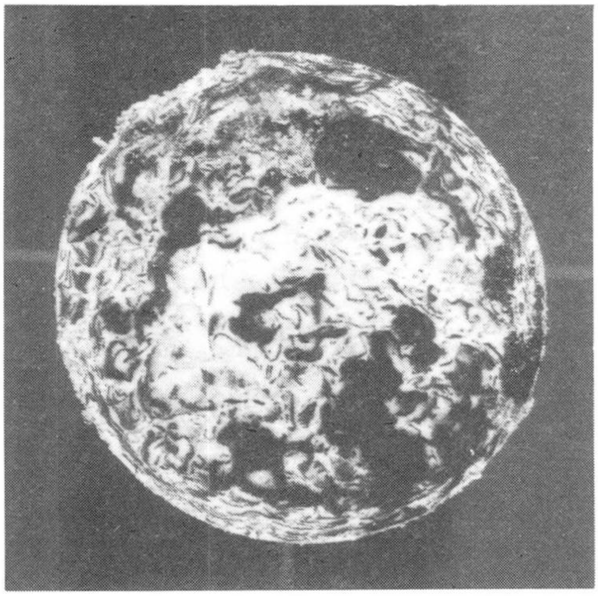


Photo.1 Section of wheat flour and red ink (liquid phase) mixture

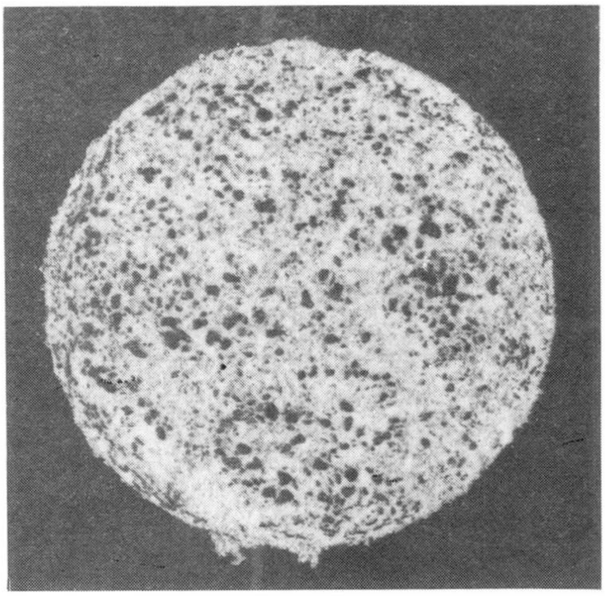


Photo.2 Section of wheat flour and red ink (frozen and sliced) mixture

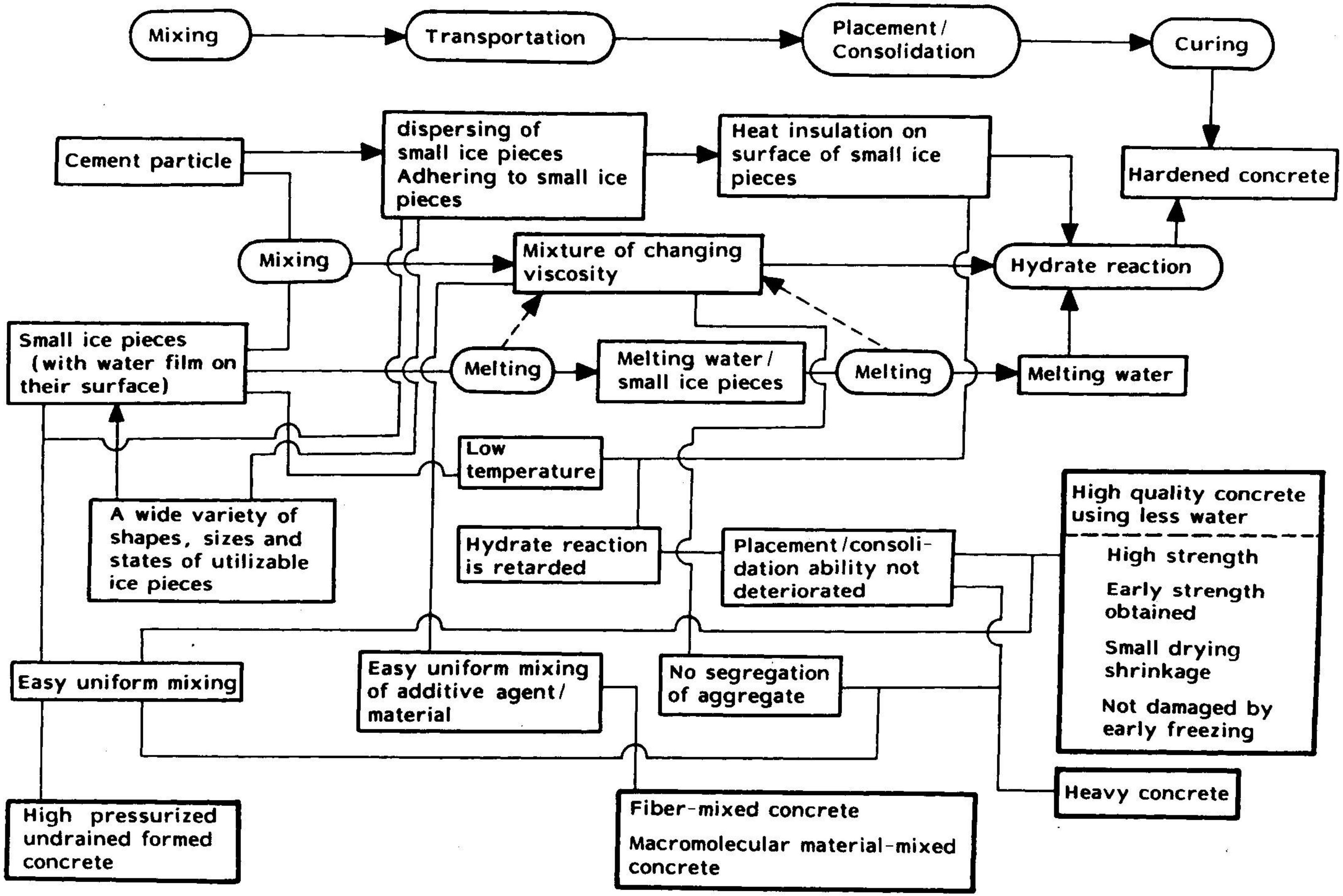


Chart 1 Characteristics of concrete production method utilizing small ice pieces and then melting process

flour and red ink mixture. Red ink in the liquid phase was used in the mixture shown in Photo.1. Red ink was frozen and flaked with an ice slicer for the mixture shown in Photo.2. All the conditions except the phase of red ink at the mixing were the same.

(2) Small ice pieces of a wide variety of shapes, sizes and states can be used.

Because the minute particles of cement have the effect of dispersing the pieces of ice, they can be separated and dispersed uniformly. This can be attained without hindrance even when macroscopic water film forms on the surface of the pieces of ice, or even when they are joined in a chain form.

Crushed ice of maximum particle size 3~5 mm, sliced ice of 1~2 mm, natural snow and very small ice pieces can be used.

(3) The viscosity changes as the small ice pieces melt, reaching an appropriate level for the uniform mixing of special additives, such as fibers.

(4) There is no aggregate segregation after uniform mixing because of the change in viscosity caused by the melting of the ice.

(5) The hydrate reaction is retarded throughout mixing until final placement.

The above characteristics are advantageous for production of the following types of concrete:
i) High quality concrete using less water
ii) Concrete with heavy aggregates
iii) Concrete mixed with fibers
iv) Concrete mixed with macromolecular materials
v) Pressured and undrained formed concrete
vi) Slow setting ready-mixed concrete

3. PROPERTIES OF CONCRETE OBTAINED USING THIS METHOD

The compressive strength of concrete produced by this method was examined comparing with that of conventional concrete. Two types of concrete compared were produced under the same conditions except the phase of mixing water. The

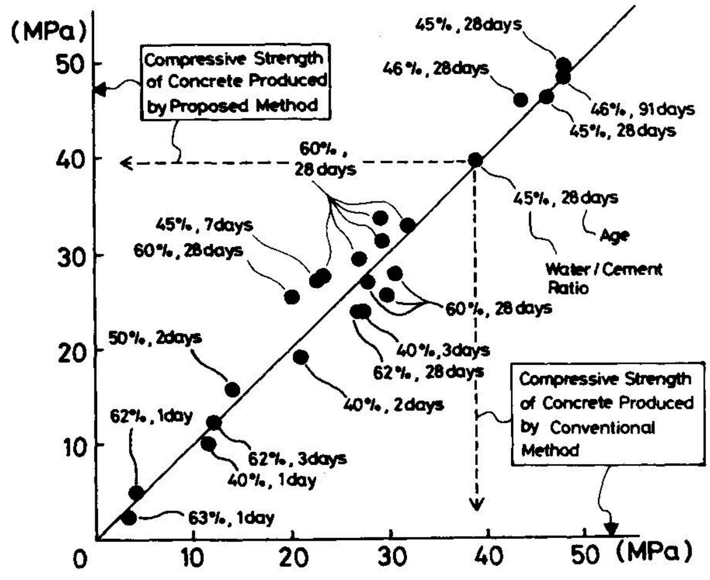


Fig.1
Compressive strength of the concrete produced by proposed method and of conventional concrete

curing period ranged from 1 day to 91 days. The experimental results are shown in Fig. 1. It can be concluded that the strength of the concrete produced by proposed method is equal to that of the conventional concrete.

The slump values of the concrete produced by this method are larger than those of conventional concrete as shown in Figs. 2 and 3. The mix proportion of the concrete shown in Fig. 2 is: Cement 264 kg/m^3, Water (Small ice pieces) 161 kg/m^3, Sand 851 kg/m^3, Gravel 1016 kg/m^3, Air entraining agent 2.8 kg/m^3. As for Fig. 3; Cement 305 kg/m^3, Water (Small ice pieces) 180 kg/m^3, Sand 777 kg/m^3, Gravel 992 kg/m^3, Air entraining agent 3.4 kg/m^3.

As shown in Figs. 2 and 3, the slump value of the concrete produced with small ice pieces became larger as time passed, though the ice pieces were perfectly melted when the concrete was mixed up. This is one of the distinctive properties of the concrete with small ice pieces.

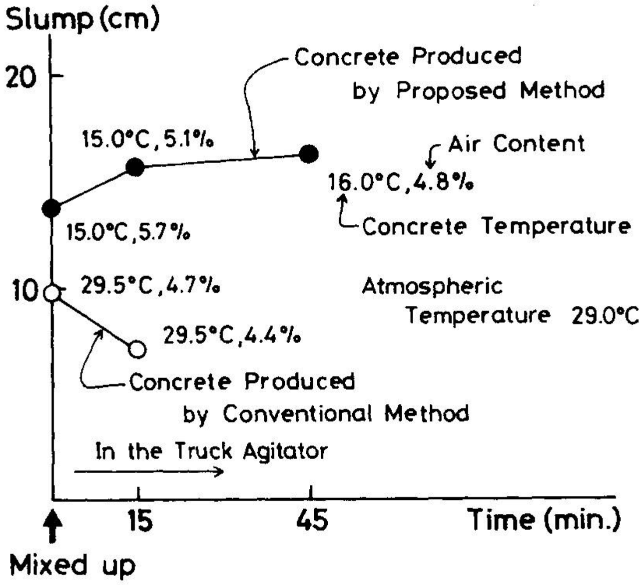


Fig.2 Slump of the concrete produced by proposed method and of conventional concrete

4. PRACTICAL USE AND ECONOMICAL ASSESSMENT

In several actual structures, the concrete produced by this method was practically used without any problem. The atmospheric temperatures at practical using of this method were about 30°C (hot weather), 20° C (mild weather), -5° C (cold weather) and so on.

The transportable plant suppying small ice pieces was designed as shown in Fig. 4, and the first experimental truck shown in Photo. 3 was made. The plant truck is working successfully according to the specifications shown in Fig. 4.

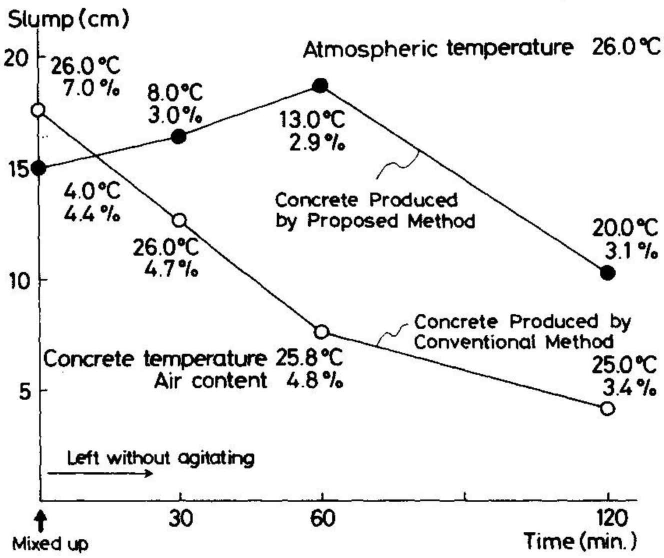
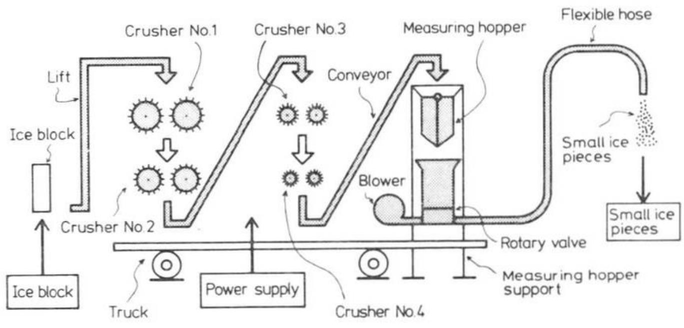


Fig.3 Slump of the concrete produced by proposed method and of conventional concrete

The cost of this proposed concrete depends on the price of ice and will be 3~15% higher than that of the conventional concrete. A rise in price by using small ice pieces is not so significant, because it should be evaluated together with the improved properties. To product high quality concrete using this method shall be economical in the final analysis.



Specification	Maximum particle size of ice pieces	Continuously supplying capacity	Allowable measuring error
	3 ∿ 5 mm	180 ∿ 220 kg/min	1 kg/ 180 kg

Fig.4
Transportable plant system supplying small ice pieces and the specifications of the plant

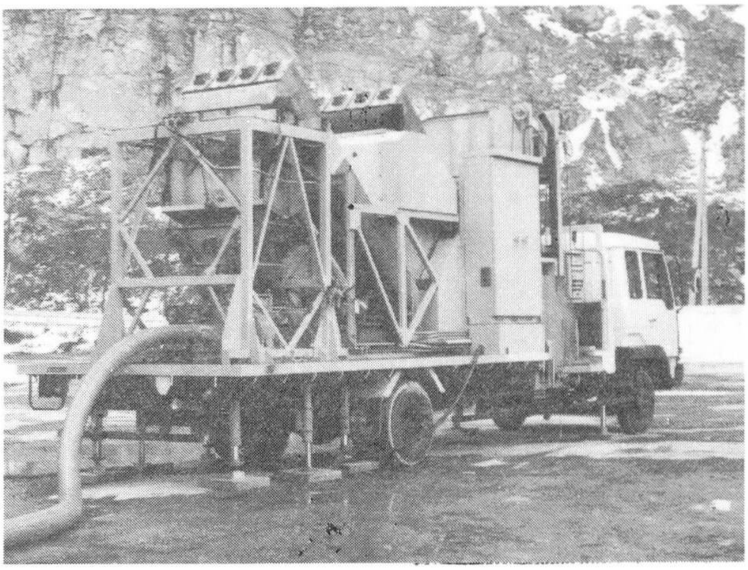


Photo.3
The first experimental plant truck

5. CONCLUSION

The various problems faced in the production process of concrete can be solved by the proposed method using small ice pieces instead of mixing water.

The concept of utilizing small ice pieces and their melting process has opened up new possibilities for the production of various types of concrete.

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Improvement of Surface Quality of Concrete Structures by Unique Formwork

Meilleure qualité de surface du béton avec une nouvelle méthode de coffrage

Verbesserung der Oberflächenbeschaffenheit von Betonbauteilen mit einer neuartigen Schalung

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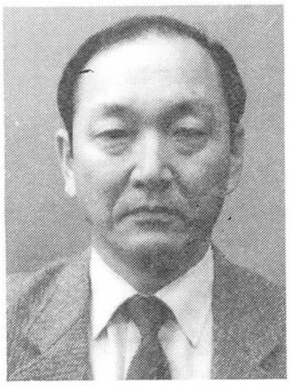

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SUMMARY
This paper outlines a unique method of permeable formwork developed for the purpose of improving surface quality of concrete structures. The major contents of the paper relate to the materials used for the permeable form, the composition of the form, the mechanism of bleeding excess water from the form, the test data of improvement of surface quality and durability, the economy of the method and several construction examples.

RÉSUMÉ
Cette contribution présente une nouvelle méthode de coffrage permettant d'améliorer la qualité des surfaces des ouvrages de béton. Elle traite des matériaux utilisés pour la réalisation des coffrages, de la composition des coffrages, du mécanisme de drainage de l'excès d'eau à partir du coffrage, des données d'essai concer nant l'amélioration grâce à l'utilisation des coffrages perméables pour les surfaces des ouvrages en béton et pour leur résistance . Elle présente aussi l'économie que cette méthode de réalisation apporte ainsi que quelques exemples de construction.

ZUSAMMENFASSUNG
Der Beitrag behandelt in grossen Zügen einen neuen Schalungtyp, welcher hinsichtlich eine Verbesserung der Betonoberfläche entwickelt wurde. Es werden vor allem die verwendeten Materialien, welche für diesen durchlässigen Schalungstyp verwendet werden, behandelt, wie auch der Schalungsaufbau und das Entwässerungssystem in der Schalung für die Ableitung des überschüssigen Wassers. Einige Versuchsergebnisse, die Wirtschaftlichkeit dieser Schalungsart und einige Ausführungsbeispiele werden besprochen.

1. INTRODUCTION

In Japan, four years ago, major mass communication reported the fact that many kinds of reinforced concrete structures began to deteriorate beyond expectations. Since then the general public take a growing interest in durability of concrete.

Now that the authors and others notice the feature that many of deterioration phenomena of reinforced concrete structures are prone to happen on the surfaces of the structures, we could conduct researches in improving surface quality of concrete structures, resulting in developing new form method entitled "Textile Form Method".

The principle of the method is that using a unique permeable form, immediately after fresh concrete is placed in the form, excess water is bled naturally out of it, thereby producing concrete having higher density toward the surface with smooth and beautiful one.

The conception which eliminates excess water from fresh concrete on purpose to improve the quality of concrete is not novel. Considerably previous to the development of this method, three following systems have been developed with similar view.

a) Using the form on which the sheet having considerable absorption has stuck;
b) Using airtight mat with vacuum pump;
c) Adding mechanical pressure to fresh concrete in a form;

Though the Textile Form Method is not above the said methods b) and c) in drainage performance, the Textile Form Method gains the following advantages being as good as the aforesaid methods a), b) and c) :-

(1) the effect of improving the quality of concrete surface is practically too much;
(2) this method is simply applicable to in-situ concrete work due to no need of any supplementary equipments, which are used for forcing a drainage.
(3) this method is not influenced by weather like a); and
(4) permeable form developed is economical because of the possibility of use from five to ten times.

Therefore, putting the Textile Form Method to practical use in 1985, it is adopted to a wide range of concrete form works.

2. MATERIALS AND COMPOSITION OF PERMEABLE FORM

2.1 Basal conditions on permeable form

The permeable form used for improving the quality of concrete surface needs to meet the following basal conditions :-

(1) the form has high aeration efficiency and permeability;
(2) the form scarcely allows cement particles to pass through;
(3) there gains smooth and beautiful concrete surface.

Moreover, for the purpose of practical use, the following economical conditions are added :-

(4) material cost and manufacturing cost to compose the form are reasonably low;
(5) the form can be used repeatedly and frequently.

2.2 Materials to compose permeable form

2.2.1 Double woven cloth for filtration and drainage

The authors and others developed the newly conceptual double woven cloth

Fiber material	Polyester & Polypropylene
Textile	Double woven cloth
Thickness	0.74 mm
Weight	440 g/m^2
Coefficient of permeability	9.5×10^{-3} cm/sec
Tensile strength	Lengthwise : 303 kg/3cm width Breadthwise: 335 kg/3cm width

Table 1 Character of double woven cloth

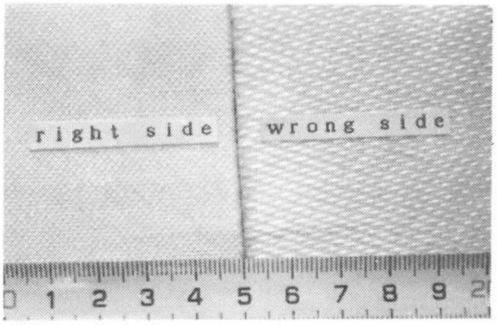

Fig. 1 Texture of double woven cloth

suitable for the permeable form as a filter material.

The character of developed cloth is shown in Table 1 and the textures of the cloth is shown in Fig.1. There is the difference between the texture of the right side and that of the wrong side of the cloth, and the right side of the cloth is used to be the concrete placing side, and the wrong side the side of form panel. The difference of both sides of the cloth implies that each of requested function also differs. Namely as for the right side, firstly the function of the filter which is pervious to water and air, but hardly to cement particles is requested, and secondly the cloth has simply to be removed from settled concrete surface. For these two reasons the closed weave is adopted to the right side of the cloth. But as for the wrong side, the open weave is adopted since fine space is maintained between panels, in order to allow water and air to be passed through.

Besides, as fiber used for this cloth, synthetic fiber such as polyester and polypropylene, which have strong chemical resistances, high strength, modulus of elasticity, small suction and which is low cost, are suitable.

2.2.2 Form panel with numerous tiny holes

To maintain air and water passed through laid cloth in the space between the cloth and the form panel is undesirable for two reasons. One reason is that in order to maintain a sufficient space in which such excess water and air are being kept after passing through the laid cloth, either much thicker cloth may be necessary, probably more than 3 mm in thickness, or some porous and flat material are required to be put into the space between the cloth and the form panel, and this results reasonably in the raise of form cost. Another reason is that internal water pressure is requested as low as possible as to the form side in order to move excess water in fresh concrete to form side.

In the developed permeable form, therefore, the form panel has many fine drilled holes or punched holes as the treatment of bleeding the air and water out of the form smoothly after passing through the cloth.
According to numerous concrete placing test results, the following values concerning these hole diameters and intervals are recommended :-

a) hole diameter is more than 3mm
b) hole interval is less than 100mm

2.3 Composition of permeable form

Appearance and sectional detail of an example of developed permeable forms are shown in Figures 2 and 3. This is that a plywood is used as a form panel, but in actual work the panel such as steel,

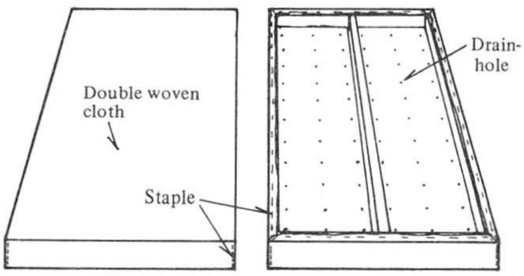

Fig. 2 Permeable form

aluminum and plastic is used instead of it.

Main considerations of production of permeable form are listed as follows :-

(1) heat cutter is used to cut the cloth on a plate of glass
(2) where the cloth is fixed on form panel with numerous tiny holes, the work is done while giving tension to the cloth to some extent so as not to wrinkle.
(3) Fixating the cloth to the panel on the portion that the cloth is folded back in the surroundings of the panel. In the case of wooden panel, stapler is used, and in case of metal panel, adhesive agent is used to set it. And besides, attention should be paid to the fact that if adhesive agent is applied to the cloth of the portion adjacent to placed concrete, the permeability becomes remarkably low.

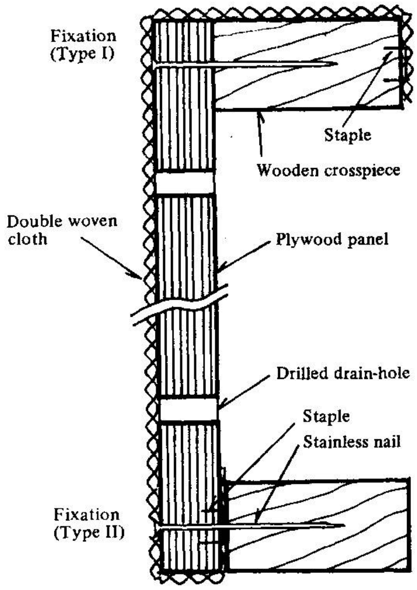

Fig. 3 Cross section of permeable form

3. MECHANISM OF BLEEDING EXCESS WATER OUT FROM PERMEABLE FORM

When concrete is placed in fabricated forms, high pore water pressure comes about in the concrete in proportion to its depth. When conventional watertight form is used, pore water pressure in a horizontal plane having a voluntary depth is fixed in any parts, thereby the lateral movement of pore water does not occur. But when the permeable form is used, the more the pore water pressure comes near the form, the more the pressure lowers due to the natural drainage from the form. As a result, pore water in the concrete moves from the high pressure inside, which is far off from the form, to a low pressure form side. Movable water in the concrete in the latter case may be thought an unnecessary excess water against long-term hydration.

The movement and drainage of the very excess water do play the most important role in improving the quality of concrete surface using permeable form.

4. FUNCTION OF PERMEABLE FORM

4.1 Permeability

The relation between water volume, which is bled from the same kind of permeable form used for form work of five different types of concrete structures, and the time elapsed is shown in Fig.4. Depending on the type of structures, the difference in drainage speed is recognized, but the difference in final water displacement per unit area lessens.

Ordinary Portland cement is used for all of these structures, while it is reported that a water displacement became more than 5l/m2 in massive concrete structures when later setting time cement was used [1].

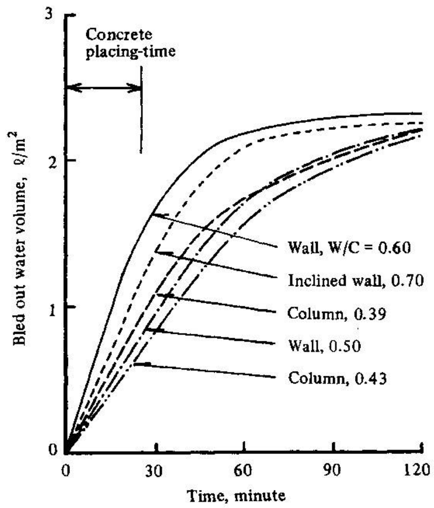

Fig. 4 Water volume bled out from permeable form

4.2 Improvement of surface quality of concrete structures

4.2.1 Appearance of concrete surfaces

Compared with concrete surface used conventional plywood form, where permeable form was used, the happenings of air bubbles and blow holes have remarkably been reduced and the concrete surface becomes smooth, evenly colored and beautiful due to natural bleeding of excess water and entrapped air in fresh concrete from the form. Especially in the surface of inclined concrete structures, such effect is highly noticeable (Fig.5).

4.2.2 Concrete surface strength

Schmidt hammer test result of wall and column, which are constructed by both permeable form and plywood form using the same concrete, is shown in Fig.6. Surface strength of concrete improves remarkably by using permeable form. And it is proven that the surface concrete from which excess water has been bled clearly high strength development as potential.

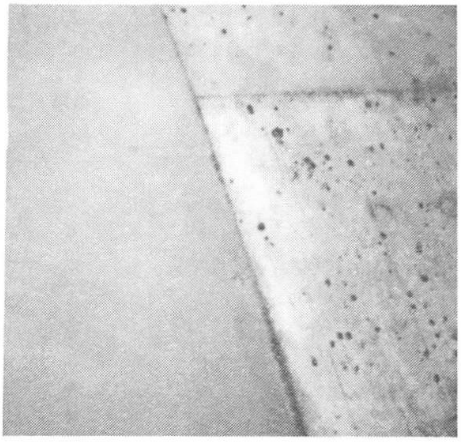

Fig. 5 Slope concrete surface of a dome constructed by permeable form (left) and plywood form (right)

4.3 Improvement of durable quality of concrete structure

Generally it is difficult for the peculiar durability of concrete structure to be valued absolutely, since it is influenced by concrete-making materials, construction accuracy, use conditions of structure, environmental conditions and so forth.

The authors and others announced the comparative durability test result of concrete cores, which are taken from simulated members of two massive walls using the developed permeable form and conventional plywood form at Annual Meeting of AIJ last year[1]. According to it, test items consist of accelerated carbonation test, freezing and thawing resistance test, salt penetration test and permeability test and so on. Putting all of these test results together, it confirmed that in case of concrete using permeable form the durable quality improves more than 50 percent as to the valuation of durable quality, as compared with that of conventional plywood form.

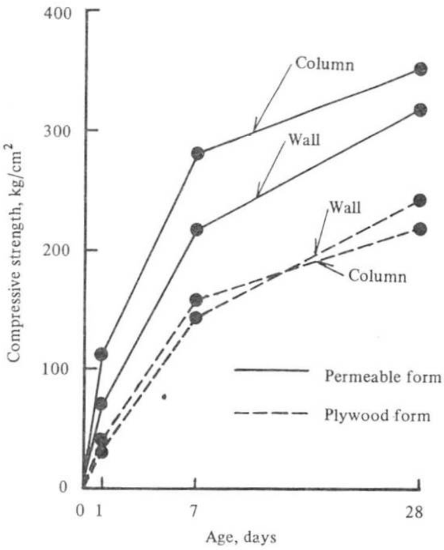

Fig. 6 Comparison of is Schmidt hammer test results

Apart from these tests, comparative accelerated carbonation test result of concrete cores, which are taken from simulated members of column is shown in Fig.7. Even in this test result the effect of high improvement of concrete quality was also demonstrated.

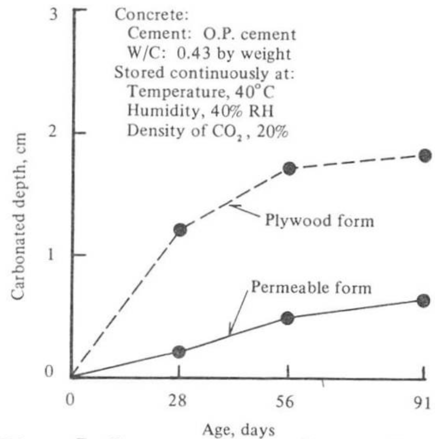

Fig. 7 Comparison of accelerated carbonation test results

5. VALUATION ON THE ECONOMICAL ASPECTS

Initial cost of the developed permeable form is about 4.5 times conventional plywood form in Japan. The breakdown of initial cost is that laying cloth is about 45 percent, panel 25 percent, drilling cost 11 percent and installation cost of cloth approximately 22 percent. Advancing the standardization and mass production in the future, there is a prospect of cutting the cost by about 3.5 times that of plywood form.

But comparison in itself of conventional form with this one having essentially special function in an aspect of initial cost is a problem.

Reasonable valuation of economic value of permeable form differs in that its usage, namely to what extent the function peculiar to this form which can improve many kinds of the qualities of concrete is able to make the most of as to an actual structure.

6. APPLICATION OF PERMEABLE FORM METHOD

6.1 Construction examples

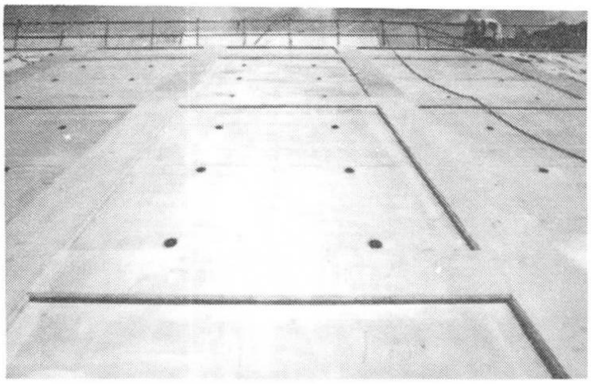

Fig. 8 Inclined retaining wall of Aseishigawa dam (Aomori pref.) [2]

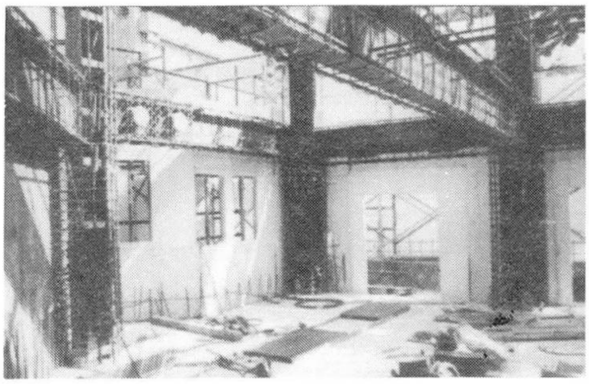

Fig. 9 External wall formwork of Ichikawa Heights bldg. (Chiba pref.)

6.2 Range of application

There are 135 construction works to which the permeable form method has been applied during the past two years.

This method has an easy application to an extensive concrete works because of its very simplicity. But the method is thought advantageous in particular to the following concrete structures considering the said performances of works:-

a) structures having slopes such as dam, retaining wall, pier, roof, and the like;
b) structure requiring high durabilities such as nuclear power plant building facilities, military facilities, huge buildings, and the like;
c) marine structures such as breakwater, sea wall, bridge, waterway, marine facilities and the like;
d) precasted concrete members having complicated shapes such as blocks used for weakening wave strength, tetrapods and the like.

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Systematische Bewertung von Modernen Betonbaumaschinen

Systematic Evaluation of Modern Concrete Construction Plants

Évaluation systématique des bétonnières modernes

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ZUSAMMENFASSUNG

Heute zeigen sich durch den falschen Einsatz von Betonbaumaschinen Bauschäden an Betonbauwerken, die durch dieselben mitverursacht werden. Es werden Einflußgrößen, Bewertungsparameter, Überlegungen zur Qualitätssicherung und Bewertungsmöglichkeiten behandelt. Durch verstärkte Mechanisierung und Automatisierung auf den Betonbaustellen sowie die Anwendung neuer Verfahrenstechniken wird eine systematische Bewertung von Betonbaumaschinen erforderlich.

SUMMARY

The damage to concrete building structures that emerges today is partly caused by improper use of concrete construction plants. Influence coefficients, evaluation parameters, possibilities for effective quality control and evaluation methods are considered. Increased mechanization and automation on concrete building sites and application of new processing techniques necessitate a systematic evaluation of concrete construction plants.

RÉSUMÉ

Il apparait actuellement dans des ouvrages en béton des dommages résultant de fautes commises lors de l'utilisation des bétonnières. L'étude traite des paramètres d'influence et d'évaluation ainsi que de l'assurance de la qualité. En raison du renforcement de la mécanisation et de l'automatisation sur les chantiers où le béton est produit et utilisé et du fait de l'adoption de procédés nouveaux, il devient indispensable de procéder à une évaluation systématique des bétonnières modernes.

1. EINLEITUNG UND ÜBERBLICK

Der Arbeitsprozeß Bereitung, Fördern und Einbau des Betons wird mit Hilfe der unterschiedlichsten Betonbaumaschinen durchgeführt. Die Frischbetonqualität kann bei diesem Ablauf durch die verfahrenstechnischen Stufen negativ beeinlußt werden. Heute zeigen sich bereits Bauschäden an Betonbauwerken, die durch falschen Einsatz von Betonbaumaschinen mitverursacht worden sind.

In der Vergangenheit sind bei den Betreibern derartiger Maschinen die Anforderungen bezüglich der Frischbetonqualität gestiegen. Für die qualitätssichere Abgabe des Betons aus den Betonbaumaschinen fehlen entsprechende Richtlinien und Normen. Durch die verstärkte Mechanisierung und Automatisierung der Betonbaustellen wird eine systematische Bewertung dieser Baumaschinen notwendig.

2. VERFAHRENSSTUFEN IM MASCHINELLEN BETONBAU

Die Leistungsfähigkeit und Wirtschaftlichkeit einer Betonbaustelle sind im wesentlichen von der Rationalisierung der Verfahrensstufen Bereitung, Förderung und Einbau des Frischbetons abhängig.

Der wirtschaftliche Baubetrieb fordert die optimale Ausnutzung der eingesetzten Betonbaumaschinen. Trotz der großen Bedeutung und des scheinbar hohen Entwicklungsstandes ist der qualitätssichere Einsatz von
- Betonmischern,
- Fahrmischern und
- Betonpumpen

teilweise mit Unsicherheiten behaftet.

Aus Gründen einer scheinbar besseren Qualitätskontrolle wird der Beton häufig in Betonwerken als werksgemischter Beton hergestellt. In Abhängigkeit vom Durchsatz, der Entfernung und den örtlichen Gegebenheiten wird der Beton mit den genannten Maschinen gefördert, transportiert und in die Schalung eingebaut.

In manchen Ländern werden die Feststoffkomponenten als Trockenmischgut in die Fahrmischertrommel und das Wasser in einen separaten Tank geladen und vor Ort gemischt. Um das unkontrollierte Ansteifen des Frischbetons während der Fahrt zu vermeiden, besteht besonders in heißen Klimazonen und bei größeren Transportentfernungen bzw. bei unkalkulierbaren Transportzeiten in gemäßigten Klimazonen der Bedarf an fahrzeuggemischtem Beton.

Die Diskussion, ob Werks- oder Fahrzeugmischung, ist hinsichtlich der Anschaffungskosten und der Betriebskosten verschiedentlich ohne Ergebnis geführt worden. Einige Normen lassen auch eine Kombination dieser beiden Verfahren (Shrink mixing) zu.

Die Förderung und der Einbau des Frischbetons auf der Baustelle erfolgen in der Regel mit Betonpumpen oder Baukränen, wobei die Betonpumpe in der Zukunft weiter an Bedeutung gewinnen wird. Der Einsatz von Betonpumpen ist aber für einige besondere Betonzusammensetzungen mit Problemen behaftet.

3. BEWERTUNGSKRITERIEN ZUR QUALITÄTSSICHERUNG

Das vorrangige Ziel beim wirtschaftlichen Einsatz einer Betonbaumaschine besteht darin, eine entmischungsfreie Betonabgabe zu gewährleisten. Dabei ist es nicht sinnvoll, für eine Verfahrensstufe bezüglich der abzugebenden Betonqualität strenge Vorschriften auszuarbeiten und die nachfolgenden Verfahrensstufen unberücksichtigt zu lassen.

Zur Überwachung der Betonabgabe bzw. zur Sicherung der Betongüte sind die genannten Betonbaumaschinen durch entsprechende Bewertungskriterien und Meßmethoden einzuordnen. Dabei werden als Beurteilungsmerkmale die Verteilung des Wassers, des Mehlkorns und der Korngruppen von 0.25 mm bis 32 mm gewählt. Die Bedeutung dieser Merkmale für die Betongüte ist aus der Literatur hinreichend bekannt.

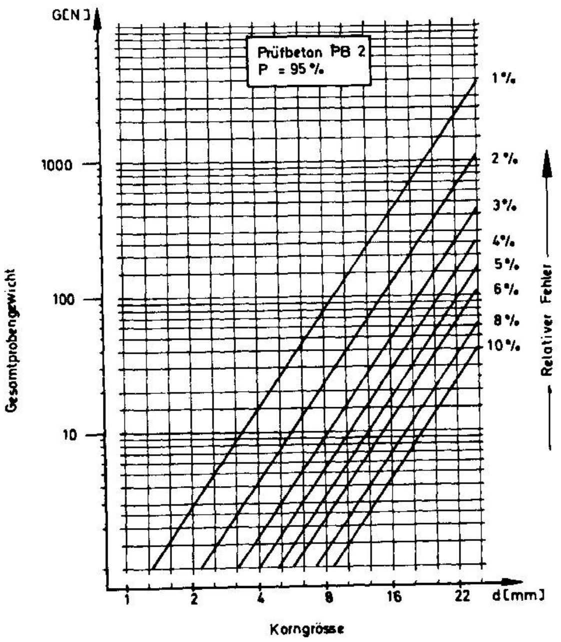

Bild 1 Nomogramm zur Bestimmung des Gesamtprobengewichtes

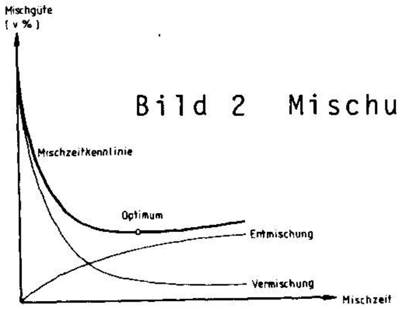

Bild 2 Mischungsverlauf

	Halbautomatische Anlagen			Vollautomatische Anlagen			
Mischergröße (m³)	0.5	bis 1.0	bis 1.5	0.5	bis 1.0	bis 1.5	2.0
Personalkosten (%)	50	31	27	34	18	15	12
Maschinenkosten (%)	17	22	21	21	26	26	28
Nebenanlagen-Kosten (%)	21	30	31	30	37	38	38
Betriebskosten (%)	6	8	10	8	8	9	10
Verschleißkosten (%)	4	5	7	4	6	7	7
Montage - Demontage - Kosten (%)	2	4	4	3	5	5	5

Tabelle 1 Spezifische Kosten anteile Betonproduktion

Die bisher praktizierte Methode zur Beurteilung der Anlagenmischer und der Transportbetonmischer nach den erzielten Betondruckfestigkeiten brachte vielfach widersprechende Prüfergebnisse. Deshalb wurden unter Berücksichtigung der Stichprobentheorie, neue Prüfverfahren erarbeitet. Mit der Teilchengrößenanalyse lassen sich die kinetischen Eigenschaften der Mischgutkollektive und des Gesamtprobengewichtes (Bild 1) besser erfassen. Dabei wird die Beurteilung der Gleichmäßigkeit der Merkmale durch den Mittelwert, die Standardabweichung und den Variationskoeffizienten vorgenommen. Für den Betrieb mit Betonpumpen müssen ähnliche Prüfverfahren ausgearbeitet werden.

4. EINFLUSSGRÖSSEN

Die Einflußgrößen bezüglich der Frischbetonqualität sind in den einzelnen Verfahrensstufen vielfältiger Art. Die charakteristischen, voneinander unabhängigen Parameter können für den Mischvorgang prinzipiell in folgende Gruppen eingeteilt werden:

- systemtechnische Einflüsse,
- betriebstechnische Einflüsse,
- betontechnologische Einflüsse.

Die systemtechnischen Einflußgrößen werden durch die

- Geometrie des Mischraums,
- Geometrie der Mischwerkzeuge,
- Bearbeitungsgeschwindigkeit und die
- externen Kräfte

im wesentlichen bestimmt.

Mit den betriebstechnischen Einflußgrößen wird der Vorgang im Mischraum erfaßt. Hierbei wird die Betonmischgüte besonders vom Füllvolumen und der Mischdauer beeinflußt.

Bei den betontechnologischen Einflüssen sind die Stoffgrößen des Mischgutes angesprochen. Dabei sind von wesentlichem Einfluß

- Rohdichte,
- Reibungskoeffizient Werkzeug/Mischgut,
- Kornform,
- Kornfestigkeit,
- Kornverteilung,
- Wasser/Zement-Wert,
- dynamische Mischgut-Zähigkeit,
- Scherfestigkeit.

Beim Einsatz von Betonpumpen sind im wesentlichen folgende Parameter zu beachten:

- Fördermenge,
- Förderleitungsdurchmesser,
- Föderleitungslänge/-Höhe,
- Betonkonsistenz,
- Kornzusammensetzung,
- W/2 - Faktor,
- Mehlkornanteil,
- Zusatzmittel/Zusatzstoffe,
- Rohrleitungszustand,
- Schieberzustand.

Neben den Einflußgrößen auf die Frischbetonqualität sind für eine systematische Erfassung auch die wirtschaftlichen Gesichtspunkte zu berücksichtigen.

Bei der Diskussion über die Wirtschaftlichkeit der unterschiedlichsten Betonbaumaschinen sind die Herstellkosten, die Stoffkosten und für die Fahrmischer die Transportkosten zu beachten.

Die Herstellkosten werden im wesentlichen durch die

- Jahresproduktion/Jahresumsatz,
- Abschreibung und Verzinsung,
- Montage- und Reparaturkosten,
- Betriebs- und Lohnkosten

bestimmt.

Die Stoffkosten werden im Vergleich gleichartiger Betonbaumaschinen kaum variieren. In Einzelfällen kann durch eine geschickte Verfahrenswahl eine Reduktion von Zement, Zusatzstoffen und Zusatzmittel möglich werden.

Die Transportkosten sind für die Fahrmischer relevant. Dabei ist die Aufgliederung der Kostenarten ähnlich wie bei den Herstellkosten vorzunehmen.

Voraussetzung für einen wirtschaftlichen Einsatz der Betonpumpe bei ununterbrochener Einsatzdauer ist die richtige Abstimmung zwischen der Zusammensetzung des Frischbetons und der Betonpumpe unter Berücksichtigung der geforderten Förderleistung.

5. UNTERSUCHUNGSERGEBNISSE

5.1 Qualitative Analyse

Die Frischbetongüte wird entscheidend von der Mischgutzusammensetzung beeinflußt. Beim Mischvorgang sind im wesentlichen die folgenden Phasen zu beobachten:

- I. Phase Vormischung
- II. Phase Hauptmischung (Bild 2)
- III. Phase Entmischung

In der III. Phase beginnen sich die einzelnen Kollektive aus dem Mischgut herauszulösen. Dabei zeigen sich bei einigen Mischsystemen umso stärker Entmischungserscheinungen je unterschiedlicher die einzelnen Stoffgrößen sich darstellen. In dieser Phase kommt es vielfach zu deutlichen Separierungs- und Agglomerationserscheinungen.

Beim Transport des Betons zeigen sich häufig Mängel bei der Durchführung mit werksgemischtem Beton. Durch den Einsatz von neuen Technologien bei der Fahrzeugmischung werden deutliche Vorteile ersichtlich. Die erzielte Betonmischgüte wird vergleichbar mit Ergebnissen aus guten Anlagenmischern. Dadurch wird es möglich, Frischbeton unter extremen Klimabedingungen ohne Qualitätsverlust bereitzustellen. Für viele Betonlieferfirmen in gemäßigten Klimazonen, die besonders in der warmen Jahreszeit Frischbeton über relativ lange Entfernungen anbieten gilt das gleiche.

Beim Entwurf einer pumpfähigen Betonmischung müssen für die Betonfestigkeitsklassen und für die Verarbeitbarkeit folgende Kriterien beachtet werden:

a) Kornzusammensetzung,
b) Zementgehalt,
c) Zementleimgehalt,
d) Mehlkorngehalt,
e) Konsistenz.

Bei einem ungesättigten Zuschlaggemisch, d.h. der Beton enthält nicht ausreichend Zementleim bzw. Feinmörtel, um den Grobzuschlag zu umhüllen und die Haufwerksporen des Zuschlags auszufüllen, wird der Druck des Förderkolbens unmittelbar auf das Korngerüst übertragen, und es kommt nach bodenmechanischen Gesetzen zu Verkeilungen und Brückenbildungen innerhalb des Zuschlaggemisches. Wegen der entstehenden Rohrverstopfer ist das ungesättigte Zuschlaggemisch nicht pumpfähig.

Enthält die Betonzusammensetzung genügend Zementleim bzw. Feinmörtelanteile, also ein gesättigtes Zuschlaggemisch, wird beim Pumpen die Kolbenkraft nach hydrostatischen Gesichtspunkten auf den Zementleim übertragen, wobei eine annähernd ungehinderte Strömung des Frischbetons entsteht, in der die Zuschlagkörner gewissermaßen freischwebend mitgeführt werden. Bei einem derartigen Zuschlaggemisch kann der Zementleimanteil auch etwas höher ausfallen als zur Sättigung notwendig. Damit ist dem Zementanteil eine entscheidende Bedeutung beizumessen.

5.2 Kostenanalyse

In der Tabelle 1 sind die spezifischen Kostenanteile der Betonproduktion zusammengefaßt. Eine ähnliche Kostenerfassung für werksgemischten und fahrzeuggemischten Beton ist in Bild 3 dargestellt.

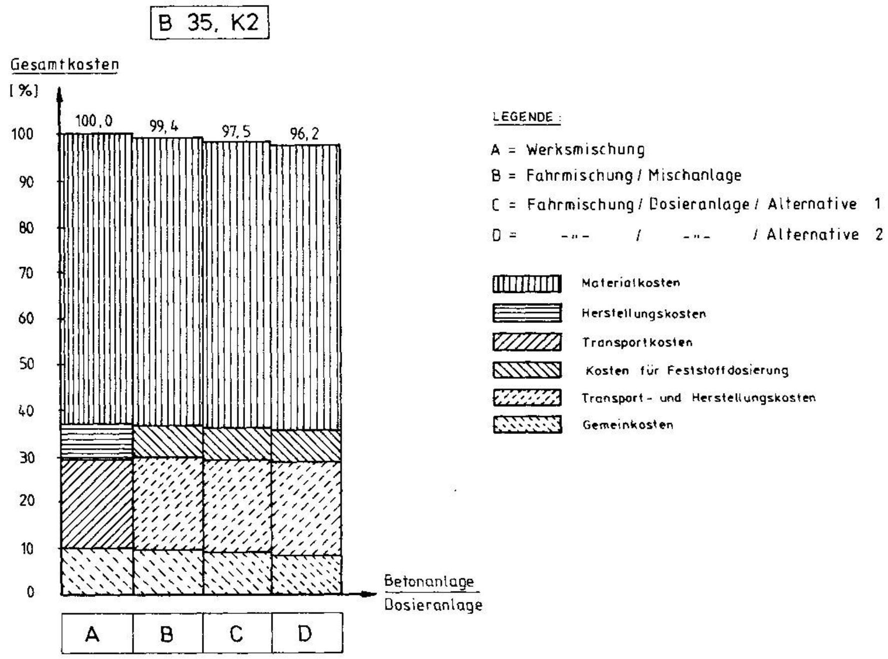

Bild 3 Kostenerfassung für Transportbeton

5.3 Bewertung

Eine systematische Bewertung erfolgt mit Hilfe der Systemanalyse in einer Bewertungsmatrix. Danach ist bei der Auswahl von Betonbaumaschinen auf folgendes zu achten:

Aus betriebstechnischen Überlegungen sollten durch eine bessere Ausnutzung des günstigen Mischzeitenbereiches (Phase 2) die vielfach zu kurzen bzw. zu langen Mischzeiten abgestellt werden. Dies würde zu einer weiteren Verbesserung der Betonqualität und zu einer Erhöhung der Ausstoßleistung führen.

Auch in energietechnischer Hinsicht könnten durch eine bessere Ausnutzung der Mischzeit Betriebskosten gesenkt werden. Die günstige Auswirkung der optimalen Mischzeit auf die Standzeiten der Verschleißelemente sollten hier noch den genannten Vorteil abrunden.

Der Fahrmischer hat innerhalb der Förderkette "Betontransport" eine nicht mehr wegzudenkende Aufgabe übernommen. Die Frischbetonqualität aus einigen Fahrmischersystemen läßt noch viele Wünsche offen. Bei einigen Neuentwicklungen zeigen sich im verfahrenstechnischen und betontechnologischen Bereich Verbesserungen.

Frischbetone mit einem guten Verarbeitungsvermögen und mit guter Verdichtungswilligkeit lassen sich in der Regel problemlos pumpen. Das gilt besonders, wenn die geräteseitigen Anforderungen zufriedenstellend erfüllt sind.

Bei einigen Betonpumpen zeigen sich systembedingte Mängel, die die Betonqualität und die Betriebssicherheit negativ beeinflußen. Allerdings kann unter Beachtung neuer Technologien der Einsatz dieser Baumaschinen selbst unter extremen Randbedingungen erfolgen.

Sulphur Concrete for Foundations in the Arabian Gulf Area

Béton de soufre pour les fondations dans les régions du Golfe Arabique

Schwefel-Beton für Fundamente im Arabischen Golf

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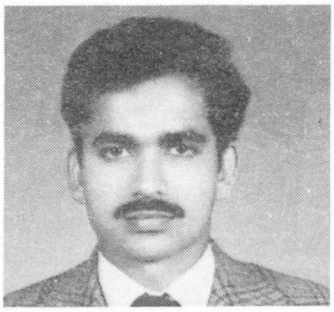

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SUMMARY
In this paper, the possibility of using sulphur concrete in foundations has been examined. The early gain in strength, high strength, low permeability and high electrical resistivity of sulphur concrete are all in favor of its use as a foundation material. However, the bond characteristics and the in-situ performance of sulphur concrete, particularly in terms of its reinforcement corrosion, are some of the parameters that need to be further investigated, before the material can be used in foundations with confidence.

RÉSUMÉ
Il existe une possibilité d'utiliser du béton de soufre dans les fondations. L'augmentation rapide de la résistance mécanique, la faible perméabilité et la haute résistivité électrique de ce béton sont favorables à une utilisation comme matériau de fondation. Cependant, les caractéristiques d'adhérence et le comportement "in-situ" du béton de soufre, la corrosion de l'armature sont quelques-uns des paramètres qui nécessitent une étude plus approfondie en vue d'une utilisation sûre de ce matériau dans les fondations.

ZUSAMMENFASSUNG
Es werden Untersuchungen über die Verwendung von Schwefel-Beton für Fundamente im Arabischen Golf beschrieben. Die hohe Festigkeit, die kleine Durchlässigkeit und der hohe elektrische Widerstand sprechen für die Verwendung von Schwefel-Beton in Fundamenten. Verbundprobleme und das Verhalten des Betons bezüglich der Bewehrungskorrosion zeigen jedoch, dass noch weitere Untersuchungen durchgeführt werden müssen, bevor dieser Betontyp praktisch verwendet werden kann.

INTRODUCTION

The foundations of one or two storey housing units in the Arabian Gulf area are usually constructed with steel reinforced Portland cement concrete (PCC) of typical mix designs that give a compressive strength of the order of 20 to 28 MPa (3000 to 4000 psi). Such PCC is relatively porous and when it is placed in grounds with high water table, as is the case all along the coastal areas of the Arabian Gulf, they act as tree-roots and transfer the salt laden ground water to the remaining structural members of the housing unit by capillary action. The concentration of salts increases in the concrete pores. This and other corrosion collaborating parameters lead to the corrosion of steel reinforcement in those concrete structural members in a short period of time.

In this paper, an experimental study has been carried out to examine the possibility of using sulphur concrete (SC) as an alternate to PCC for foundation constructions in the Arabian Gulf area. SC, is a relatively impermeable thermoplastic material which is composed of sulphur, coarse aggregate, sand, and filler powder. The present SC technology utilizes a modified (plasticized) sulphur cement which is prepared by reacting elemental sulphur with chemical modifiers, unlike the pure elemental sulphur used in early developments [1-4]. The modified sulphur cement reduces the brittleness of the resulting SC product, associated with elemental sulphur, and improves the durability of SC in aqueous environments. The SC produced by using modified sulphur cement, developed by U.S. Bureau of Mines researchers [1,2], has been successfully used in a major rehabilitation project in which 2700 m^2 (29000 ft^2) of a deteriorated PCC floor was overlaid with 9 cm (3.5 in) thick unreinforced SC, and 530 m^2 (5700 ft^2) of walls and piers were lined or encapsulated with 10 cm (4 in) thick unreinforced SC [5]. The U.S. Bureau of Mines modified sulphur cement is produced by reacting elemental sulphur and chemical modifier in a ratio of 19:1 at 145^{o}C (293^{o}F) for 6 hours. The chemical modifier consists of dicyclopentadiene (DCPD) and oligomers of cyclopentadiene (CPD) in equal proportions. The other examples of commercial application of SC are the repair of PCC foundations and highways [6,7].

The production technique of SC involves the blending and heating of coarse aggregate, sand and filler powder to a temperature of 171 to 193^{o}C (340 to 380^{o}F) and then mixing these constituents with modified sulphur cement. A hot homogeneous mixture in the temperature range of 127 to 149^{o}F (260 to 300^{o}F) is obtained which can easily be poured, vibrated and finished [8]. Various types of mixers such as heat jacketed concrete transit mixers and modified mobile asphalt batch plants are used in in-situ SC casting. The working time available for placing and finishing of SC is relatively short. It is approximately 30 minutes for large SC batches. Efforts are needed to minimize the heat loss from the SC mixture during the entire casting operation. Concrete buggies with insulated hoppers should be used to transport the hot SC mixture from the mixer to the placement area. Some heat source such as an infrared heating unit should be used to reheat SC if it starts solidifying before finishing. The SC cures within 6 hours and attains about 80% of its final strength. Unlike PCC, the casting of SC requires more safety measures. This includes the use of protective clothing, safety glasses, face shields, gloves and hard hats by the personnel involved in SC casting. The emission of toxic gases such as sulphur dioxide and hydrogen sulphide can be controlled well below the allowable threshold limit, if the temperature of hot SC mixture is maintained in the range of 127 to 149^{o}C (260 to 300^{o}F).

The compressive strength of SC ranges between 35 MPa to 62 MPa (5000 to 9000 psi) depending upon the sulphur cement content and the type and grading of the aggregate. The tensile and flexural strengths of SC are 12 to 15% and 15 to 20% of its compressive strength respectively. Its modulus of elasticity is of the order of 27.5 GPa (4 x 10^6 psi) [2]. SC is classified as a concrete with high resistance of chemical environments, unlike PCC which is easily attacked by various chemicals, particularly acids. The long term metallurgical and fertilizer processing plants has been evaluated in a joint research program by U.S. Bureau of Mines and The Sulphur Institute [2]. After 3 to 5 years of exposure, the performance of SC in most of the acidic environments has been reported satisfactory. In similar environments, PCC has been partially or completely destroyed.

A cost analysis by Muir [9] for the year 1985 indicates that on cost basis SC is competitive with PCC. The cost of 35 to 55 MPa (5000 to 8000 psi) PCC ranges from 45 to 68 dollars per cubic meter. The same strength SC can be produced at a cost of 39 to 46 dollars per cubic meter.

EXPERIMENTAL STUDY

In this study, some properties of SC considered significant in its evaluation as a foundation material such as water absorption, electrical resistivity, resistance to reinforcement corrosion and bonding with PCC, were determined. These properties were then compared with those of PCC. The SC was produced by using the modified sulphur cement, Chement 2000 developed by U.S. Bureau of Mines [1,2]. The locally available coarse aggregate, dune sand and limestone filler powder (minus 75 micron sieve) were used in the SC production. The laboratory casting procedure followed was similar to that described in the preceding section.

For water absorption, electrical resistivity and reinforcing steel corrosion tests, 75 mm ϕ x 150 mm (3 in ϕ x 6 in) cylindrical specimens were prepared from different SC and PCC mixes. For the reinforcing steel corrosion tests, the cylindrical specimens were provided with a 13 mm diameter (#4) steel bar placed in the center with a clear cover of 32 mm (1.25 in). Three mix designs for SC, 15/10/75 (SC-1), 18/10/72 (SC-2) and 22/10/68 (SC-3) by weight of sulphur cement, filler powder and total aggregate were used. The coarse aggregate and sand were used in equal proportions. The PCC mixes were designed with different water-cement (w/c) ratios of 0.4 (PCC-1), 0.55 (PCC-2) and 0.7 (PCC-3). A cement content of 450 kg/m^3 (758 lb/yd^3) was used with coarse aggregate and sand in equal proportions.

The water absorption was measured by calculating the percentage increase in the weight of dry SC specimens after 24 hours of immersion in water at 21^{o}C (70^{o}F). In case of PCC, the specimens were oven-dried at 110^{o}C (230^{o}F) for 24 hours and then allowed to cool before immersion in water. The electrical resistivity measurements were made by using a commercially available resistivity meter, Nilsson 400. The impressed voltage, half-cell potential, and corrosion rate measurement techniques used in this study for corrosion testing have been described elsewhere [10].

To study the bond behavior of SC with PCC, composite cylinders, 150 mm ϕ x 300 mm (6 in ϕ x 12 in) were prepared with one half consisting of SC and the other half PCC having a diagonal bond line joining the two halves of the cylinder at a plane of 30^o from the longitudinal axis (see Fig. 1). When casting, the test of SC (or PCC) portion of the composite cylinder was allowed to cure properly before casting the other portion of the cylinder. In one set of specimens, SC was cast first followed by casting PCC while in another set of specimens PCC eas cast first then SC. In these tests, SC with mix proportions of 25/10/36.5/28.5 by weight of sulphur, silica flour filler, limestone aggregate, and dune sand; and PCC with w/c ratio of 0.4 were used. The compressive strength of the composite cylinder was compared with similar complete cylinder made of SC and PCC.

The composite cylinder test developed by Arizona Highway Department is used to test the bond between epoxy compounds and PCC in repair works. In these tests, if the composite cylinder consisting of two halves of PCC and an epoxy compound, has 90% of the compressive strength of the homogeneous PCC cylinder, then the bond of the epoxy compound with PCC is considered satisfactory.

All the tests were carried out on a set of at least 3 specimens.

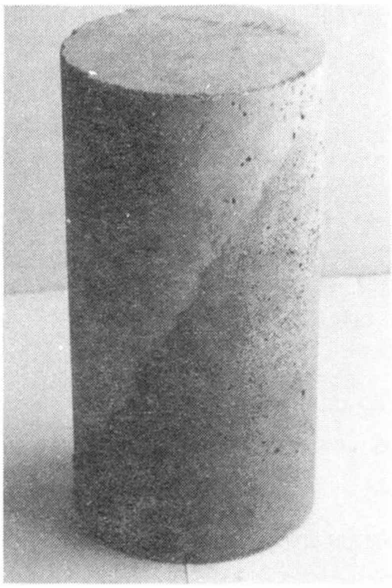

Fig. 1 Composite Cylinder of SC and PCC used for bond test

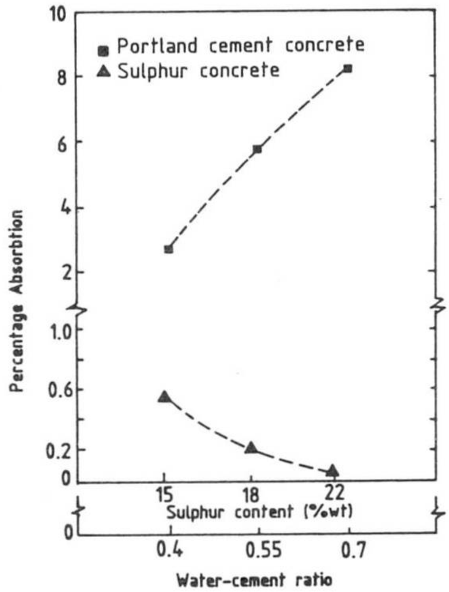

Fig. 2 Absorption of SC and PCC

RESULTS AND DISCUSSION

Water Absorption

The water absorption of the different SC and PCC mixes is shown in Fig. 2. It is noticed that the water absorption of the SC mixes varies from 0.04 to 0.55% as compared to 2.75 to 8.23% in case of the PCC mixes. The low permeability of SC is one of the main factors responsible for its high resistance against various corrosive solutions. The recommended water absorption of a corrosion resistant SC is less than 0.1% and preferably below 0.05% [8]. In this study, the acceptable water absorption is obtained in a SC mix containing 22% sulphur cement (see Fig. 2). This amount is in excess of optimum sulphur cement requirement from the point of view of strength and it causes undesirable shrinkage of finished SC. A sulphur cement content of 18% which has been found optimum in this study as far as strength and workability are concerned, has yielded SC with water absorption of 0.2%. On the other hand, Sullivan [8] has produced SC with less than 0.05% water absorption by using 16% sulphur cement. The higher water absorption of SC, even with increased sulphur cement content, obtained in this study is attributed to the quality of the coarse aggregate. As the limestone coarse aggregate used does not meet the criteria of an acceptable aggregate used in SC production. The water absorption of the aggregate used is 2.2% as compared to a maximum recommended value of 1% [2]. It is worth mentioning that a compressive strength of 57 MPa (8265 psi) has been obtained in this study for the optimum mix SC-2.

Electrical Resistivity

The SC and PCC specimens subjected to electrical resistivity measurements have been partially immersed in 5% sodium chloride solution for 15 months. The electrical resistivity of all the SC mixes, SC-1, SC-2 and SC-3 have been found in excess of 4.9×10^6 ohm-cm respectively. The electrical resistivity of SC could not be measured beyond 4.9 x 10^6 ohm-cm because the instrument used has a maximum range of 1.1 mega ohms which corresponds to an electrical resistivity value of 4.9 x 10^6 ohm-cm for the size of specimens used in this study.

The high electrical resistivity of SC is due to the fact that SC, unlike PCC, does not include water in its composition and also its low permeability reduces the ingress of environmental moisture. The hydrated cement paste of PCC provides an easy path for charge transfer. The significance of electrical resistivity has been realized in the reinforcement corrosion of PCC. Stratfull [11], on the basis of his field investigation has pointed out that deterioration of PCC due to reinforcement corrosion is negligible when its electrical resistivity is in excess of 60,000 ohm-cm.

Corrosion of Reinforcing Steel

In the impressed voltage testing, SC and PCC specimens have been tested under a constant voltage of 6 volts versus saturated calomel electrode (SCE). The variation of current has been recorded with time. Usually, a sharp rise in current has been found coincident with the time of visible cracking of concrete. The reinforced specimens have been immersed in 5% sodium chloride solution for a period of 5 months before testing.

The obtained results indicate that the current flow in all the SC specimens is negligible. A maximum current of 2.7 mA has been recorded after 60 days of testing. None of the specimens from the SC mixes have shown any cracking after 60 days. On the other hand, in case of PCC specimens current values as high as 170 mA have been recorded and the time needed for the appearance of first crack has been found to be 60, 36 and 12 hours for mixes PCC-1, PCC-2 and PCC-3 respectively. Figure 3 compares the variation of current with time in typical SC and PCC specimens. On the basis of these results, one may conlcude that the resistance of SC against reinforcement corrosion is much higher than that of PCC. In this case, the better performance of SC is attributed to its low permeability and high electrical resistivity. As the low permeability reduces the ingress of corrosion inducing agents (oxygen, water and chlorides) to the steel surface and the high electrical resistivity reduces the current associated with electrocemical corrosion process.

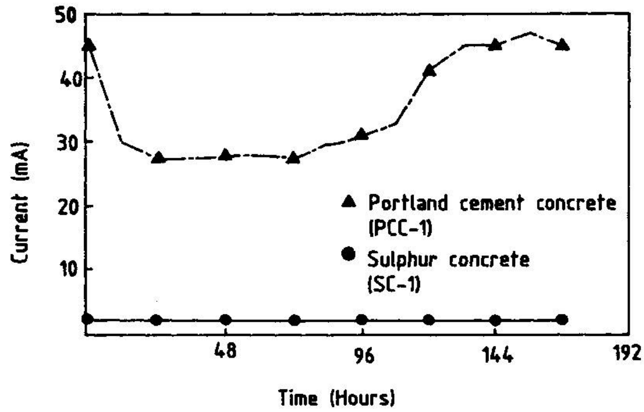

Fig. 3 Variation of current with time in SC and PCC specimens in impressed voltage testing

The corrosion monitoring results of another set of SC and PCC specimens partially immersed in 5% sodium chloride solution (without impressed voltage) for an extended period of 24 months are shown in Table 1. An important observation that can be made from these results is that the time to active potential of reinforcing steel in SC specimens is much longer than that in PCC specimens, but the corrosion rates are not as low as it was expected after the impressed voltage testing.

Mix Designation	Time to Active Potential (days)	Half-cell Potential (mV vs. SCE)	Corrosion Rate (μm/year) (Linear Polarization Resistance Technique)
Sulphur Concrete			
SC-1 (15/10/75)	126	-577	0.63
SC-2 (18/10/72)	++	-205	0.33
SC-3 (22/10/68)	77	-587	0.11
Portland Cement Concrete			
PCC-1 (w/c = 0.4)	84	-450	0.88
PCC-2 (w/c = 0.55)	77	-500	2.60
PCC-3 (w/c = 0.70)	36	-550	4.40

++ Passive Potential after 24 months of immersion in 5% sodium chloride solution.

Table 1 Corrosion monitoring results after 24 months immersion of SC and PCC specimens in 5% sodium chloride solution.

One of the factors that has probably contributed to lowering the reinforcement corrosion resistance of SC during the long term exposure in sodium chloride solution is the use of low quality coarse aggregate in this study. The access of corrosion inducing agents to the steel surface in SC is possible after long time immersion in aquous solution by the micro or macro cracks developed due to non-compatible volume changes of sulphur matrix and the moisture absorptive aggregates. These type of cracks may also be developed due to the presence of swelling minerals in the aggregate or filler powder. The presence of swelling mineral mica has been identified in the limestone filler powder used in this study [12]. Cracks of such nature have been found on some unreinforced SC specimens after 24 months of exposure time, although they were not detected at earlier stages. The unreinforced specimens from the same mixes were immersed together with reinforced specimens for detecting these type of cracks. It is expected that the use of good quality aggregates and filler powder would ensure a long term superior performance of SC against reinforcement corrosion.

Bonding Behavior of SC with PCC

The compressive strength of composite cylinders of SC and PCC along with those of SC and PCC cylinders is shown in Table 2. It can be observed that the compressive strength of composite cylinders is 27 to 45% of that of SC cylinders and 20 to 32% of that of PCC cylinders. The set of composite cylinders that had a rough bond surface made by a chisel have shown 13% higher strength than similar cylinders cast with smooth surface. The failure of most of the specimens is in the form of slip along the bond surface. Based on the bond criteria developed by Arizona Highway Department, the bonding between SC and PCC is not adequate.

Specimen Type	Average compressive strength [kg/cm^2 (psi)]
Composite; PCC top, SC bottom Smooth bond surface.	108 (1536)
Composite; SC top, PCC bottom Smooth bond surface.	154 (2190)
Composite; SC top, PCC bottom Rough bond surface.	177 (2517)
Complete SC.	397 (5645)
Complete PCC.	555 (7892)

Table 2 Compressive strength of SC and PCC composite cylinders having a diagonal bond line.

CONCLUDING REMAKRS

The results obtained in this study indicate that SC may be potentially used in foundation construction. However, there are certain problems that must be resolved before the use of this material in such a critical application can be ascertained. Most important is the selection of a good quality aggregate that would ensure production of good quality SC with extremely low moisture absorption and improved resistance against reinforcement corrosion in long term exposure. The selected aggregates, as apecified by some U.S. Bureau of Mines reports, must be clean, hard, tough, strong, durable and free from any swelling constituents. Further, field testing need to be conducted and monitored to assess long term performance of this material in in-situ environment. In this regard, it is suggested that reinforced SC specimens made with acceptable quality aggregates and filler powder be burried in grounds of high water table and tested at different periods to observe the changes in material properties that may take place. Another problem of concern in using SC as a foundation material is the poor bonding between SC and PCC. This problem might be resolved by investigating the use of some bonding agent such as a mixture of styrene butadiene rubber (SBR) latex and cement slurry used in PCC repairs.

ACKNOWLEDGEMENTS

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Polymer Concrete for Construction and Repair of Bridges

Béton de polymère pour la construction et la réparation de ponts

Polymer-Betone für Bau und Reparatur von Brücken

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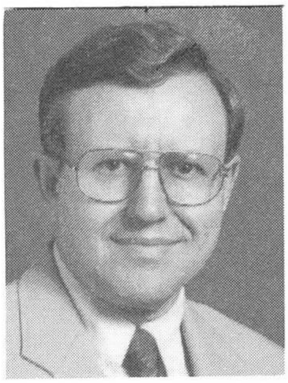

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SUMMARY
The use of concrete-polymer materials for bridge deck construction and repair has increased significantly. Polymer concrete repairs and overlays have proven to be fast, durable, and cost effective. The use of polymers for sealing cracks has been found to be very simple, effective, and economical.

RÉSUMÉ
L'utilisation des matériaux en béton à fibres de polymère pour la construction et la réparation des ponts a considérablement augmenté. Les réparations et les recouvrements en béton de polymère ont montré qu'elles étaient durables et efficaces. L'utilisation de "polymères" pour sceller les fissures est très simple, efficace et économique.

ZUSAMMENFASSUNG
Die Anwendung von Polymer-Beton für die Erstellung von Brückenfahrbahnplatten und Reparaturen hat deutlich zugenommen. Ausbesserungen und Ueberzüge aus Polymer-Beton sind schnell ausgeführt und dauerhaft. Die Anwendung von Polymeren für die Versiegelung von Rissen ist einfach, wirksam und kostengünstig.

1. INTRODUCTION

The development of latex-modified concrete (LMC) in the mid-fifties marked the first beginning in the United States of the use of concrete-polymer materials. Styrene-butadiene (SBR) latex was first used for a bridge deck overlay in Michigan about 30 years ago [1]. Polymer concrete (PC) began to be used in the United States for bridge repairs in the 1970's, and later polymer concrete overlays were used for new and old bridges. More recently high molecular weight methacrylate (HMWM) has been used to seal cracks in bridges.

The most widely used monomers and resins used to produce polymer concrete for bridge applications are methyl methacrylate (MMA), polyesters, HMWM, and epoxies. It is essential that their strength, coefficient of thermal expansion, and elongation be properly selected for the intended application.

2. PROPERTIES

The properties of polymer concrete are highly dependent upon the monomer or resin used, aggregate type and gradation, polymer content, and temperature. Compressive strengths generally range from 50 to 100 MPa while flexural strengths range from 10 to 25 MPa. Modulus of elasticity usually varies from 3,500 MPa to 30,000 MPa although some polymer concretes used for overlays have values as low as 1,000 MPa. The coefficient of thermal expansion ranges from values slightly higher than for portland cement concrete to 3 or 4 times as great. Curing shrinkage is usually several times greater than for PCC, although for PC made of some polymers such as epoxies, the shrinkage is less than for PCC.

Creep, at a given stress level, is higher than that of PCC. Specific creep, measured in strain per unit stress, is in the same range of creep for PCC (2). At elevated temperatures, however, creep of PC increases significantly. Fatigue of PC is similar to PCC when the stress ratio, i.e. ratio of applied stress to modulus of rupture, is considered; however, based on absolute stress, PC has considerably more fatigue resistance (Fig. 1)[2].

Durability of PC is usually excellent when compared to PCC. The much higher impermeability of PC results in much greater resistance to freeze-thaw deterioration and chemical attack. Abrasion and wear are good to excellent.

The polymer content of PC, which varies as a function of the aggregate size and gradation and mixing methods, is usually in the range of 6 to 15 percent. The density of PC is usually about 90 to 95 percent of PCC made with the same aggregate.

3. STRUCTURAL BEHAVIOR

Polymer concrete is not being used for entire beams or bridge decks. However, it has been extensively used for partial depth and full depth repair of bridge decks. Some full depth repairs span several meters. Load-deflection tests on reinforced beams indicate good ductility (Fig. 2). Load-deflection behavior can be predicted with reasonable accuracy. Ultimate strength of PC beams can be predicted by using an equivalent rectangular stress block similar to that used for PCC except with slight modifications to the stress block constants [3].

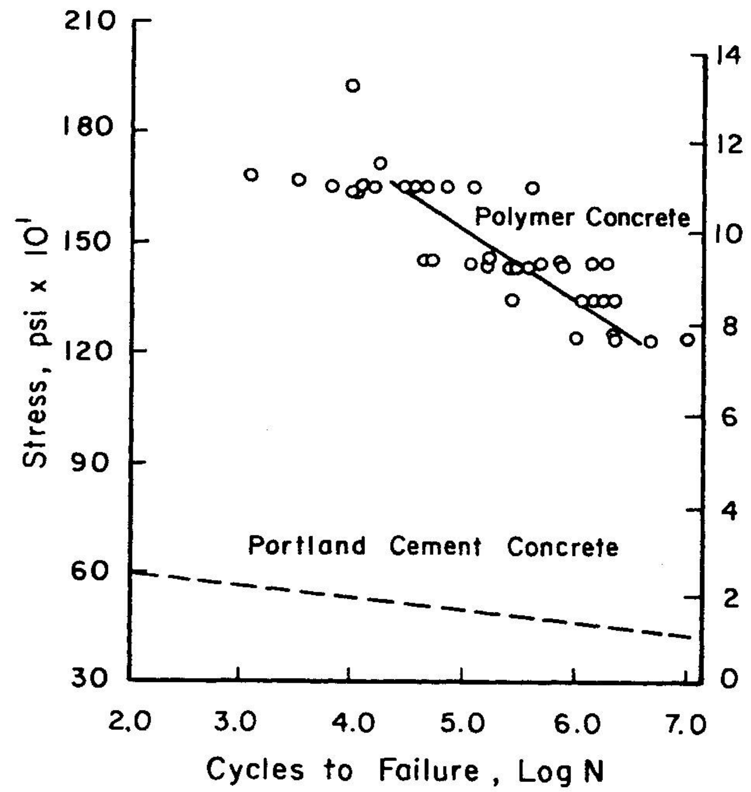

Fig. 1: Comparison of Fatigue Strength of Beams for R = 0.05 and R = 0.25

4. POLYMER CONCRETE APPLICATIONS

4.1 Repairs

The initial uses of PC in the United States was for repair of bridges. The very good bond between PCC and most polymer concretes and the very rapid cure time (30 to 90 min.) in a wide range of ambient temperatures make PC an attractive repair material. There are several ways of batching and placing PC. In all methods the finishing is similar to that of PCC.

4.1.1 Preplaced Aggregate

The simplest method is to preplace the blended aggregate in the repair area and then pouring or injecting monomer until the aggregate voids are filled. This method requires (1) a low viscosity monomer such as MMA and (2) relatively shallow lifts of 100 mm or less. The quality and resulting strength are not as high as for the other methods, but minimum equipment and clean-up are required.

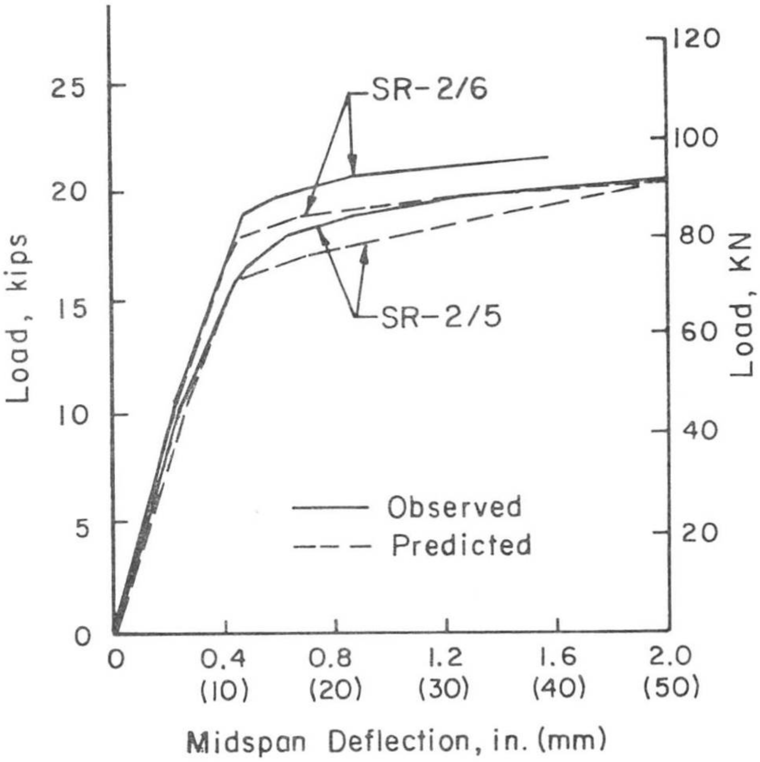

Fig. 2: Load-Deflection Responses for Beams (SR-2/5 has 4.0% steel and SR2/6 has 5.6% steel)

4.1.2 Batched Polymer Concrete.

The most common method, especially for commercially-available polymer concretes, is to batch the PC in small drum mixers (Fig. 3). The commercially-available PCs usually come in two components: a container of monomer and a bag of dry materials which includes graded fine aggregate, initiator, colorants, and thickners. When mixed together, a material with the workability of grout is produced. By adding an amount of coarse aggregate up to the volume of the PC mortar, the batch can be extended and made less costly.

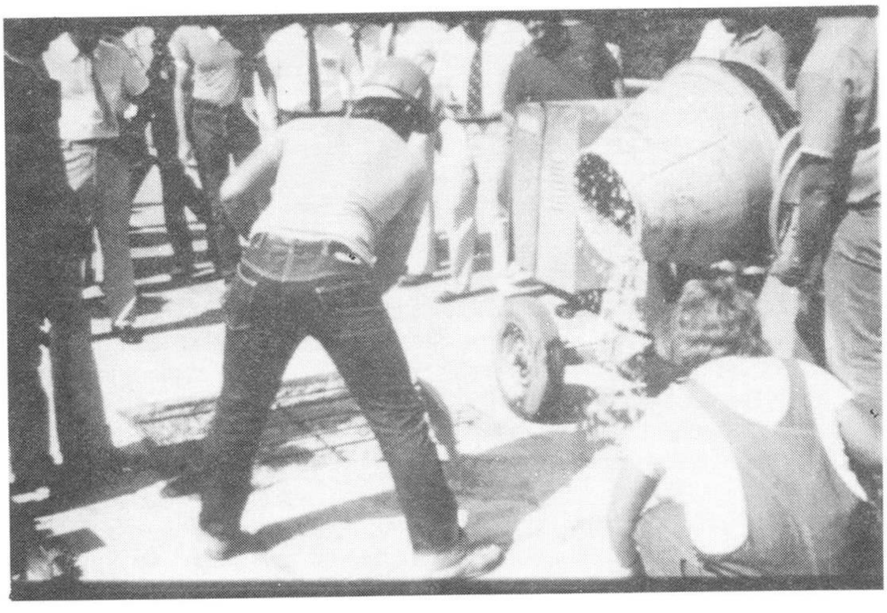

Fig. 3 Placing PC into Repair Area on Bridge

Automated batching equipment has been successfully used. Mobile concrete batching equipment used for PCC has been adapted to store all of the PC components and mix them in the mixing auger. The volume of material can be easily controlled, and the mixing time can be reduced to about one minute which is very important in hot weather to provide more time for placing and finishing.

4.2 OVERLAYS

4.2.1 Advantages

The need for overlays to provide protection and skid resistance for new and old bridge decks has created a market for PC overlays. PC has several advantages as an overlay:

1. Provides very good bond to PCC.
2. Requires a thickness of about 10 to 15 mm which results in a minimum dead load and eliminates the need to reconstruct the approach slabs.
3. Permits the construction of overlays during the night or day between periods of maximum traffic due to the fast curing of the PC.

4.2.2 Evaluation of Materials and Performance

Many monomers and resins have been used to construct PC overlays. Not all of them have resulted in durable overlays, however. MMA, polyesters, and epoxies are the materials most often used to construct overlays. Epoxy asphalt concrete has been found to provide a good wearing surface for bridge decks. The binder is a proprietary epoxy asphalt [4].

The State of Virginia has had a very active PC overlay program since 1981. Their experience has shown that the tests which provide the best indication of performance are:

1. Tensile elongation (ASTM D 638).
2. Rapid permeability test (AASHTO T 277).
3. Shear bond test in which the overlay - PCC interface is subjected to direct shear.
4. Tensile bond test recommended by ACI 503R.
5. Thermal cycling test in which cores or cylinders with overlays are subjected to thermal cycling in air between - 18°C and 38°C for up to 300 cycles. At different times during the test specimens are removed and tested for permeability and bond strength.

Virginia recommends that the tensile elongation be in the range of 20 to 50 percent. Their current choice is a polyester resin which has an elongation of 23 percent and has a modulus of 335MPa [5].

Generally the high modulus and the relatively high coefficient of thermal expansion of epoxies have resulted in poor performance as a binder for PC overlays. However, there are a few epoxies with a relatively low modulus which produce a PC with a modulus of only ~ 100MPa. Overlays made of those materials have been in place up to 10 years.

4.2.3 Overlay Applications

Polymer concrete overlays have been widely used to protect bridge decks. The applications require that a clean, dry, sound surface be provided. Shot blasting, similar to sand blasting except that small steel balls are used, is one of the most common methods. Polyester overlays are often applied in layers. A truck-mounted spray bar is used to apply the catalyzed resin (Fig. 4) at a rate of 9.3 Pa followed by a uniform layer of silica sand at a rate of 90 Pa. A second layer of resin (12Pa) is followed by a second layer of sand. A third application of resin (14.6 Pa) is followed by a third layer of sand. The thickness is about 10 mm [5].

Epoxy PC overlays have been successfully placed by using automatic mixing and dispensing units used for producing precast PC. Vibratory screeds are used to level the mix and additional aggregate is broadcast onto the surface to provide a non-skid surface. Such an overlay was used on the Brooklyn Bridge [6].

Epoxy asphalt concrete overlays are produced in a modified hot-mix asphalt plant and applied and compacted with standard asphalt paving equipment [6]. Many large bridges in the U.S. including the Golden Gate, have been successfully overlaid with this material.

PC has also been used as a wearing surface on aluminum orthotropic bridge decking. The shop-fabricated aluminum panels have a PC wearing surface applied in the shop. The panels are then attached to the bridge girders in a relatively short time. The PC/aluminum decking weights 18 to 25 lbs/ft^2 which is 1/6 to 1/8 the weight of a conventional concrete deck. The PC wearing surface is 3/8-in. thick. Although polyester was used initially, epoxy PC is currently being used since tests showed that it had superior performance [7].

Electrically conductive polymer concrete overlays have been developed for use in cathodic protection systems. Several resins, including polyester and vinyl ester, have been used with various types of coke breeze to produce composites with our electrical resistivity of 10 ohm-cm or less. These materials can be sprayed on bridge sub-structures including vertical and overhead surfaces [8].

It is estimated that at least 100 bridges in the United States and Canada have been overlaid with PC. About 20 have used epoxy; most of the rest were constructed with polyester PC, although a few have used MMA. The in place cost for polyester PC overlays in Virginia in 1985 ranged in cost from $30.50 to $43.00 per sq. m. [5].

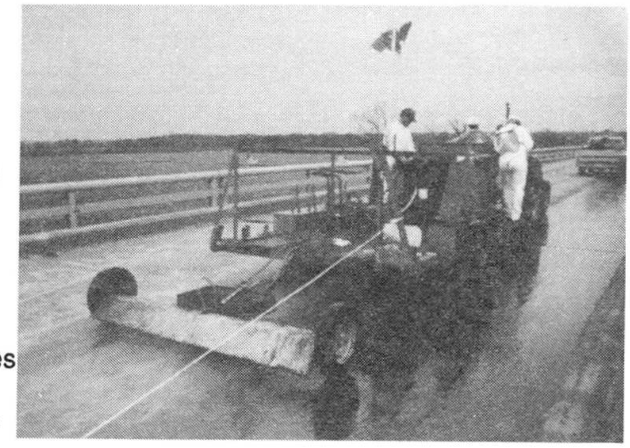

Fig. 4. Polyester Resin Applied by Spray Bar for Construction of Overlay

PC headers have been used in Texas at bridge joints when asphalt concrete overlays are added. The excellent bond to portland cement concrete and the very good impact resistance of the cast in place PC have resulted in a very durable header. The headers are about100 mm wide and 40 mm high.

5. POLYMER CRACK REPAIR

5.1 Monomer

Monomers have been developed which have the ability to penetrate open, narrow cracks in portland cement concrete, fill most of the crack and structurally bond the concrete (9). High molecular weight methacrylate (HMWM) has a viscosity nearly as low as MMA, but has a higher flash point and low odor. Similar to MMA, HMWM can be cured at a wide range of ambient temperatures.

5.2 Laboratory Tests

Laboratory tests have been performed on cracked reinforced portland cement concrete slabs. The variables were crack width at the surface (0.2 to 2 mm) and moisture levels. Monomer was brushed on the surface of the slab and permitted to cure. Slabs were then recracked and cut perpendicular to the cracks. The recracking stress averaged about 90 percent of the initial cracking stress. About 90 percent of the new cracks were outside the repaired cracks. For dry concrete, over 80 percent of the crack length was filled. For wet concrete permitted to dry for at least 24 hours, at least 50 percent or more of the crack length was filled.

5.3 Field Applications

Many bridge decks have been treated with HMWM. In some cases the bridges were new, with cracks resulting from plastic shrinkage. In other cases bridges were up to 40 years old. The procedure for treating a bridge deck consists of:

(1) Cleaning the bridge deck using a light sand blast if the surface is contaminated.

(2) Pouring monomer onto the deck and brooming it into the cracks or, for larger areas, applying monomer with a truck-mounted spray bar.

(3) Applying a light application of sand on the surface to improve skid resistance. Although the surface usually appears slick, the skid resistance is about the same after the treatment as before.

The application rate is about 2 to 3 sq. m/l and the cost ranges from 3 to 5 sq. m.

6. OTHER DEVELOPMENTS

With the trend toward precast construction, there is a strong likelihood that PC will find an even greater role in the construction of bridges. Precast PCC bridge deck panels with a factory-applied thin PC overlay would provide a tough, durable water-tight membrane and wearing surface. Ribbed or sandwich panels with a PC top skin reinforced with steel fibers could potentially result in a strong, lightweight, durable panel.

Hollow precast median barriers are currently being manufactured in the U.S. Due to their low weight they can be economically transported to the site where they are filled with concrete to provide the needed mass. The smooth, attractive, durable PC exterior requires less maintenance than conventional PCC barriers. It should also be possible to produce lightweight, complex-shaped PC guard rails that are aesthetically pleasing.

7. CONCLUSIONS

The use of polymer concrete has been found to be a very effective material for repairing bridges. The excellent bond, rapid curing, and excellent mechanical properties result in cost effective and durable repairs.

The use of high molecular weight methacrylate monomer for repairing cracked bridge decks provides a relatively low cost, simple and effective method.

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High Performance Fiber Reinforced Cement Composites

Bétons de fibres à haute performance

Hohe Festigkeit Faserverstärkter Betone

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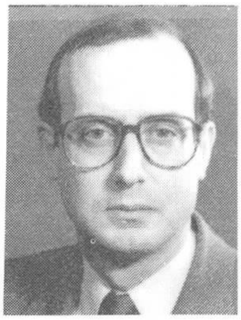

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SUMMARY
A brief synthesis of the behavior and mechanics of fiber reinforced cement based composites in tension, compression, and flexure is presented with particular emphasis on their stress-strain, stress-elongation, or load-deflection relationships. The latest advances in the field are cited and optimum strength and toughness properties achievable using current technology are pointed out.

RÉSUMÉ
Un sommaire synthétique du comportement en tension, compression, et flexion des bétons armés de fibres est présenté. Leurs courbes typiques de contrainte-déformation sont décrites. Les limites mécaniques optimales pouvant être atteintes en l'état actuel de leur développement sont mentionnées.

ZUSAMMENFASSUNG
Eine Zusammenfassung über das mechanische Verhalten faserverstärkter Betone wird gegeben. Dabei wird vor allem das Spannungs-Dehnungsverhalten beschrieben. Die neuesten Erkenntnisse werden aufgeführt, und das Vorgehen für Erhalt optimaler Festigkeit und Fähigkeit unter Ausnützung des heutigen Technologiestandes wird erläutert.

1. DEFINITIONS

The definition of a cementitious composite is that of a portland cement based matrix reinforced with fibers. The term high strength implies here strengths of up to 140 MPa (20 ksi) in compression. The term high performance refers here to the combination of strength and toughness-ductility as imparted by the addition of fibers. Concrete generally refers to concrete, mortar, or paste. Conventional FRC (Fiber Reinforced Concrete) implies premixing of discontinuous fibers with a fresh cement based matrix in proportion of less than about 3% by volume. SIFCON (Slurry Infiltrated Fiber Concrete) is a composite obtained by infiltrating with a rich cement based slurry a tridimensional network of fibers preplaced in a mold. The slurry is primarily composed of a fluid cement paste to with several additives are added such as superplasticizers, fly ash, microsilica, or polymers. SIFCON composites generally contain high volume fractions of fibers (8% to 20%) which could not be otherwise achieved by premixing the fibers with the matrix.

2. TENSION

2.1 Load-Elongation Response

Cement based matrices are known to fail in tension in a rather brittle manner and to show extremely small tensile strains at failure. The addition of fibers to such matrices, whether in continuous of discontinuous form, may lead to a composite with properties substantially improved in comparison to the properties of the unreinforced matrix. Several tensile properties of interest may be studied in details, among which the modulus of elasticity, the stress at cracking, the maximum postcracking stress, the post-peak portion of the stress strain response which symbolizes ductility, and the toughness of the composite. Although it is not within the scope of this paper to address the case of cement composites with continuous fibers or meshes like ferrocement, such composites should not be overlooked as they provide a useful basis for comparing some limiting mechanical properties.

The tensile load-elongation response of a high performance fiber reinforced cement composite such as SIFCON can be divided in the most general case into three parts (Fig.1). For all practical purposes the first part can be considered linear up to first cracking in the matrix (cracking stress), and is very similar to the load elongation response of the unreinforced matrix. Cracking is a drastic event identified by a significant change in the slope of the load-elongation curve. If the maximum post-cracking stress is larger than the cracking stress, then a second stage of behavior can be identified as the multiple cracking stage, and corresponds to the portion of the load elongation curve that joins the cracking stress point to the maximum post-cracking stress point (peak point of the curve). Several cracks develop along this portion of the curve and up to the maximum stress, defined as the strength of the composite. Beyond the peak point a third stage of behavior exists characterized by failure and/or pull-out of the fibers about a single critical crack. The corresponding descending branch of the load elongation curve can be steep or of moderate slope depending on the fiber reinforcing parameters and whether a brittle or a ductile failure occurs. Along stages I and II (Fig.1) the

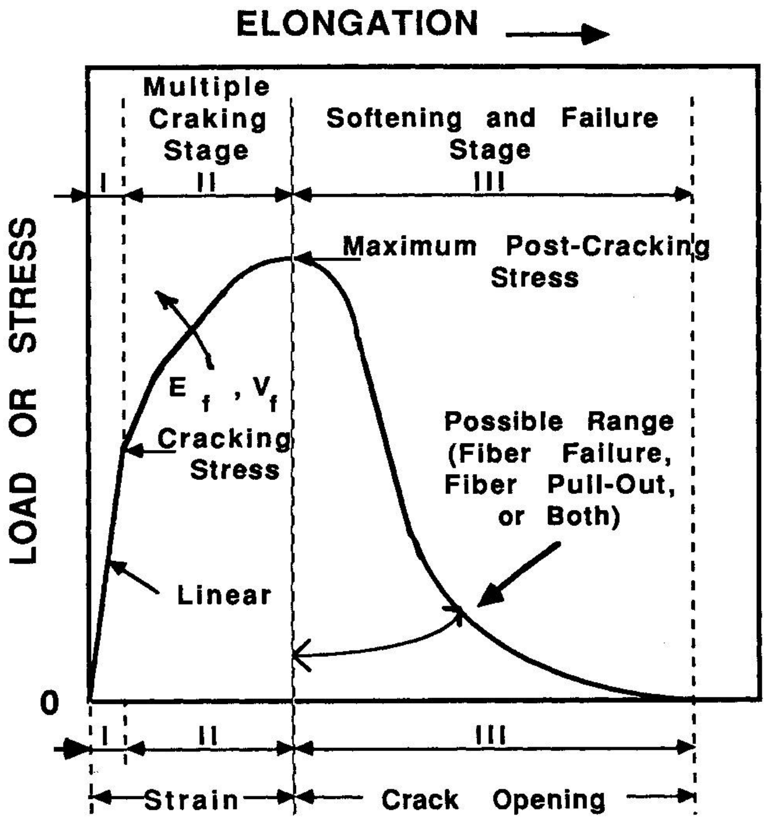
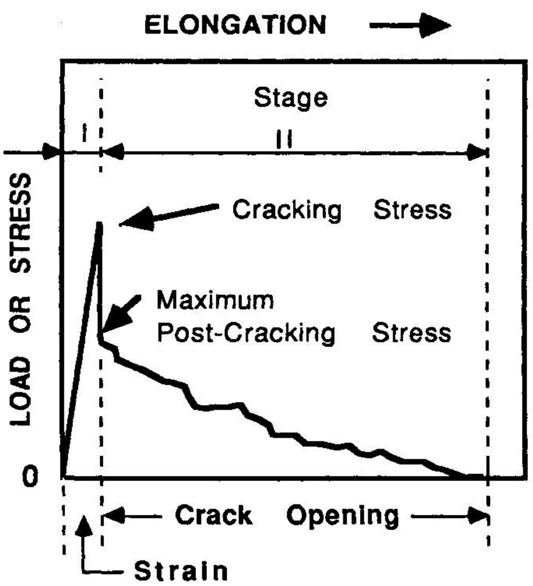


Figure 1. Typical Load Elongation Response in Tension of a High Performance FRC Composite Such as SIFCON.



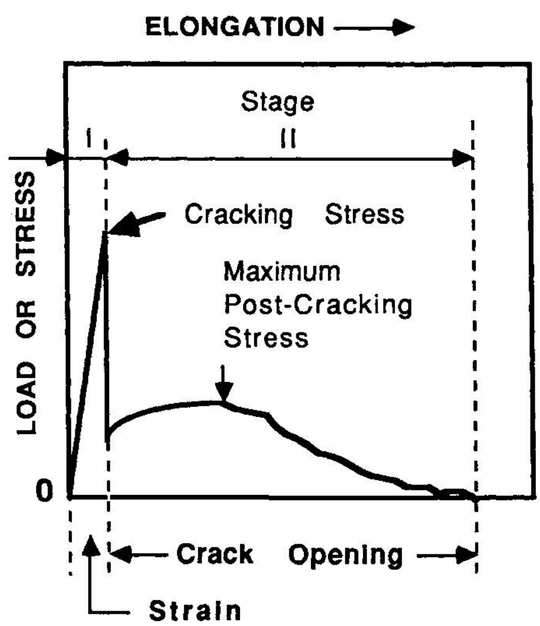


Figure 2. Typical Load Elongation Response in Tension of Fiber Reinforced Concrete: a) Using Premixed Steel Fibers, and b) Using Premixed Polypropylene Fibers.

elongation of the composite (assumed measured along a defined gage length) can be transformed into an equivalent strain. However along stage III, the elongation corresponds primarily to the opening of a single critical crack and cannot be translated into strain since crack opening is independent of the gage length.

It should be noted that multiple cracking (stage II) occurs only if the maximum post-cracking stress is larger than the cracking stress. Otherwise the second portion of the curve vanishes and the load elongation response is reduced to two main parts (stages I and III) as illustrated in Figs.2a and 2b. The typical curves of Fig.2 are characteristic of the tensile response of conventional fiber reinforced concrete with a relatively small volume fraction of fibers. The curve of Fig.2a is due to high modulus fibers such as steel fibers, while that fo Fig.2b is due to low modulus fibers such as polypropylene fibers. A typical comparison of load elongation curves of a steel fiber reinforced mortar specimen and a SIFCON specimen is shown in Fig.3.

2.2 Strength Prediction

To predict the main characteristics of the stress-elongation curve of fiber reinforced cement composites in tension, several analytical approaches can be used such as the mechanics of composite materials, fracture mechanics, damage mechanics, and empirical approches. Following are some predictions equations developed by the writer and based on the mechanics of the composite.

The tensile stress in the composite at cracking of the matrix can be predicted from the following equation [1]:

$$\sigma_{cc} = \sigma_{mu}(1 - V_f) + \alpha_1\alpha_2\bar{\tau}V_f L/d \qquad (1)$$

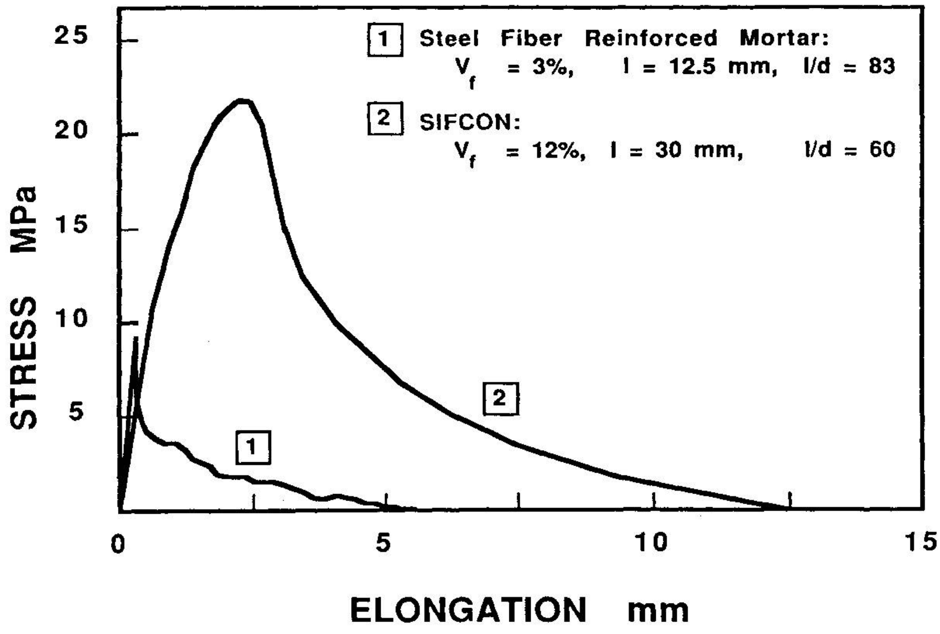


Fig.3 Typical Stress-Elongation Curves of Conventional Fiber Reinforced Mortar and SIFCON.

In which σ_{mu} is the tensile strength of the matrix, V_f is the volume fraction of fibers, L and d are respectively the length and diameter of the fiber, $\bar{\tau}$ is the average bond strength at the fiber matrix interface, α_1 is a bond coefficient representing the fraction of bond mobilized at matrix cracking, and α_2 is the efficiency factor of fiber orientation in the uncracked state of the composite.

The strain at cracking can be obtained from the stress at cracking and the modulus of elasticity of the composite assuming linear behavior. The elastic modulus of the composite can be estimated as a first approximation from the law of mixtures, that is:

$$E_c = E_m V_m + E_f V_f \tag{2}$$

in which E is used for modulus, V for volume fraction, and the subscripts c, m, and f represent the composite, the matrix and the fiber respectively.

The maximum postcracking stress can be estimated from the following equation [1] which assumes that: 1) a crack exists across the section of the test sample, the crack is normal to the tensile stress field, and 3) the contribution of the cracked matrix to the tensile strength of the composite is negligible:

$$\sigma_{pc} = \lambda_1 \lambda_2 \lambda_3 \bar{\tau} V_f L/d \tag{3}$$

in which λ_1 is the expected pull-out length ratio, λ_2 is the efficiency factor of orientation in the cracked state, and λ_3 is a group reduction factor associated with number of fibers pulling out from the same area.

No information is known to this writer on the quantitative prediction of the multiple cracking portion (stage II of Fig.1) of the stress-strain curve in tension. For conventional steel fiber reinforced concrete with low fiber contents in which stage II vanishes, the strain at maximum post-cracking stress can be taken equal to the strain at first cracking.

In order for multiple cracking to occur, the maximum post-cracking stress must be larger than the cracking stress. Using Eqs. 1 and 3 leads to the following general condition:

$$V_f\left[1 + \frac{\bar{\tau}}{\sigma_{mu}} \frac{L}{d} (\lambda_1 \lambda_2 \lambda_3 - \alpha_1 \alpha_2)\right] > 1 \tag{4}$$

Equation 4 can be used to derive a critical volume fraction for a given fiber, or a critical combination of fiber properties to achieve multiple cracking.

Little information exists on modeling the descending branch of the stress-elongation curve (stage III of Figs. 1-2), also called stress-displacement curve, stress crack opening curve, or stress softening curve. However, for steel fiber reinforced concrete in which fiber pull-out occurs through a single critical crack, two prediction equations were proposed in Refs. 2 and 3 and are suggested for use when needed. This information, combined with the use of Eqs. 1 to 4 should allow for the prediction of the entire load elongation curve of fiber reinforced concrete in tension.

3. COMPRESSION AND BENDING

Because of manuscript length constraint, sections regarding these properties have been severed from this shortened version of the paper. However, a copy of the full length paper can be obtained from the author upon request and availability. Additional information can also be found in Refs. 4 to 8.

ACKNOWLEDGEMENTS

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Kunststoffasern als Bewehrung gegen Schwindrisse im Beton

Reduced Shrinkage Cracking Using Fibre Reinforced Concrete

Armature de fibres en plastique contre les fissures de retrait dans le béton

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Toni Steiner, geboren 1935, diplomierte 1960 als Bauingenieur an der ETH Zürich. Er arbeitete während 18 Jahren in verschiedenen Bauunternehmungen im In- und Ausland, meist in Wasserbau und Betonarbeiten. Seit 1984 beschäftigt sich Toni Steiner mit Betontechnologie, speziell mit Schwindproblemen und der Verwendung von Kunststoffasern im Beton.

ZUSAMMENFASSUNG
Die Bewehrung von Beton mit Kunststoffasern gibt dem Ingenieur die Möglichkeit, die Dauerhaftigkeit des Betons in bestimmten Fällen wesentlich zu verbessern. Der Ersatz von Schwindbewehrung aus Stahl durch nicht verrottbare Kunststoffasern wird von der praktischen und wirtschaftlichen Seite her beleuchtet.

SUMMARY
The reinforcement of concrete with plastic fibres allows the engineer to enhance durability of concrete in certain cases. The replacement of steel reinforcement by plastic fibres is discussed from practical and economical viewpoint.

RÉSUMÉ
Le renforcement du béton avec des fibres en plastique donne à l'ingénieur la possibilité d'améliorer considérablement la durabilité du béton. La substitution de l'armature en acier par des fibres qui ne corrodent pas est présentée du point de vue pratique et économique.

1. DAS PROBLEM

Bewehrungsstahl in Oberflächennähe eines der Witterung ausgesetzten Betons ist immer eine potentielle Gefahr für diesen Beton! Den Beweis für diesen Grundsatz liefern die Schäden, welche den Baustoff Stahlbeton in gewissen Kreisen in der letzten Zeit in Misskredit gebracht haben. Es ist keineswegs bewiesen, dass korrodierender Stahl in einer Betonbaute einzig auf unsorgfältig ausgeführte Arbeit zurückzuführen ist. Auch das Argument der fehlenden Ueberdeckung trifft längst nicht in allen Fällen zu.

Zahlreiche Untersuchungen haben ergeben, dass der Beton zwischen der Oberfläche und der Bewehrung eine Anreicherung der Feinbestandteile (Sand, Zement und Hohlräume, die ehemals mit Wasser gefüllt waren) aufweist. Das Grobkorn wurde durch die Bewehrung von den Feinbestandteilen getrennt, da es durch die Behinderung weniger rasch in die engen Räume fliessen kann. Beim Verdichten des Betons kommt es auch vor, dass der Vibrator die Bewehrung berührt und diese in Schwingung bringt. Dadurch werden die Grobbestandteile vom Stahl wegbefördert, zurück bleibt ein mehr oder weniger dicker Mantel von Feinbestandteilen. Eine Anreicherung von Sand, Wasser und Zement bedeutet immer ein höheres Schwindmass, eine Vielzahl von kleineren und grösseren Rissen ist die Folge. Diese Risse sind meist in der ersten Phase, durch die Zementleimhaut, die sich längs der Schalung gebildet hat, verdeckt, oder sie sind so klein, dass man sie nicht sehen kann.

Nach Abwitterung der Oberflächenhaut, einige Jahre später, sind diese jedoch offen. Durch Kapillarität gelangt dann Feuchtigkeit in den Bereich der Bewehrung, die bekannten Vorgänge setzen die Korrosion in Gang, und das Zerstörungswerk beginnt (Stichworte: Saurer Regen, Salz, Karbonatisation).

Aus diesen Ueberlegungen können folgende Schlüsse gezogen werden:

1. Damit sich der Beton in Oberflächennähe nicht entmischt, darf die Bewehrung nicht zu engmaschig sein.
2. Die effektive Ueberdeckung muss grösser sein als der Durchmesser des grössten Kornes der Zuschlagstoffe.
3. Die Schwindriss- oder Rissverteilungsbewehrung, die ja immer in Oberflächennähe ist, ist durch andere Mittel zu ersetzen.
4. Der Beton muss so eingebracht werden, dass sein optimaler Kornaufbau in den äussersten Schichten auch nach der Verdichtung, nach dem Vibrieren, erhalten bleibt.

Die ersten beiden Punkte dieser Aufzählung leuchten sofort ein und bedeuten nichts Neues, auf die Punkte 3 und 4 soll im folgenden näher eingegangen werden.

2. KUNSTSTOFFASERN ALS BEWEHRUNG GEGEN SCHWINDRISSE IM BETON

Spätestens seit dem Mittelalter ist bekannt, dass die Präsenz von Fasern (Tierhaare, Stroh etc.) in kalkgebundenen Medien eine starke Rissbehinderung bewirken. Dasselbe gilt in den verschiedenen Lehmbauweisen. Heute wissen wir, dass auch in zementgebundenen Mörteln und im Beton durch Beigabe von Fasern eine nicht unbedeutende Rissbehinderung entsteht [1]. Lange Zeit konnte diese Erkenntnis nicht im grösseren Rahmen angewandt werden, da das gleichmässige Verteilen, die garantierte Präsenz der Fasern in jedem cm^3, durch Mischen in den üblichen Mischanlagen nicht möglich war. Erst als man, Ende der 60iger Jahre, dazu überging, anstelle von monofilen Fasern sogenannte Fibrillen zu verwenden, die sich durch die innere Reibung des Mischgutes während des Mischprozesses öffneten, gelang es, dieser Technologie im grössern Stil zum Durchbruch zu verhelfen. Heute sind Produkte auf dem Markt, die sich in jedem normalen Mischer mit etwas verlängerter Mischzeit gleichmässig im Mischgut verteilen.

Zur Verhinderung von Schwindrissen sind 0.1 Volumenprozente Polypropylenfasern, sofern sie homogen verteilt sind, absolut genügend. Dabei wird in Kauf genommen, dass die Druck-, Zug- und Biegezugfestigkeit nur sehr wenig erhöht wird, so wenig, dass sie der Statiker nicht berücksichtigen darf.

Alle statisch nachgewiesenen Zugkräfte sind auch im kunststoffaserbewehrten Beton mit Bewehrungsstahl aufzunehmen. Eine sauber durchgezogene Randbewehrung ist nach wie vor notwendig. Die Kunststoffasern übernehmen jedoch die innern Zwängsspannungen infolge Schwindens, Austrocknens und lokaler Temperaturdifferenzen. Sie verteilen im Frühstadium des Betons, dann, wenn die Eigenfestigkeit geringer ist als die innern Spannungen, die Inhomogenitäten, die Schwachstellen in kleinern "Einheiten" im Raum.

Die Ausnützung dieser Eigenschaft erlaubt uns heute, diejenige Stahlbewehrung, die einzig gegen Schwindrisse vorgesehen ist, durch inerte, unverrottbare Kunststoffasern zu ersetzen.

Die Tatsache, dass jeder Bewehrungsstahl, der nicht in Oberflächennähe eines der Witterung ausgesetzten Betons eingebracht wird, nicht rosten und dadurch die Bauten schädigen kann, ist nun als Herausforderung an den Ingenieur zu verstehen.

Bei jeder Bewehrung in Oberflächennähe hat sich der Ingenieur zu überlegen, brauche ich sie aus statischen Gründen; wenn ja, so schütze ich sie durch guten, homogenen Beton. Wenn nein, so ersetze ich die feingliedrige Schwindbewehrung aus Stahl durch ein dreidimensionales Skelett aus Kunststoffasern. Speziell in dünnen Querschnitten, wo Stahlbewehrung das Einbringen des Betons stark behindert, ihn entmischt, ist die Verwendung von Faserbeton besonders vorteilhaft.

3. HERSTELLUNG DES MIT P.P.FASERN VERSTAERKTEN BETONS

Betonmischung:	Kornabstufung, Zementgehalt und Wasserdosierung sind unverändert von ihren Normmischungen zu übernehmen.
Forta-Fibre Fasern:	1 kg Forta-Fibre Fasern auf 1 m^3 Fertigbeton
Betonzusatzmittel:	Alle Betonzusatzmittel (Plastifizierungsmittel, Verflüssiger, Luftporenbildner, Frostschutz etc.) können wie im normalen Beton verwendet werden.
Mischzeit:	Die minimale Mischzeit von Faserbeton beträgt 2 Minuten, die Faserbündel müssen geöffnet sein.

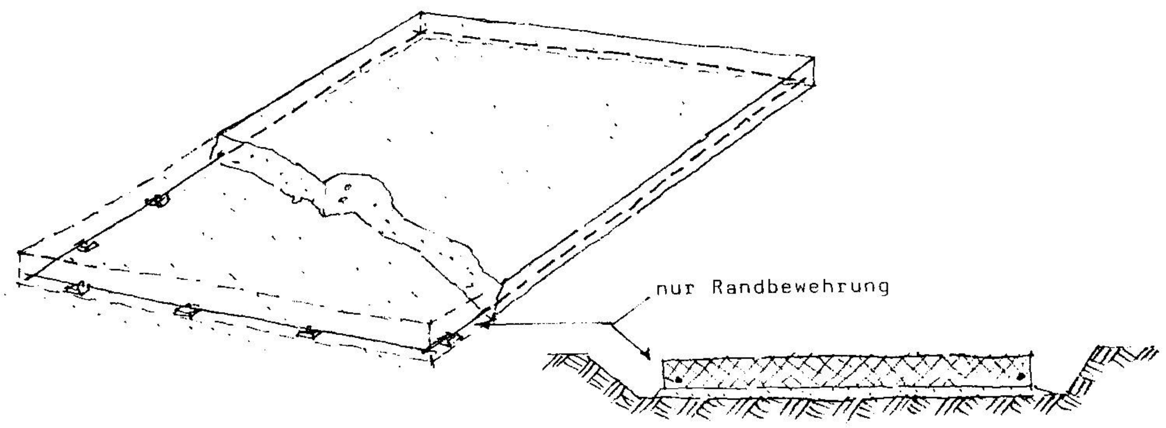
Zugabe der Fasern:	Fasern entweder direkt durch die Reinigungsöffnung nach der Zementzugabe in den Mischer geben oder, was meist einfacher ist, zusammen mit Kies oder Sand in den Waagebehälter schütten.
Empfehlung:	Eine Trockenmischzeit von ca. 30 Sekunden von Kies, Sand, Zement oder Fasern verbessert die Qualität und Verarbeitbarkeit des Betons. Bei Fliessbeton ist dies unerlässlich.
Faserzugabe in Fahrmischer:	Es besteht die Möglichkeit, die Forta-Fibre Fasern im Fahrmischer beizumischen. Dabei gilt die Faustregel: Anzahl m^3 im Fahrmischer ist gleich Anzahl Minuten Mischzeit.
Verstopfung bei Umschlaggerät und Krankübel:	Hier ist es ratsam, den innern Reibungswiderstand mit einem Plastifizierungsmittel zu korrigieren, damit sich bei leichter mechanischer Einflussnahme (z.B. Vibrator) der Beton gut umschlagen lässt (Kombimittel: Ligninsulfonate mit Naphtalinen oder Melaminen, Feststoffanteil 35-45 %, Dosierung 0.2-0.5 % des Zementgewichtes).
Kosten: (zusätzlich zum Anschaffungspreis der Fasern)	Kosten für Magazinieren, Faserzugabe manuell, verlängerte Mischzeit. In der Schweiz können diese Kosten mit Fr. 3.-- bis Fr. 5.--/m^3 abgedeckt werden.

4. PRAKTISCHE ANWENDUNGEN 1974 BIS 1987

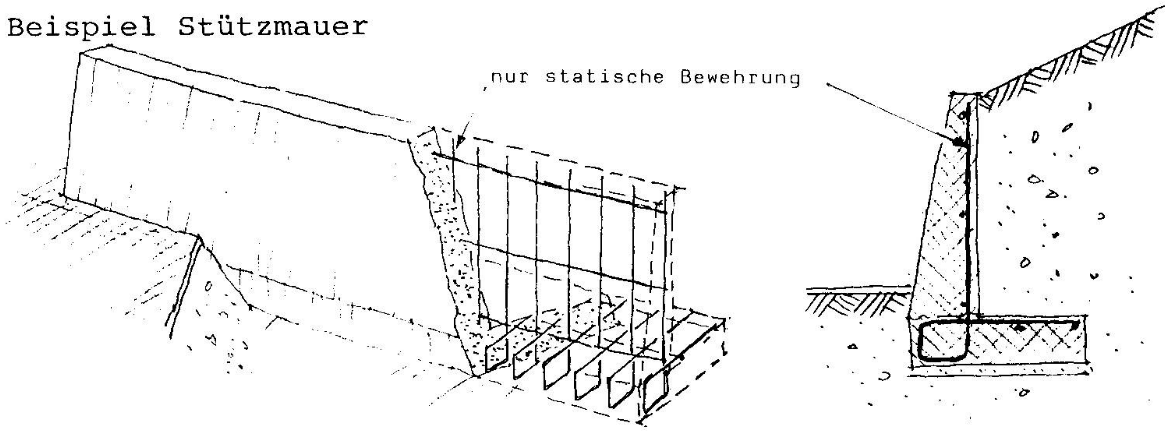
4.1 Ortsbeton

Bodenplatten, Betonstrassen, Schutzbeton auf Böschungen, Isolationen und Abdichtungen, Gefällbeton, Auffangwannen, Ortsbetonkanäle, Stützmauern, Fundamentplatten, Schwimmbäder, Maschinenfundationen, See- und Hafenbauten, Beton in Kontakt mit aggressiven Medien, Beton über Bodenheizungen, Estriche (Unterlagsböden), Ueberzüge.

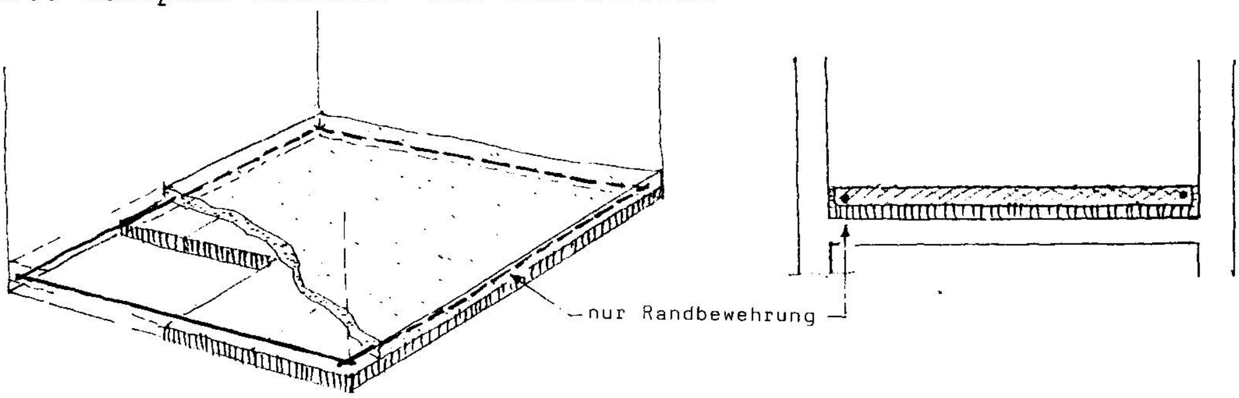
4.1.1 Beispiel Bodenplatte



4.1.2 Beispiel Stützmauer



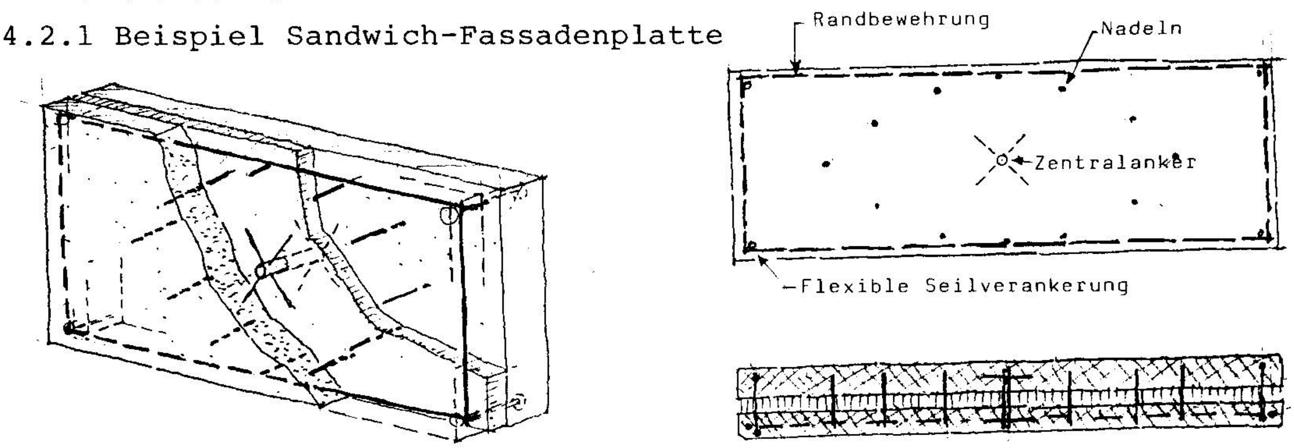
4.1.3 Beispiel Gefälls- und Schutzbeton



4.2 Vorfabrikation

Fassadenelemente, speziell in Sandwichbauweise, Treppen, Brüstungen, Liftschächte, Trafo- und Stromverteilhäuschen, Blumentröge, Brunnentröge, Gartenmobiliar aus Beton, Skulpturen, Restaurationen in Kunstsandstein.

4.2.1 Beispiel Sandwich-Fassadenplatte



4.3 Diverses

Gunnit, Spritzbeton, Verputz, feuerfester Beton

5. WIRTSCHAFTLICHKEITSBETRACHTUNG

In Europa und Amerika sind verschiedene fibrillierte Fasern europäischer und amerikanischer Provenienz auf dem Markt. Die Preise für diejenigen, die eine homogene Mischbarkeit und eine gute Verarbeitbarkeit garantieren, bewegen sich um US$ 16 pro kg. Die harte Konkurrenz wird diese Preise noch nach unten korrigieren. Unter der Annahme, dass eine fertig verlegte Stahlbewehrung, unter Berücksichtigung von Verschnitt, Ueberlappung, zusätzlichen Distanzkörben, Distanzhaltern usw. auf US$ 1.60 pro kg kommt, gilt die Faustregel, dass bei Substitution von 10 kg Stahl pro m^3 Beton Preisgleichheit mit dem Faserbeton besteht. Mit andern Worten, wenn es gelingt, **pro m^3 Beton mehr als 10 kg Schwindrissbewehrung aus Stahl** durch mit Polypropylenfasern verstärkten Beton zu ersetzen, **wird die Baute billiger, die Qualitätsverbesserung und die Arbeitserleichterung ist dann gratis.**

6. KONTROLLIERBARKEIT

Ein nicht zu unterschätzender Faktor im heutigen Baugeschehen ist die sofortige Kontrollierbarkeit. Faserbeton ist jederzeit kontrollierbar, bereits beim Einbringen sehen die Aufsichtsorgane, ob der Beton gut gemischt ist, ob genügend Fasern drin sind und dass er sich nicht entmischt. Die Verarbeitung erfordert keine speziellen Vorkehrungen und ist einfach zu überwachen.

7. SCHLUSSWORT

Ein grosser Förderer des kunststoffaserbewehrten Betons hat im Jahre 1976 an der Internationalen Erfindermesse in Genf die Goldmedaille erhalten. Sicher zu recht, zahlreich sind heute die sinnvollen, wirtschaftlichen Verwendungen dieser Technologie. Die Verwendung von Kunststoffasern im Beton ist bestimmt eine echte Innovation, ein echter Fortschritt im Bestreben, unsere Bauten aus Beton für uns alle wirtschaftlich in bester Qualität herzustellen.

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Bétons à hautes performances par l'emploi de fluidifiants et de fibres d'acier

Hochleistungsbeton durch Verwendung von Verflüssigern und Stahlfasern

High Performance Concretes by the Use of Superplasticizers and Steel Fibres

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RÉSUMÉ

Cette communication présente les méthodes mises au point pour obtenir des bétons à très hautes résistances aisés à mettre en œuvre. Ces méthodes font appel à une introduction fractionnée d'un adjuvant fluidifiant ainsi qu'à l'emploi éventuel d'une microsilice. Les bétons à très faibles teneurs en eau que l'on prépare ainsi présentent un retrait de durcissement très élevé par rapport aux bétons ordinaires. L'emploi de fibres permet de réduire les phénomènes de fissuration pouvant résulter de ce retrait.

ZUSAMMENFASSUNG

Der Beitrag stellt die Methoden vor, die zur Aufbereitung von leicht einbaubaren Hochfestigkeitsbetonen entwickelt werden. Diese Methoden stützen sich auf eine fraktionierte Zugabe eines Betonverflüssigers sowie den eventuellen Einsatz Mikro-Siliziumoxid. Die auf diese weise aufbereiteten Betone mit sehr schwachem Wassergehalt weisen gegenüber den herkömmlichen Betone ein sehr hohes Erstarrungsschwinden auf. Die Verwendung von Stahlfasern ermöglicht eine Senkung der Rissbildungserscheinungen, die aus diesem Schwinden resultieren können.

SUMMARY

This paper presents current methods developed to obtain very high strength concretes that are easy to place. These methods are based on the fractioned addition of a superplasticizer and occasionally, the use of a microsilica. The very-low-water content concretes so prepared exhibit much more hardening shrinkage than ordinary concretes. Fibres may be used to reduce the cracking this shrinkage may cause.

1. INTRODUCTION

Pour obtenir un béton à hautes performances avec des matériaux, ciment et granulats, bien définis, il est indispensable qu'il possède une teneur en eau minimale et un maximum de compacité. Ceci peut être réalisé à l'aide d'adjuvants à pouvoir réducteur d'eau très élevé et d'addition d'éléments ultrafins qui remplissent les vides existant entre les grains de ciment. C'est ainsi que la conjugaison de l'emploi de fumées de silice et de fluidifiant conduit à des bétons dont la résistance à la compression à 28 jours peut être supérieure à 100 MPa.

Toutefois, ces formulations doivent pouvoir être exécutées sur chantier sans faire appel, de préférence, à des moyens exceptionnels. Ceci impose à ces bétons la nécessité de présenter des maniabilités satisfaisantes, à leur sortie de la bétonnière et lors de leur mise en œuvre, c'est-à-dire 20 à 60 minutes après leur confection. Or, il est bien connu qu'un raidissement du béton intervient très rapidement après l'introduction du fluidifiant non retardé.

Nous avons donc étudié le mode d'introduction optimal du fluidifiant permettant d'obtenir, d'une part, un béton ayant un affaissement au cône à la fin de la fabrication d'au moins 4 cm et, d'autre part, un béton fluide (affaissement > 20 cm) jusqu'à 60 ou 90 minutes après fabrication.

Par ailleurs, le critère de haute résistance ne suffit pas à conditionner la qualité du béton, il faut aussi que les autres caractéristiques soient très performantes. De nombreux travaux (1) ont montré que l'aptitude à la fissuration du béton à hautes résistances est élevée. Nous avons donc analysé ce phénomène et cherché à réduire cette fissuration en introduisant dans la composition des fibres métalliques.

2. FORMULATION DE BETON A HAUTES PERFORMANCES ET A MANIABILITE MAXIMALE

2.1. Béton traditionnel

Les fluidifiants présentent une efficacité optimale (2) lorsqu'ils sont introduits entre 20 et 30 minutes après le mélange, que nous appellerons primaire, des granulats, du ciment et de l'eau. Cette propriété permet d'obtenir avec un béton à rapport eau/ciment faible (entre 0,26 et 0,35) des affaissements de 22 cm. Toutefois ces mélanges primaires présentent lors de leur fabrication des affaissements nuls et sont irréalisables en centrale. Une étude approfondie du mode d'introduction optimal des fluidifiants nous a permis de préconiser l'introduction du fluidifiant en deux temps, c'est-à-dire : une fraction dans l'eau de gâchage ou juste à la fin du malaxage du mélange primaire et la fraction restante environ 30 minutes après. La figure 1 résume les résultats obtenus avec cinq modes différents d'introduction et trois dosages différents d'extrait sec d'un fludifiant à base de résine mélamine formaldéhyde. On constate ainsi que le dosage de 0,7 %, introduit à raison de 0,48 % dans l'eau de gâchage et 0,22 % 30 minutes après la fin du mélange primaire, conduit, pour un rapport eau/ciment de 0,33, à un affaissement de 22 cm qui se maintient pendant 90 minutes.

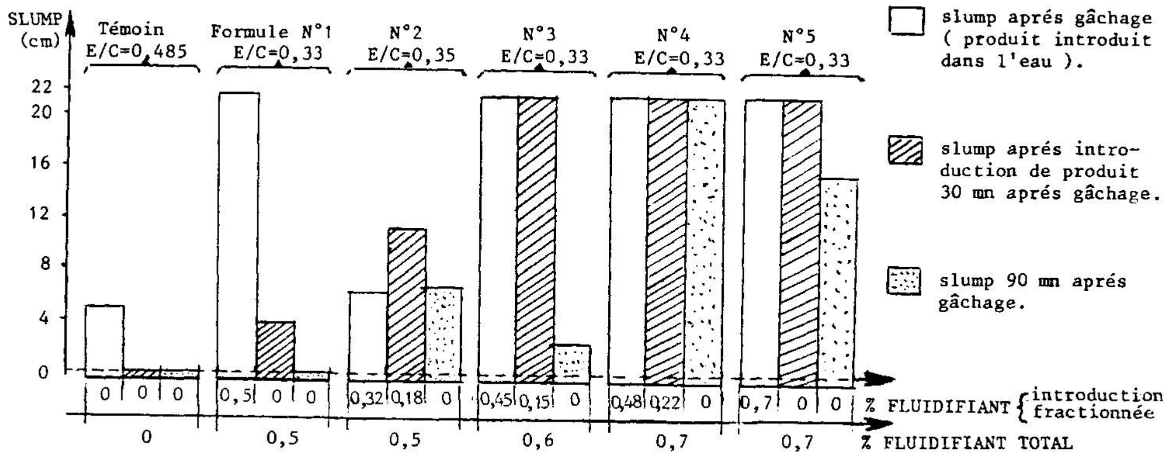
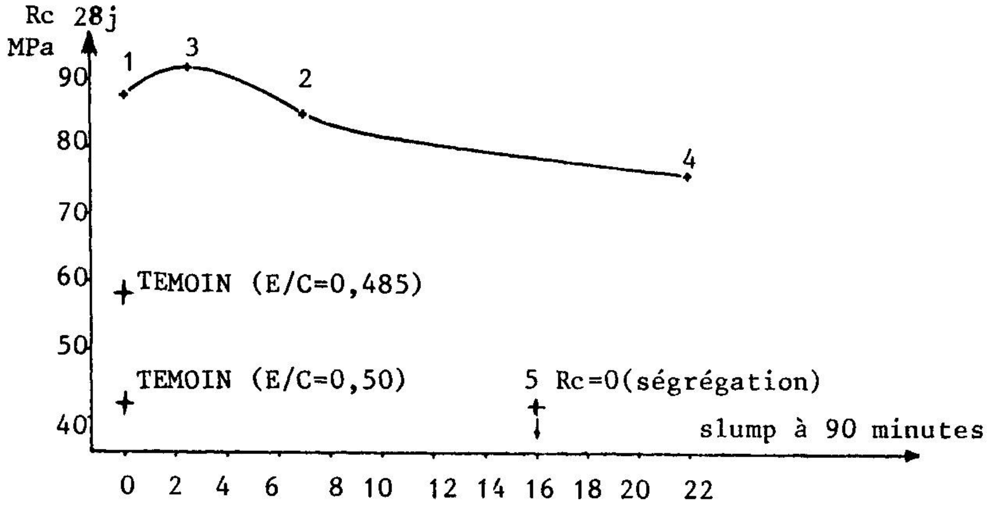


Figure 1 : Affaissement au cône d'Abrams (slump) d'un béton fluidifié en fonction du dosage en fluidifiant, de son mode d'introduction et du temps.



Du point de vue des résistances, la figure 2 montre que, par la seule réduction d'eau, obtenue avec l'introduction fractionnée du fluidifiant, on obtient des résistances à la compression à 28 j comprises entre 76 et 92 MPa. Les résistances les plus élevées se placent dans le cas où l'affaissement de 22 cm n'est maintenu que pendant 60 minutes (formule 1 et 3).

Figure 2 : Résistances à la compression à 28 jours en fonction de la valeur de l'affaissement au cône d'Abrams 90 mm après fabrication du béton

2.2. Double malaxage

Le tableau n° 1 montre que la formule 4, à affaissement maximum, peut être améliorée en préparant séparément un coulis avec l'eau, le ciment et la première fraction du fluidifiant, puis en mélangeant ce coulis avec les granulats du béton.

TABLEAU 1
Caractéristiques du béton à hautes performances confectionné avec la méthode de double malaxage

	% total de fluidifiant (extrait sec)	Introduction fractionnée % de fluidifiant dans chaque phase		E/C	Slump à 90 min. en cm	R. compres. à 28 j. en MPa
Témoin formule 4 de la fig. 1 Malaxage traditionnel	0.7	0.48 à la fin du malaxage	0.22 30 min. après fabrication	0.33	22	76.0
Double malaxage <u>à haute turbulence</u> : Coulis (eau + ciment + adjuvant) mélangé après fabrication aux granulats du béton	0.7	0.48 dans le coulis	0.22 dans le béton 30 min. après confection	0.33	22	85.0
Coulis (70 % de l'eau totale + 66 % du ciment + adjuvant) mélangé après confection aux granulats + 30 % de l'eau + 34 % du ciment	0.7	0.48 dans le coulis	0.22 dans le béton 30 min après confection	0.35	22	88.5

2.3. Application au béton à très hautes performances avec fumées de silice

Compte tenu des résultats des travaux précédents nous avons adapté la méthode d'introduction fractionnée du fluidifiant au béton avec fumée de silice. Ainsi, un tiers du dosage total a été incorporé 30 s après le mélange granulats-ciment-fumées de silice, ce qui a conduit à un affaissement de 20 cm, puis les deux tiers restant ont été mélangés au béton après 4 min. d'attente. L'affaissement au cône obtenu était de 22 cm environ pour atteindre entre 30 et 40 min. après, 18 à 20 cm.

TABLEAU 2
Composition du béton avec fumées de silice

Gravillon	Sable	Ciment CPA 55	Fumée de silice (% ciment)	Fluidifiant (% ciment)	Eau totale (% ciment)	Slump fin de malaxage	Slump après 40 mm d'attente
1 265 kg/m³	652 kg/m³	421 kg/m³	10 %	1,8 %	26,7	20 cm	20 cm

TABLEAU 3
Caractéristiques physiques et mécaniques du béton avec fumées de silice

	1 j	3 j	7 j	14 j	28 j	90 j
Résistance en compression (MPa)	27	72	86	93	101	110
Résistance au fendage (MPa)	2,2	5,4	6,4	6,1	6,5	-
Module d'Young (GPa)	35	49	51	52	53	54

Les tableaux n° 2 et 3 donnent la composition et les résistances de ce béton. On obtient ainsi un béton à très hautes performances (100 MPa) qui présente entre 3 et 14 jours une montée en résistance très rapide.

3. CONTRIBUTION DES FIBRES METALLIQUES A LA DIMINUTION DE L'APTITUDE A LA FISSURATION DU BETON A TRES HAUTES RESISTANCES

Les réductions exceptionnelles de porosité et de perméabilité que l'on atteint dans les bétons à hautes performances conduisent à une amélioration remarquable du comportement de ces bétons, par rapport aux bétons ordinaires, face aux mécanismes traditionnels de dégradation par migration d'éléments nocifs dans le réseau poreux du matériau.

Par contre, on constate que dans le cas où ces bétons sont soumis à des conditions de déformation empêchée dès le coulage, il convient de prendre garde à leur retrait d'hydratation. Ce retrait est principalement engendré par l'« autodessiccation » du béton au cours de son durcissement (3), c'est-à-dire par la diminution spontanée progressive de l'humidité relative en équilibre avec l'eau interne du béton, protégé de toute évaporation, sous l'effet de l'hydratation du ciment.

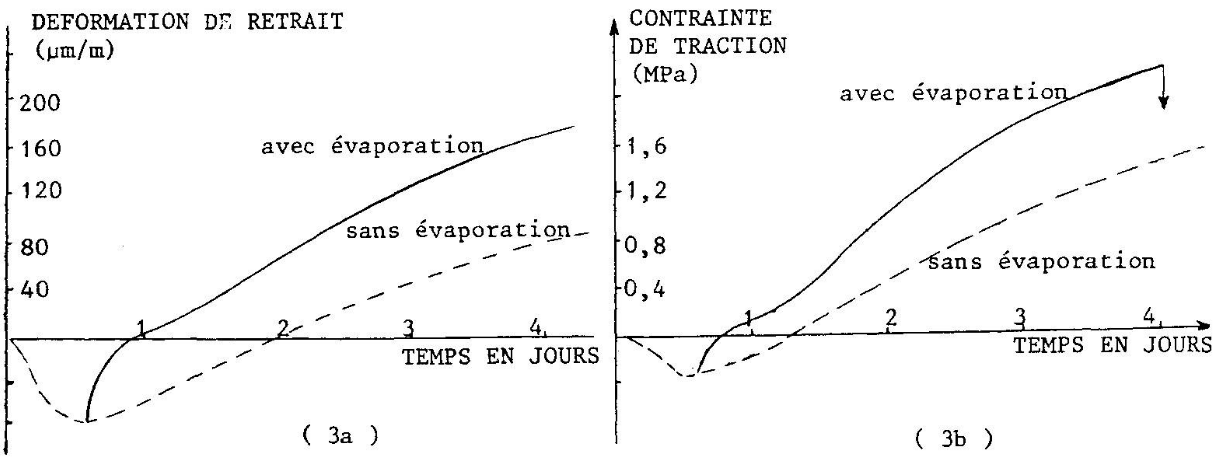


Figure 3 : Evolution des déformations libres et des contraintes dans l'essai au banc de fissuration pour un béton traditionnel avec évaporation et sans évaporation.

Les essais effectués avec le banc linéaire LCPC de fissuration du béton (4) ont mis en évidence l'influence de ces phénomènes sur l'aptitude à la fissuration du béton avec fumées de silice. Le principe du banc de fissuration est le suivant : il enregistre, dès la mise en place du béton frais dans le moule, les différentes déformations (expansion et retrait) engendrées dans le béton au cours de son durcissement. D'autre part, il permet de mesurer les contraintes engendrées dans l'éprouvette lorsque celle-ci est maintenue à une longueur constante (déformations empêchées) à l'aide d'un système d'asservissement. Ces contraintes conduisent à la rupture du béton par traction, après un certain temps qui caractérise l'aptitude à la fissuration de celui-ci.

La figure 3 présente l'influence du retrait d'hydratation et d'évaporation sur la fissuration d'un béton traditionnel (rapport eau/ciment = 0,44, 425 kg de ciment par m^3). L'évolution des contraintes et des déformations obtenues avec un béton à très hautes performances avec fumées de silice (E/C = 0,26, 425 kg de ciment par m^3 et 64 kg de fumées de silice par m^3) est représentée dans la figure 4. On constate que l'éprouvette soumise à l'évaporation fissure immédiatement après démoulage à 14 h et que l'éprouvette protégée de toute évaporation est rompue au bout de 4 jours sous une contrainte de traction supérieure à 3,5 MPa (fig. 4b). Dans les mêmes conditions expérimentales, le béton traditionnel engendre des contraintes beaucoup plus faibles et l'éprouvette protégée ne fissure pas jusqu'à 28 jours, échéance à laquelle les essais ont été arrêtés (fig. 3b).

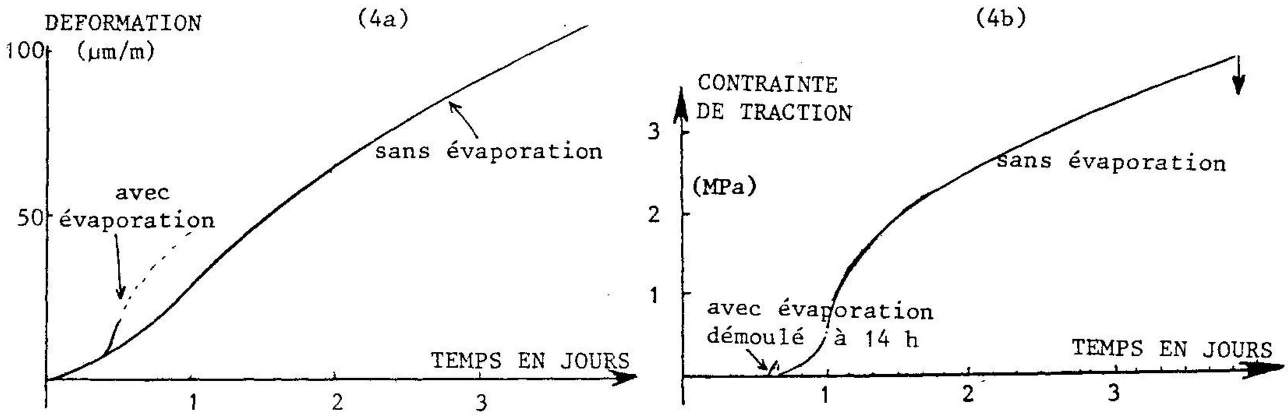


Figure 4 : Evolution des déformations libres et des contraintes dans l'essai au banc de fissuration pour un béton à très hautes performances avec fumées de silice, avec évaporation et sans évaporation.

Ces résultats sont en accord avec la très forte autodessiccation qui a pu être mesurée sur le béton de fumées de silice de cette étude. La figure 5 montre que l'humidité interne d'un tel béton est abaissée à 75 % au bout d'un mois, alors qu'un béton traditionnel à E/C = 0,44 reste pratiquement sous humidité saturante pendant ce même temps (5). Ainsi donc, pour les très faibles rapports eau/ciment (par ex. 0.25) atteints dans les bétons à très hautes performances, l'autodessiccation n'est plus un phénomène négligeable. Il en résulte un retrait d'hydratation élevé qui peut engendrer une fissuration précoce dans le cas d'une déformation empêchée.

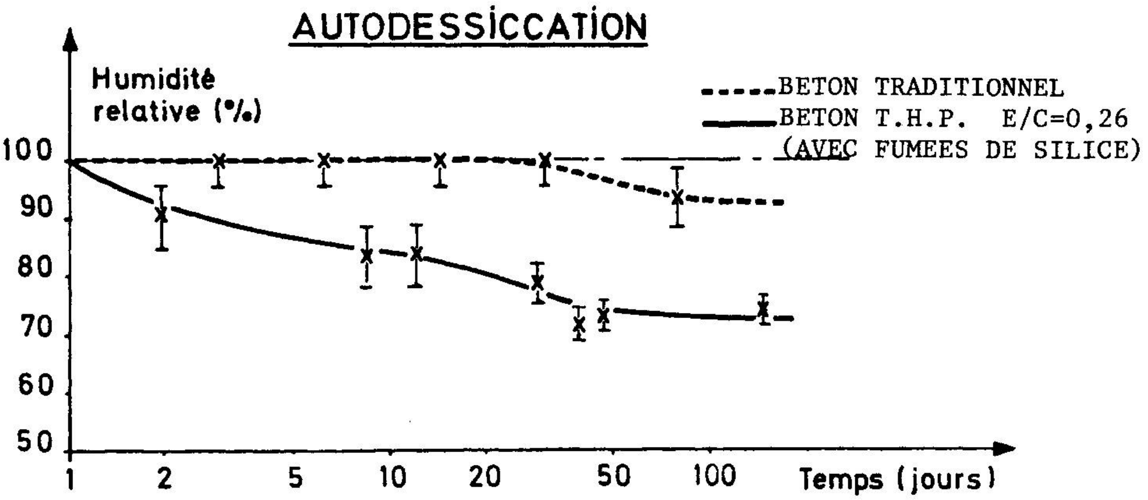
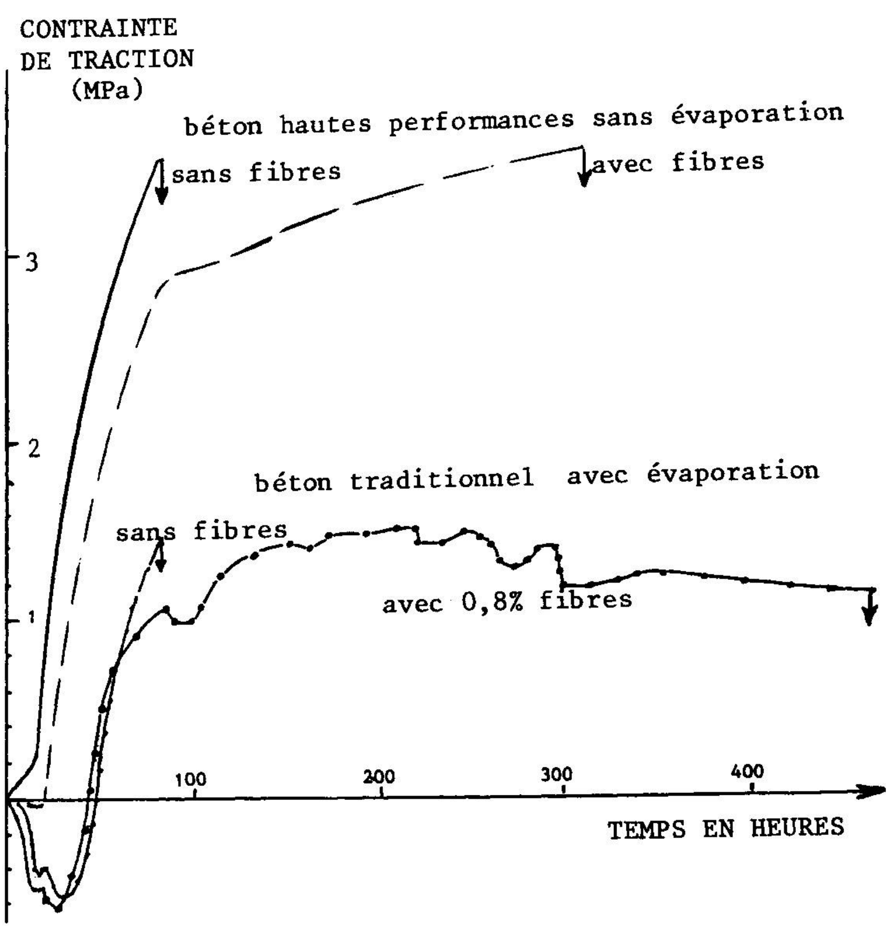


Figure 5 : Evolution de l'humidité interne avec le temps d'hydratation pour un béton traditionnel et pour un béton à très hautes performances avec fumées de silice (éprouvettes protégées de l'évaporation).



L'addition de fibres à la composition des bétons à très hautes performances permet de réduire ce phénomène de fissuration franche précoce, en favorisant la création d'une microfissuration répartie et en couturant les microfissures dès leurs apparitions.

La figure 6 montre le comportement du béton à très hautes résistances avec fumées de silice et une addition de 0,8 % de fibres d'acier 50/50 (rapport de la longueur en mm sur le diamètre en centièmes de mm). La fissuration, en absence de toute évaporation, est retardée de 7 jours. Ceci montre l'importance du phénomène d'autodessiccation, puisque l'amélioration apportée par les fibres introduites dans un béton traditionnel, soumis à l'évaporation, correspond à un retard à la fissuration de l'ordre de 10 à 15 jours.

Figure 6 : Evolution des contraintes dans l'essai au banc de fissuration pour un béton à très hautes performances avec fumées de silice, protégé de l'évaporation, sans fibres ou avec fibres et pour un béton traditionnel, soumis à l'évaporation, sans fibres ou avec fibres.

4. CONCLUSIONS

L'ensemble de ces résultats de recherche montre que l'introduction fractionnée d'un fluidifiant permet de confectionner et de mettre en œuvre, dans des conditions satisfaisantes, des bétons à teneurs en eau extrêmement basses. On peut obtenir par ce procédé des bétons à très hautes résistances (de l'ordre de 80 MPa). La combinaison de ce mode d'introduction avec l'addition de fumées de silice, permet d'atteindre des niveaux de résistance de 100 MPa. Par ailleurs, nous avons mis en évidence un éventuel aspect pathologique de ces bétons : leur fissuration précoce par retrait de durcissement exceptionnellement élevé, qui peut intervenir lorsque ce retrait est empêché dès le début de l'hydratation du ciment. L'emploi de fibres métalliques permet de limiter cette fissuration.

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Properties and Performance of High-Strength Concrete

Propriétés et performances du béton à haute résistance

Eigenschaften und Verhalten von hochfestem Beton

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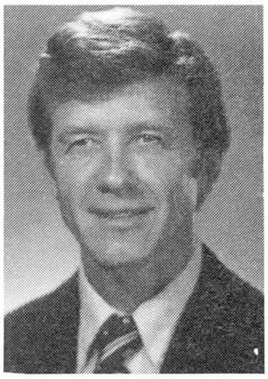

The author received his BS degree from Stanford University, MS from Cornell, and PhD from the University of California. Prior to joining the faculty at Cornell he was engaged in professional practice. He is the author of two texts on concrete design, as well as numerous technical papers.

SUMMARY
This paper is a summary of certain results from a 10-year program of research on high-strength concrete which had the following objectives : to establish the fundamental nature of the material ; to establish its engineering properties and to relate these to differences in internal response ; and to study the behavior of structural members made using high-strength concrete.

RÉSUMÉ
Cet exposé résume certains résultats d'un programme de dix ans de recherche sur le béton à haute résistance dont les objectifs étaient : d'établir la nature fondamentale du matériau ; d'établir les propriétés mécaniques et d'établir le rapport entre les différentes réponses intérieures ; et d'étudier le comportement des éléments en béton à haute résistance.

ZUSAMMENFASSUNG
Der Beitrag gibt eine Uebersicht über die Ergebnisse aus einem zehnjährigen Forschungsprogramm über hochfesten Beton, welches folgende Zielsetzungen hatte : die Erforschung der fundamentalen Beschaffenheit des Materials, das Feststellen der mechanischen Eigenschaften hochfester Betone und das Studium des Verhaltens von Stahlbetonbauteilen aus hochfestem Beton. Es werden vor allem die ersten beiden Aspekten, welche die Materialeigenschaften behandeln, besprochen.

1. INTRODUCTION

The past decade has seen a rapid growth of interest in high-strength concrete, with compressive strength in the range from about 40 to 85 MPa. Concretes in this strength range can be produced economically using carefully selected, but commonly available, cement, sand, and stone, through use of very low water-cement ratios and careful production control. Workability is achieved by high-range water-reducing admixtures, the so-called super-plasticizers.

Remarkably, use of high-strength concrete has preceded full knowledge of its properties, or of the behavior of structural members made using it. Much of our design methodology, including many design equations, is based on tests of members for which the material strengths were less than about 40 MPa.

In an effort to provide the data needed for design, an intensive program of research was initiated at Cornell University with three objectives: (a) to establish the fundamental nature of the material, (b) to establish the engineering properties, and (c) to study the behavior of reinforced and prestressed concrete members using high-strength concrete. Results summarized in this paper relate mainly to parts (a) and (b) of the investigation.

2. INTERNAL RESPONSE TO LOADS

2.1 Microcracking Under Short-Term Compression

It is generally recognized that many characteristics of concrete can be explained by progressive internal microcracking that occurs as load is gradually increased. Microcracking results mainly from the differences in stiffness and strength between the mortar and the stone. Bond cracks start on the interface between the mortar and aggregate. As load is increased, cracking spreads through the mortar, leading to an interconnected network of cracks, discontinuity of the material and, eventually, failure.

Significant differences were found between low- and high-strength concrete [1]. Microcracking starts at about 35 percent of ultimate load for low-strength concrete, and, at loads above about 65 percent, the interconnected crack pattern is well established. For high-strength concrete, there is little or no microcracking at low loads. Bond cracks start between about 65 and 80 percent, and even at 90 percent of ultimate load the bond cracks are mostly isolated, not interconnected.

A typical fracture surface of a compression cylinder of low strength concrete is rough and rugged. Cracks follow around the stone inclusions, then branch through the mortar. There is substantial energy dissipation associated with development of such a surface. For high-strength concrete, the typical failure surface is a clean fracture plane, as in Fig. 1. with cracks passing through stone and mortar without bias.

2.2 Microcracking Under Sustained Compression

Other studies have shown that sustained load behavior can also be related to differences in internal microcracking [2]. Sustained-load compression tests were of two types: (a) creep tests at loads from 40 to 80 percent of the short-term failure load, and (b) time-dependent failure tests at loads from 75 to 95 percent of short-term strength.

For sustained loading, three distinct stages of microcrack development were identified, associated with: (1) Linear creep, for which creep strain is proportional to stress and elastic strain; this is the range for which the usual creep coefficient applies; (2) Nonlinear creep, for which creep strains are disproportionately larger than creep coefficient times elastic strain, the ratio increasing with increasing load; and (3) Failure under high sustained load less than the short-term strength. Differences between low-strength and high-strength concrete behavior under sustained load are summarized in Table 1 [3].

Stage	Stress as Percent of Short-Term Strength		Microcracking
	Low-strength	High strength	
Linear creep	to 45%	to 65%	Some increase in bond cracks for low-strength; negligible for high
Nonlinear creep	to 75%	to 85%	Bond cracking increases for both
Failure	above 75%	above 85%	Bond cracks, mortar cracks, and combined cracks increase sharply

Table 1 Sustained load microcracking

3. ENGINEERING PROPERTIES

3.1 Compressive Stress-Strain Curve

Compressive stress-strain curves are shown in Fig. 2. The range of approximately linear elastic behavior is extended to 80 to 90 percent of maximum stress. Strain at maximum stress is about 0.002 for normal concrete, but increases to about 0.003 for high-strength specimens. The maximum strain reached is less [4]. While the shape of the compressive stress-strain curve after the peak stress is reached is highly dependent on testing methods, typically, for high strength concrete, there is a rapid dropoff of stress after the peak. The long descending branch displayed for normal concrete, corresponding to the gradual development and spread of microcracking, is absent.

3.2 Static Modulus of Elasticity

It was found that the ACI Code equation for elastic modulus E_c overestimated by as much as 20 percent. Modified predictor equations were established for both normal-weight and lightweight concrete [4,5].

3.3 Poisson's Ratio

Poisson's ratio in compression ranged from 0.15 to 0.26. The average value was essentially 0.20 regardless of compressive strength, curing conditions, or test age [4,5].

3.4 Tensile Strength

The two measures of tensile strength used in U.S. design practice are modulus of rupture f_r and the split cylinder strength f_{ct}. Data was obtained for each, for both normal and lightweight concrete, and is summarized in Table 2 [4,5]. Values stated in the ACI Code are shown for comparison.

Type of Concrete	Predicted Values of f_r		Predicted Values of f_{ct}	
	Cornell	ACI	Cornell	ACI
Normal-weight moist cured	$0.90\sqrt{f'_c}$	$0.63\sqrt{f'_c}$	$0.68\sqrt{f'_c}$	$0.54\sqrt{f'_c}$
Sand-lightweight	---	$0.53\sqrt{f'_c}$	---	$0.48\sqrt{f'_c}$
All lightweight moist cured	$0.66\sqrt{f'_c}$	$0.47\sqrt{f'_c}$	$0.51\sqrt{f'_c}$	$0.42\sqrt{f'_c}$
All lightweight dry cured	$0.36\sqrt{f'_c}$	$0.47\sqrt{f'_c}$	$0.42\sqrt{f'_c}$	$0.42\sqrt{f'_c}$

Table 2 Modulus of rupture f_r and split cylinder strength f_{ct} (all units MPa)

3.5 Creep Coefficient

One of the most significant differences between normal-strength and high-strength concrete is the greatly reduced creep coefficient, C_{cu}. Coefficients given for high strength concrete in Table 3 were obtained by extrapolating from tests of 6 month duration or less [3,6], but appear reasonable as an extension of 5 year data for lower strengths.

Material	f'_c MPa	C_{cu}	$C_{cu}/C_{cu,low}$
Low-strength concrete	21	3.1	1.00
Medium-strength concrete	28	2.9	0.94
" " "	41	2.4	0.77
High-strength concrete	55	2.0	0.65
" " "	69	1.6	0.52

Table 3 Creep Coefficients (adapted from Ref. 2 and 3)

3.6 Sustained Load Strength

It is well known that the strength of concrete under sustained loading is less than that determined by short-time loading. Based on earlier studies, the load that will produce failure if sustained over a period of time was thought to be related to the discontinuity stress. Because this is higher for high-strength concrete, it was expected that the sustained load strength would be higher.

This is true as illustrated by Fig. 4. For low-strength concrete, loads below about 75 percent of short-term strength could be sustained without failure, while loads above that level produced failure. For high-strength concrete, loads as high as 85 percent of short-term strength could be sustained at least for 60 days [3].

4. MEMBER BEHAVIOR

With differences in material behavior that were very significant in some respects, it was expected that there would be important differences in the behavior of members made using high-strength concrete. Some matters of particular concern included: (1) Differences in shape of the compressive stress-strain curve brought into question the validity of the equivalent rectangular stress block used for beam and column strength calculations; (2) The smaller compressive strain limit in axial compression tests required investigation of the validity of the assumption of ultimate flexural strain of 0.003 normally used in U.S. design; (3) Beam deflection ductility could be significantly less because of the more brittle nature of high-strength concrete; (4) Short-term beam deflections would be incorrectly predicted unless a more accurate equation for E_c were used; (5) Time-dependent beam deflections would be greatly over-predicted by present design methods that do not recognize the much lower creep coeffcients; (6) Predictions of beam shear strength might be unsafe for high-strength concrete beams because of the lack of aggregate interlock across diagonal cracks, a result of the typically smooth fracture surfaces.

These concerns led to the third stage of the investigation, involving tests of flexure-critical and shear-critical reinforced concrete beams [7,8], shear-critical prestressed concrete beams [9], reinforced concrete beams under sustained loading [10], and axially loaded columns [11]. A summary review of design implications is presented in Ref. 12.

5. CONCLUSION

The essential fact that has become clear is that high strength concrete is in many respects a new material, in most ways greatly superior to normal concrete, but with special characteristics that require careful consideration. Extrapolation of empirically-based design equations such as are found in all national codes cannot be considered safe practice. A thorough review of these codes, in the light of newly-available information, is essential.

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F' Path of cracks

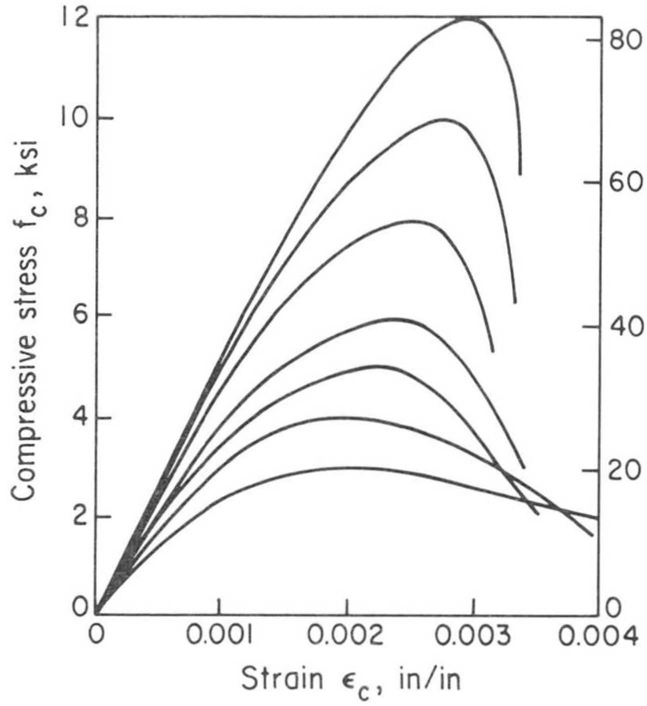

Fig. 2 Typical compressive stress-strain curves

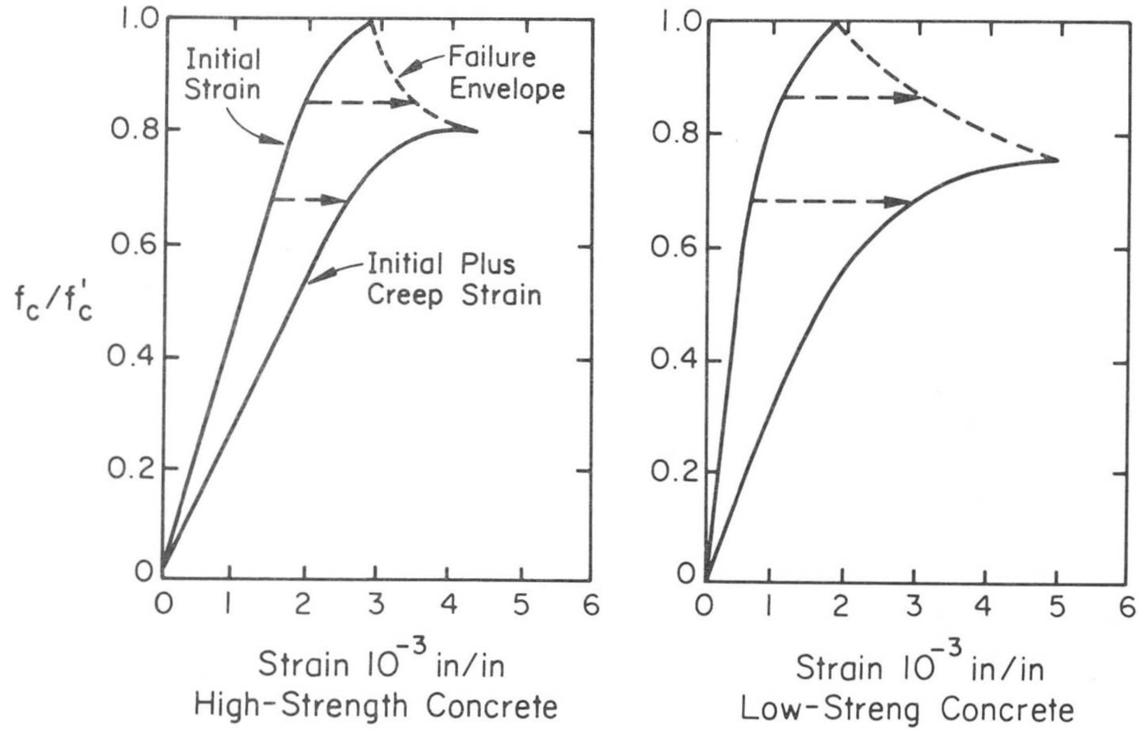

Fig. 3 Sustained load stress strain curves

Mechanical Properties of High Strength Concrete

Propriétés mécaniques du béton à haute résistance

Mechanische Eigenschaften vom hochfesten Beton

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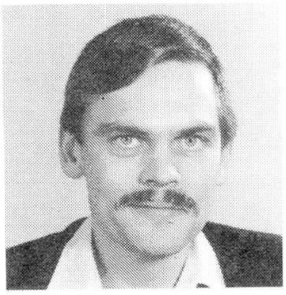

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SUMMARY
This report deals with experimental data on high strength concrete. Compressive stress-strain relationship, ductility, lap splice strength, and fire resistance are discussed.

RÉSUMÉ
Des résultats de quelques essais sur béton à haute résistance sont présentés. Les relations contrainte de compression / déformation, ductilité, résistance des recouvrements d'acier et résistance au feu sont discutés.

ZUSAMMENFASSUNG
Ergebnisse von Versuchen mit hochfestem Beton werden vorgestellt. Das Spannungs-Dehnungsverhalten die Duktilität, die notwendigen Ueberdeckungslängen bei Bewehrungsstössen und der Feuerwiderstand werden diskutiert.

1. STRESS-STRAIN RELATIONSHIP IN UNIAXIAL COMPRESSION

It is generally recognized that for high strength concrete the shape of the ascending part of the stress-strain curve is more linear and steeper, the strain at maximum stress is slightly higher, and the slope of the descending part is steeper than for normal strength concrete.

Extremes of the stress-strain curves were obtained by Tomaszewicz [2] and by Wang et al [1]. Tomaszewicz investigated high strength concrete made with silica fume as additive. The descending part of the stress-strain curve was obtained by using a closed-loop testing machine so that the specimen could be loaded to a constant rate of strain increase avoiding unstable failure. Tomaszewicz represented the stress-strain curves mathematically as:

$$\sigma = f_c \cdot \frac{\varepsilon}{\varepsilon_{co}} \cdot \frac{n}{n - 1 + (\frac{\varepsilon}{\varepsilon_{co}})^{k \cdot n}}$$

$$n = \frac{8.32}{8.32 - f_c^{0.475}}$$

$$k = \begin{cases} \frac{f_c}{20} & \varepsilon > \varepsilon_{co} \\ 1 & 0 \leqq \varepsilon \leqq \varepsilon_{co} \end{cases}$$

$$\varepsilon_{co} = 0.0007 \cdot f_c^{0.31}$$

in which f_c is the compressive strength in MPa and ε_{co} is the strain at peak stress. Wang et al [1] used a simple method of obtaining a stable descending part of the stress-strain curve by loading the concrete cylinders in parallel with a concentrically placed large diameter, hardened steel tube with such a wall thickness that the total load exerted by the testing machine always increased. Wang et al represented the stress-strain curve mathematically by the equation

$$\sigma = f_c \cdot \frac{A \cdot (\frac{\varepsilon}{\varepsilon_{co}}) + B \cdot (\frac{\varepsilon}{\varepsilon_{co}})^2}{1 + C \cdot (\frac{\varepsilon}{\varepsilon_{co}}) + D \cdot (\frac{\varepsilon}{\varepsilon_{co}})^2}$$

in which f_c is the compressive strength in MPa, ε_{co} is the strain at peak stress, and A, B, C, and D are constants.

Two different sets of constants were used for the ascending and the descending parts of the curve. From Wang et al [1] details of the constants and of ε_{co} can be found.

Fig. 1 shows the test results obtained by Tomaszewicz and Wang et al. It can be seen that the slope of the curve in the post maximum stress range becomes steeper as the compressive strength of the concrete increases. For the same peak stress the shape of the ascending and especially the descending part of the curves from Tomaszewicz's investigation are steeper than those of Wang et al's investigations.

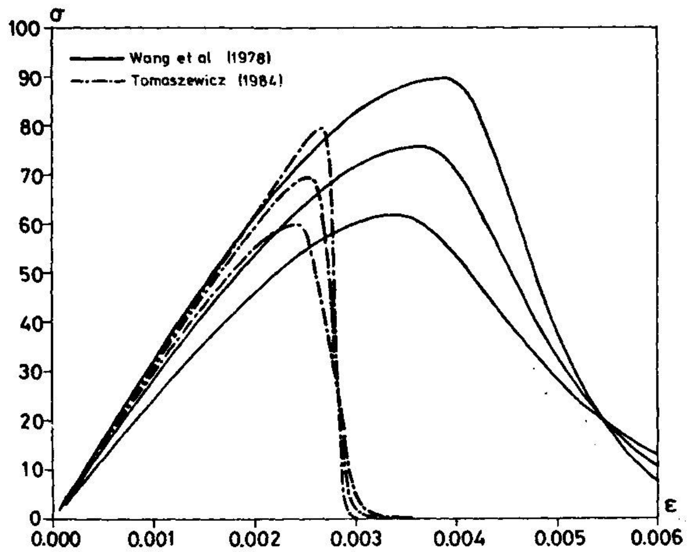

Fig. 1: Stress-strain curves from uniaxial compression tests.

2. DUCTILITY

High strength concrete is less ductile than normal strength concrete. It is not possible to express the relative ductility (or brittleness) in a quantitative manner, since no rational standard method of measuring this quantity currently exists. Attempts using nonlinear fracture mechanics to define fracture toughness are being made.

Ductility can be quantitatively expressed by the slope of the post-peak response of concrete subjected to uniaxial compression. If this slope is zero for instance, then the material is perfectly plastic, while for perfectly brittle material, the slope is infinite. Fig. 1 shows that the slope increases with increasing concrete strength, especially the types of high strength concrete reported by Tomaszewicz [2].

According to the above definition, high strength concrete is more brittle than normal strength concrete; however the same is not necessarily true for reinforced high strength concrete as compared to reinforced normal strength concrete.

The deflection ductility index for reinforced concrete beams will be defined as:

$$\mu = \Delta_u/\Delta_y$$

where

Δ_u = mid-span beam deflection at failure load

Δ_y = mid-span beam deflection at the local load producing yield of the tensile reinforcement.

This ratio depends not only on the compressive stress-strain curve of the concrete but also on the amount of longitudinal reinforcement, the shape of the beam cross section, the loading conditions and other factors.

The effect of the concrete compressive strength on the deflection ductility of a reinforced concrete beam under third-point loading was theoretically calculated by Ahmed and Shah [3] for three reinforcement ratios and five compressive strength levels. The amount of tensile reinforcement was varied so that the ratio between the actual steel content, ρ, and the balanced steel content, ρ_b (defined and calculated according to the ACI

Code [5]) remained essentially the same for the beams with the five different concrete strengths.

Ahmad and Shah [3] compared the theoretically calculated deflection ductility with experimental research results conducted at Cornell University [4]. They found that the theoretical prediction was close to the experimentally observed values.

Fig. 2. shows the theoretically calculated deflection ductility values [3] and the experimentally determined values [6]. The experimentally determined values are for high strength concrete containing silica fume (Si/C < 0.15). These beams were third point loaded and included compressive reinforcement and lateral confinement steel in the form of closed stirrups.

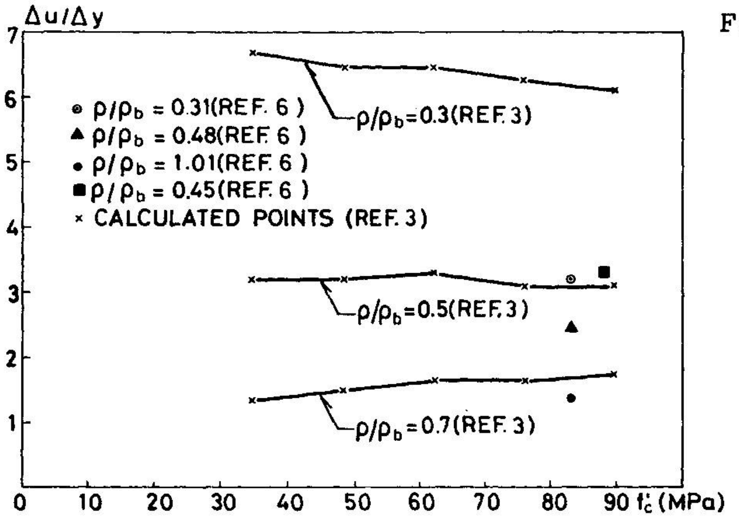

Fig. 2: Relationship between deflection ductility and compressive concrete strength.

3. SPLICES

Although some information regarding development length and anchorage of tensile steel has recently become available for high strength concrete, not enough data have been reported at the moment.

Tepfers [7] has investigated the effect of concrete strength on the lap splice strength. Fig. 3 shows the measured relationship between concrete compressive cube strength, σ_{cube}, and the splice strength represented by the ultimate tensile stress, σ_{su}, in the reinforcement just outside the splice. The tensile reinforcement used was Swedish deformed bars, ø 16 mm with yield stresses of 60 and 90 MPa and a splice length of 520 mm. The concrete cover in the vertical direction was 16-24 mm and in the horizontal direction 26-37 mm. No stirrups were used.

Fig. 3 shows that the splice strength increases with increasing concrete strength up to $\sigma_{cube} \sim 70$ MPa. For larger values of the concrete strength the opposite is the case. Tepfers explains this by the shrinkage of the concrete. The shrinkage creates concrete tensile stresses (hoop stresses) around the reinforcing bars. These stresses increase the tendency to splitting of the concrete. Shrinkage increases with increasing amount of cement. Tepfers's concrete with $\sigma_{cube} \sim 110$ MPa contained 1693 kg cement per m^3 concrete.

Tepfers's tests suggest that the high strength should preferably be obtained - not by a high cement content - but by other means, for instance, by use of silica fume, fly ash and/or superplasticizers.

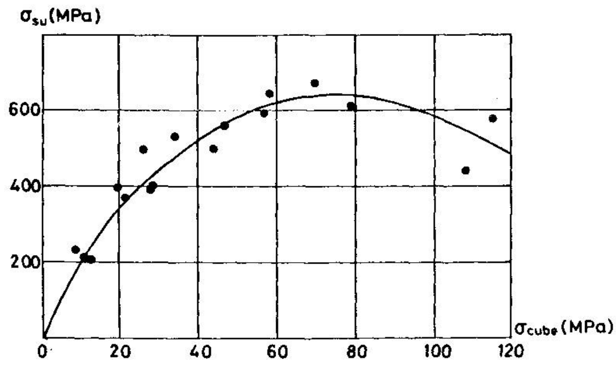

Fig. 3: Relationship between compressive concrete strength and splice strength represented by σ_{su}.

4. FIRE RESISTANCE

When producing high strength concrete by using superplasticizers and silica fume the concrete gets a very low permability. Due to the dense microstructure high strength concrete is far more resistant to many physical and chemical influences than normal strength concrete. However, damage may occur from the internal steam pressure build up when the high strength concrete is heated during a fire.

Hertz [8] investigated the lack of fire resistance by heating (in an electrical oven) concrete cylinders with a compressive strength level of 150-170 MPa. He concluded that high strength concrete possesses a high risk of damage due to steam pressure and the low permability, even at a low heating rate of 1° C per minute. The damage ratio was 67% and the silica fume content was 20% of the cement by weight. Cement content was 500 kg per m^3 concrete.

Recently high strength concrete cylinders were tested at the Technical University of Denmark in order to study the lack of fire resistance for concretes with a lower strength than those tested by Hertz.

Three series of ø 100 mm by 200 mm cylinders were made with intended compressive strengths of 50, 70 and 90 MPa. Cement content was 250 kg, 300 kg and 350 kg, respectively, and the silica fume content was 10% of the cement by weight. The cylinders were cured in two different ways: a) 7 days in water and then 21 days in the laboratory atmosphere (20° C and 60% RH), b) 7 days in water and then sealed for 21 days with plastic-aluminum foil.

All test pieces were heated in an electrical oven at a rate of 2.5° C per minute to a temperature of 600° C, which was maintained for 2 hours. They were then cooled down at a rate of maximum 1° C per minute. Figs. 4a and 4b show the measured compressive strength and the damage percentage for cylinders cured under condition a and b, respectively. Each point in the figures represents the average of three cylinders. (Damaged cylinders had totally lost their integrety).

It appears from figs 4a and 4b that although the heating rate was higher than in Hertz's experiments, the tendency to damage is moderate especially for the cylinders cured under condition a. (It must be emphasized that the total number of cylinders tested was only 36).

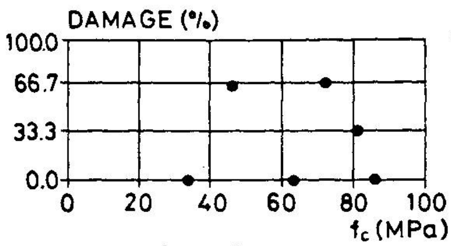

Fig. 4a.

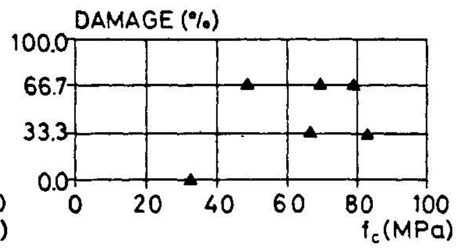

Fig. 4b.

Fig. 4a: Damage percentage for cylinders cured under condition a

Fig. 4b: Damage percentage for cylinders cured under condition b

5. CONCLUSIONS

On the basis of this work the following conclusions can be drawn:

There are significant differences in the shape of the compressive stress-strain curves from normal and high strength concrete, especially high strength concrete containing silica fume. The curve for higher strength concrete is much more linear to a much higher fraction of the compressive strength. The slope of the post peak range increases as the strength increases.

High strength concrete is less ductile than normal strength concrete. For reinforced concrete beams, the deflection ductility is independent of the concrete compressive strength if the ratio ρ/ρ_b is kept constant.

Only little information is reported regarding bond and anchorage of reinforcement in high strength concrete. Investigation conducted by Tepfers showed that the splice strength increased with increasing concrete strength up to $\sigma_{cube} \sim 70$ MPa. For larger values of the concrete strength the opposite is the case. Further investigations are needed in order to study anchorage problems in high strength concrete.

From investigation at the Department of Structural Engineering at the Technical University of Denmark, high strength concrete damage percentage during fire heating appears moderate. Further investigations are needed on this subject.

It is the author's opinion that more information is needed regarding shrinkage and creep and regarding the durability of high strength concrete.

ACKNOWLEDGEMENT

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Béton à hautes performances

Hochfester Beton

High-Strength Concrete

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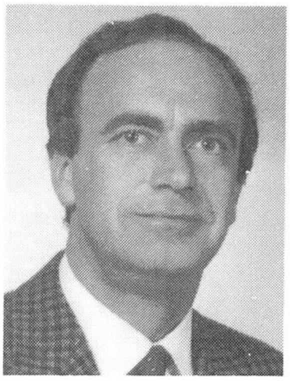


Gaël Cadoret, né en 1949, obtient son diplôme d'Ingénieur à l'INSA de Lyon en 1972. En 1976, il reçoit le titre de Docteur Ingénieur de l'Université Pierre et Marie Curie. Après avoir exercé une activité de laboratoire et de Contrôle sur chantier, il intègre Bouygues en 1983 où il est actuellement Chef de Service à la Direction Scientifique.

RÉSUMÉ
L'article rappelle les principes de production d'un béton de qualité et présente des résultats de recherches concernant les propriétés du béton à hautes performances. A l'occasion d'essais en production, l'entreprise a montré qu'elle savait produire industriellement des bétons à hautes performances et avec de faibles dosages en ciment. Sur trois chantiers actuels, la quantité totale produite à ce jour est de 20 000 m^3.

ZUSAMMENFASSUNG
Dieser Bericht fasst kurz die Prinzipien der Herstellung von hochwertigem Beton zusammen und stellt die Forschungsresultate über die Charakteristiken des hochfesten Betons dar. Proben von Beton, der an Ort hergestellt wurde, haben gezeigt, dass die Firma fähig ist, hochfesten Beton mit schwachen Zementmengen in industriellem Mass zu produzieren. Bis heute beläuft sich die Gesamtproduktion in drei aktuellen Bauprojekten auf 20'000 m^3.

SUMMARY
The article briefly summarizes the principles for manufacture of high quality concrete, and discusses research data relevant to the properties of high strength concrete. Test sampling of concrete manufactured on site has demonstrated the company's ability to produce high-strength concrete on an industrial scale. In three current building projects, output totals 20,000 cubic meters to date.

1. INTRODUCTION

L'amélioration des performances du matériau béton fait l'objet de travaux depuis de nombreuses années. En 1949, Eugène Freyssinet utilisait déjà une technique particulière d'essorage après coulage afin d'accroître la compacité de voussoirs préfabriqués en usine. Il obtenait ainsi des résistances de 100 MPa après quelques jours. 'Une heure après moulage, nos bétons, dont l'épaisseur totale pouvait dans certains cas descendre à 12 cm, résistaient à plus de 50 MPa ; leur charge de rupture atteignait après quelques jours 100 MPa'. Eugène Freyssinet, Conférence du 21 Mai 1954. Evocation de ses réalisations de 1933.

D'une manière générale, l'accroissement des performances et en particulier des résistances dans les bétons est obtenu, ainsi que l'avait parfaitement exprimé M. CAQUOT, en augmentant leur compacité.

Cette diminution des vides est acquise en diminuant l'eau servant au malaxage du béton, l'optimum étant atteint quand l'eau apportée est celle strictement nécessaire à l'hydratation des composés hydrauliques du ciment.

Une deuxième action visant à améliorer les caractéristiques des bétons consiste en l'incorporation de produits pouzzolanicites (Cendres Volantes, Fumées de Silice) dont l'action va se traduire, d'une part sur la rhéologie du béton frais (accroissement d'éléments fins voire très fins), et d'autre part sur la nature chimique des composés hydratés. Cette dernière action (pouzzolanique) consiste en la réaction des hydroxydes de calcium avec la silice amorphe pour former des silicates de calcium hydrates. Ces derniers composés, contrairement à l'hydroxyde de calcium, sont chimiquement résistants et géométriquement de petite taille.

2. LA CONCEPTION DU BETON A HAUTES PERFORMANCES

La définition d'un béton à hautes performances ne nécessite que l'application rigoureuse des principes et règles qui régissent la conception de tout béton de qualité.

La qualité du matériau béton est acquise si les trois paramètres essentiels que sont la DURABILITE, la RHEOLOGIE, et les CARACTERISTIQUES MECANIQUES sont maîtrisées

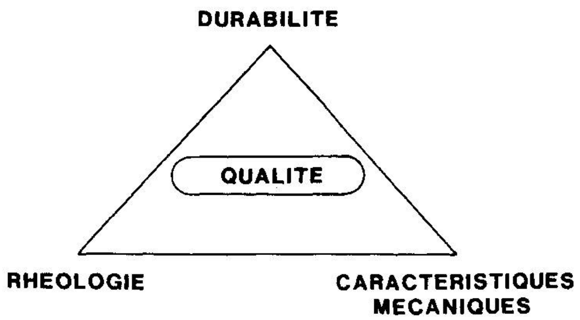


FIGURE 1 - LA QUALITE DU MATERIAU

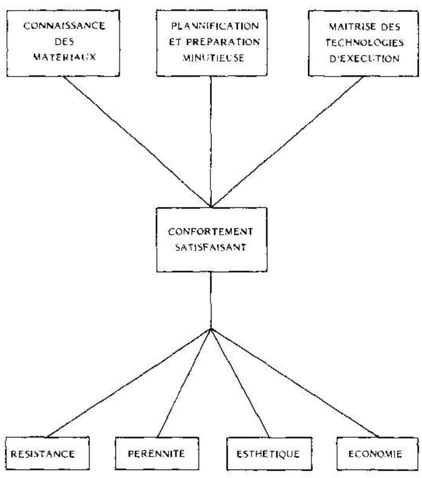


Fig. 2 - La qualité de la réalisation

Par ailleurs, la qualité de réalisation ne peut être obtenue que si la définition du béton est conçue en fonction des technologies de fabrication, transport et mise en oeuvre.

En particulier, la production de béton à hautes performances requiert généralement l'usage de fluidifiants afin de permettre une réduction d'eau importante tout en conservant une maniabilité convenable. Un défaut de maîtrise dans la chaîne matériaux de base, fabrication, transport se traduit par des variations importantes et inacceptables de consistance du béton à l'arrivée sur chantier.

Dans ce contexte, parmi les paramètres que nous considérons comme essentiels, nous pouvons citer :

Pour le ciment : sa composition chimique, début et fin de prise avec effet du E/C et de la température

Pour les adjuvants : effet de défloculation, incidence sur le temps de prise, optimisation du dosage.

Pour la fumée de silice : composition chimique, spectre granulométrique.

3. LES PROPRIETES GENERALES DU BETON A HAUTES PERFORMANCES

3.1 Rhéologie

Par la mise en oeuvre de dispositions appropriées, il est possible de produire des bétons dont la maniabilité est assurée pendant des temps de 1 h à 2 h après fabrication. Ainsi à l'occasion du coulage d'une dalle de couverture de culée (PLM A86) le slump après pompage a varié de 7 cm à 5 cm en deux heures, le béton ayant une résistance moyenne de 77 MPa à 28 jours et 86 MPa à 90 jours.

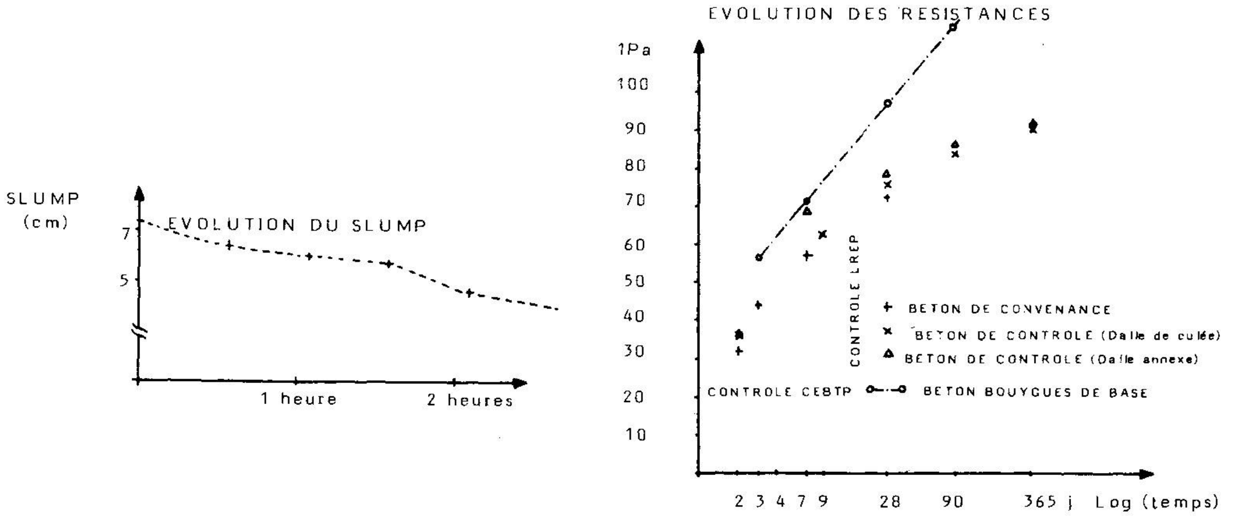


Fig. 3 : CHANTIER PLM A86 - Suivi du béton

Sur notre chantier de Tête de Défense, le bétonnage des Mégapoutres est assuré avec un béton qui après pompage a un slump restant compris entre 20 et 25 cm pendant plus d'une heure.

En fait, l'incidence de la rhéologie (béton fluide ou plastique) sur les résistances mécaniques est faible. Pour le béton étudié dans le cadre de la liaison Ré-Continent, la variation à 28 jours est de 2 MPa pour un slump passant de 7 à 22 cm, la résistance moyenne à 28 jours étant respectivement de 93,1 et 91,3 MPa.

3.2 Résistances mécaniques

Ce paramètre 'résistance mécanique en compression' doit s'apprécier en fonction des trois critères suivants :

Résistance au jeune âge (12-18 h) nécessaire pour la tenue des cycles de production (décoffrage - manutention - précontrainte)

- Résistance contractuelle généralement mesurée à 28 jours
- Résistance au cours du temps.

Dans ces domaines, les bétons à hautes performances ont un comportement semblable aux bétons ordinaires, à ceci près qu'il convient de tenir compte, le cas échéant, de l'incorporation d'agents pouzzolaniques très efficaces tels que les fumées de silice, et de la modification des vitesses d'hydratation du ciment ainsi que de leur composition chimique.

Ainsi, nous pouvons, soit valoriser les résistances à court terme, soit au contraire favoriser les gains à moyenne échéance.

A titre d'illustration, l'incorporation de 7% de fumée de silice dans les bétons de l'Ile de Ré nous a permis un gain de 7 MPa à 15 heures pour un béton non étuvé, fabriqué à 20°C et contenant 400 kg de CPA 55.

3.3 Durabilité

La résistance aux agents chimiques des bétons à hautes performances est améliorée pour les deux raisons suivantes :

- Une compacité plus grande se traduisant par une diminution de la porosité et de la perméabilité.
- Une structure chimiquement plus résistante dans la mesure où des produits à activité pouzzolanique élevée ont été incorporés au béton. Toutefois la fixation des hydroxydes de calcium devra être limitée afin de laisser au béton son caractère basique qui contribue à la protection des armatures.

3.4 Fragilité - Ductilité

Afin de vérifier et de mieux connaître le comportement à la rupture des bétons, dont la résistance moyenne en compression s'étend de 70 à 100 MPa, nous avons fait essayer en laboratoire (CEBTP) des éléments de structure. Ces mesures qui ont été faites à ces occasions sont en accord avec les conclusions de recherches semblables dont nous avons eu connaisance (Contrat SETRA-UTI n° 8440020 et ACI), et dont nous rappelons ci-après les principaux enseignements :

- En compression faiblement excentrée le béton à haute résistance apporte, à section égale, un gain important de capacité portante par rapport au béton normal.
- En flexion pure, la ductilité à la rupture du béton HR est légèrement meilleure que celle du béton normal.

Enfin, dans tous les rapports d'essais, il est noté des déformations ultimes plus grandes et importantes pour le béton à hautes résistances (cf figure 4) :

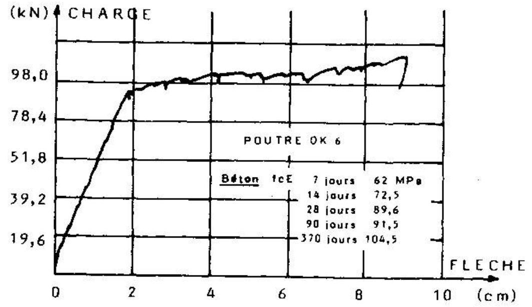
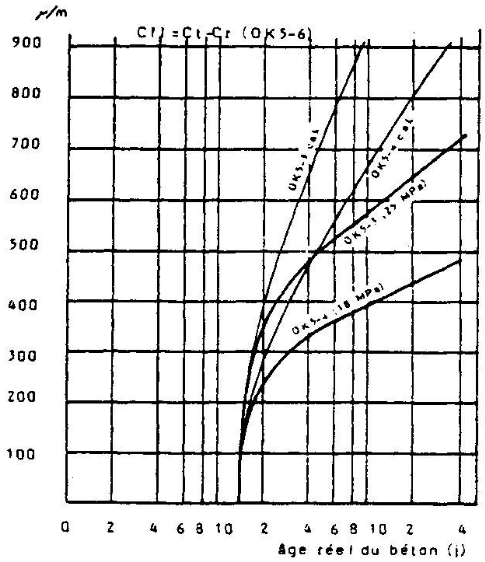


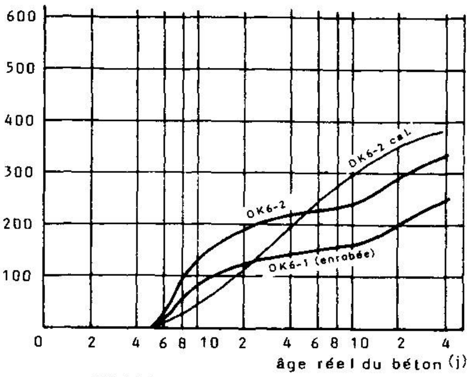
FIGURE 4 - DIAGRAMME CHARGE FLECHE AU CENTRE POUR OK 6



DEFORMATIONS DE FLUAGE
FIGURE 6

4. DEFORMATIONS DIFFEREES

4.2 Retrait et fluage



DEFORMATION DE RETRAIT

FIGURE 5

D'une manière générale le retrait n'est que légèrement réduit par rapport à sa valeur estimée à partir des règles FIP-CEB. Par contre, l'évolution de ce retrait est sensiblement différente comme illustré par la figure 5.

Les déformations de fluage sont considérablement réduites (facteur de l'ordre de 2) et ceci est d'autant plus marqué que la maturité du béton est avancée au jour de la mise en charge (7 et 14 jours). Cf fig. 6.

5. LE BETON A HAUTES PERFORMANCES SUR NOS CHANTIERS

A ce jour, sur trois de nos chantiers, l'Entreprise met quotidiennement en oeuvre du béton dont les résistances caractéristiques sortent du cadre réglementaire (limité à 40 MPa).

5.1 Grande Arche de la Défense

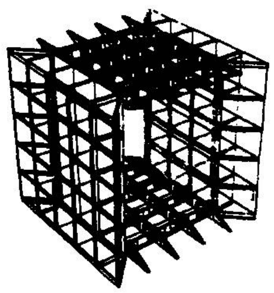


FIGURE 7

Il s'agit d'un bâtiment exceptionnel (fig. 7) constitué de blocs de 7 niveaux en béton armé se logeant à l'intérieur d'une méga-structure d'éléments de grandes dimensions et constituant l'ossature principale du bâtiment.

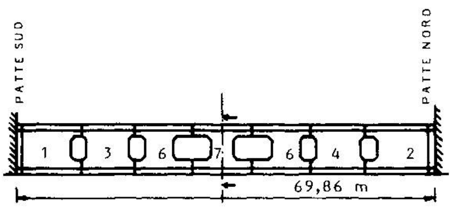
Pour les poutres principales et leurs noeuds ainsi que pour les chapiteaux des 12 appuis, il était nécessaire d'obtenir un béton de résistance caractéristique de 50 MPa.

Par ailleurs, pour des questions de mise en oeuvre et compte-tenu de la densité du ferraillage (300 kg d'acier/m3), il était nécessaire d'avoir un béton fluide pendant une durée au moins égale à 60 mn (en fait 90 mn).

Depuis Octobre 1985 jusqu'à ce jour, plus de 20 000 m3 de béton ont été coulés.

La résistance moyenne à 28 jours ressort à 60 MPa, ce qui procure une résistance caractéristique de 55 MPa.

Le tableau suivant illustre les résultats des contrôles de béton sur les Mégapoutres du plateau inférieur.



POUTRE	NBRE Prélèv	Rc 7j	Rc 28j	Rt 28j	Rc 90j	SLUMP (cm)
QE	7	44,4	57,5	4,7	66,1	22
QD	5	46,9	62,2	5,1	70,1	21
QC	9	44,5	60,3	5,0	68,8	22
QB	6	47,9	EN	COUR	S	22

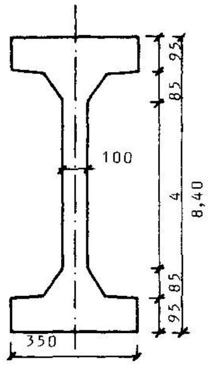


FIGURE 8

Pour le plateau supérieur, la formulation va évoluer et les résistances qui seront atteintes avoisinneront les 80 MPa à 28 jours.

5.2 Viaduc de Sylans

Il s'agit d'une structure de pont triangulée dont les éléments constitutifs de chaque voussoir sont préfabriqués en cellule.

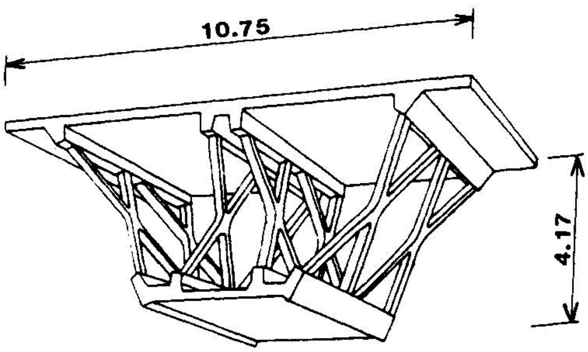


FIGURE 9

A ce jour, pour une exigence contractuelle de 40 MPa à 28 jours, nous constatons des valeurs moyennes de 30 à 40 MPa à 24 h, et à 28 j les résistances sont comprises entre 65 et 75 MPa

Le béton n'est pas étuvé, son dosage est de 400 kg de ciment par m3 de béton et il n'y a pas d'incorporation de fumée de silice. Le slump est compris entre 20 et 25 cm. L'obtention des résistances au jeune âge nécessaires pour le décoffrage et la manutention est ce qui a conduit le chantier vers cette formulation qui, au demeurant, permet de faire l'économie d'un poste d'étuvage.

5.3 Liaison Ré-Continent

Il s'agit d'une réalisation prestigieuse pour laquelle la préfabrication à terre des voussoirs se fait à la cadence exceptionnellement élevée de 7 à 8 unités par jour (1 unité = env. 40 m3).

Le béton avec 400 kg de CPA55 et 30 kg de fumée de silice permet d'obtenir pour un slump voisin de 16 cm et sans étuvage 22 MPa à 15 h, 50 MPa à 7 jours et plus de 65 MPa à 28 jours.

6. CONCLUSIONS

L'expérience et le vécu sur nos chantiers nous ont appris et confirmé qu'il est possible, de manière industrielle, de produire en quantité du béton à hautes performances. Ces valeurs acquises à ce jour en production (28 jours) sont encore modestes (65-80 MPa) mais ne figuraient pas dans nos objectifs sur ces chantiers où nous recherchions une performance à court terme.

Pour ce qui concerne le comportement tant mécanique que d'un point de vue de vieillissement de ces bétons et des structures dont ils sont partie intégrante, l'ensemble des essais nous ont montré un accroissement qualitatif du matériau et des réactions tout à fait prévisibles.

High-Strength Concrete in Chicago High-Rise Buildings

Béton à haute résistance pour les gratte-ciel de Chicago

Hochfester Beton für Chicagos Hochhäuser

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SUMMARY
Commercial ready-mix high-strength concrete is being delivered in the Chicago area for high-rise construction. High-strength concrete is a product of present urban market requirements. It has been the result of the commitment of the construction industry to an optimum economic return in high-rise construction. Historical, technical and marketing steps to incorporate the different levels of high-strength concrete in Chicago high-rise buildings are presented as are the physical properties of various levels of this concrete.

RÉSUMÉ
Un béton préfabriqué, ayant une haute résistance à la compression, est utilisé pour les gratte-ciel de Chicago. Le béton à haute résistance est un produit requis par les conditions actuelles du marché urbain. Il résulte de l'engagement de l'industrie de la construction pour réduire le prix de revient de construction des bâtiments élevés. Les étapes historiques, techniques et économiques de ce développement à Chicago sont décrites, ainsi que les propriétés mécaniques de ces bétons.

ZUSAMMENFASSUNG
In Chicago und Umgebung wird Transportbeton geliefert, welcher hohe Druckfestigkeiten erreicht. Die Verwendung solch hochfester Betone wurde ein Marktbedürfnis infolge wirtschaftlicher Optimierung bei der Erstellung von Hochhäusern. Die historischen, technischen und marktorientierten Schritte, welche zur Verwendung verschiedener Beton-Güteklassen in Hochhäusern führten, werden aufgezeigt. Auch auf die physikalische Zusammensetzung der Betone wird eingegangen.

1. INTRODUCTION

Chicago has been the city of the high-rise buildings and the year, 1986, commemorated one hundred years of high-rise construction. The Home Insurance Building was only twelve stories high, however, it has been considered the first high-rise because of the innovative steel skeleton within its masonry construction in 1886.

Commercial high-strength normal weight concrete has been one of the factors contributing to the development of high-rise buildings in Chicago. The development of this innovative material was by Material Service Corporation, the midwest's largest producer and supplier of concrete building materials and Flood Testing Laboratories. Latest development of this product has been by the quality control department of Material Service Corporation. It has fulfilled the needs of the various members of the construction team.

The developers' requirements for higher structures and larger rentable floor areas in expensive downtown real estate properties have been satisfied by this product. The architects' and engineers' needs to satisfy those demands has been accomplished with smaller and larger capacity columns. Commitment, technology and communication among the construction team members have accomplished the task of increasing the compressive strength of normal weight ready-mix concrete from 5,000 to 14,000 psi (34.5 to 96.6 MPa). This process has taken twenty years.

The complex technical and marketing process to bring each new concrete strength to the construction market has been a lengthy but rewarding process since it has made it possible to divert structures to concrete solutions.

2. ECONOMIC CONSIDERATIONS

Attached to products in the developmental stage is a premium that has to be paid for the additional benefits they provide. The engineer must evaluate the cost of these benefits at the job site, and the ready-mix producer must evaluate the production cost and the price the market can afford for the product.

Figure 1 shows the engineers' perception of the economic considerations in the use of High Strength Concrete. The most economical column has one percent reinforcing using the lowest possible strength of concrete. The information shown in the graphs has been obtained using 1986 Material Service Corporation book prices for concrete and average Chicago steel prices.

For the ready-mix supplier, the development of concretes above 6,000 psi (41.4 MPa) may not show direct economical benefits. The trial and error research program is a long and costly proposition, especially for a product which accounts for no more than a fraction of one percent of total concrete deliveries. The strict quality control for consistent production requires experienced and knowledgeable technicians and reliable equipment. The promotion and sale require professionals capable of answering questions on properties and design. Special equipment and knowledge is needed to test cylinders above 10,000 psi (68.9 MPa) concrete. Few ready-mix suppliers have the company infrastructure, resources, and attitude for such a project.

We feel that all of the previous negative reasons are counterbalanced by some indirect benefits to the ready-mix supplier. Through experience with high-strength concrete, the ready-mix producer is able to improve quality of the lower strength concretes. A better understanding of concrete allows the ready-mix supplier to develop special concretes. High-strength is a product differentiation which facilitates the sale of lower strength concretes. It improves the technical image of the company. These benefits have to be evaluated by the ready-mix company before the decision of moving from a comfortable and known low-strength market into an unknown market with a highly vulnerable position.

3. HIGH-RISE APPLICATIONS

Since 1965, 7,500 psi (51.7 MPa) concrete has been the high-strength concrete most widely used and accepted by the construction industry in the Chicago area. More than 50 projects have received the benefit of this strength. It fulfills the architectural and structural requirements for the lower columns of 20 to 25 story residential buildings with maximum column spacing up to 24 feet (7.2 Mt). This strength is also used for intermediate columns in buildings with higher strength in lower columns.

Starting in 1972, 9,000 psi (62.1 MPa) concrete became more frequently specified and it has been used in more than 40 buildings in the Chicago area. This strength was used in the Water Tower Place and its instrumentation has provided basic information shown later in this paper. Also, this concrete strength has been used for caisson construction.

The use of 11,000 psi (75.9 MPa) concrete was limited to two experimental columns in the River Plaza project in 1976. However presently, in 1986, its use has become more common for columns in different types of high-rise buildings.

Strengths above 14,000 psi (96.6 MPa) concrete were obtained for two columns in the Chicago Mercantile Exchange project and other projects. Instrumentation to measure concrete temperatures and actual shortening was placed on those two columns. A mock column was built at Material Service's yard 1 to obtain cores and their strength to be compared with the strength of standard 6x12 in. cylinders. Cylinders were made to measure creep and shrinkage and they have been tested by the Portland Cement Association.

Because only a small number of buildings are designed for over 50 stories, concretes over 14,000 psi (96.6 MPa) have limited application. Consequently, high-strength concrete technology for high-rises in the Chicago area presently has exceeded market requirements. The knowledge obtained from the development of high-strength concrete has led Material Service Corporation to several new products, including chemically resistant concrete, flowing, impermeable, corrosion resistant, fast track and other special concretes being sold as performance concretes.

4. PHYSICAL PROPERTIES

Sufficient information is now available for the safe use of this material by the construction industry. However,various research organizations are conducting additional research for better understanding of the physical properties of high-strength concrete.

4.1 Creep and Shrinkage

Creep and shrinkage have been part of the development of this product and measurements for these properties have been done in the laboratories of the Portland Cement Association. The specific creep (creep strain by unit of applied stress) decreases with the increase of strength, see Figure 2.

4.2 Measured Shortening in Columns

Actual shortening in columns has been measured in the same buildings mentioned previously for creep and shrinkage measurements. The results for the Chicago Merchantile project for the 14,000 psi (96.6 MPa) concrete columns are shown in figure 3. These measured shortenings are lower than the calculated shortenings.

4.3 Heat of Hydration

The increase of temperature within the concrete depends upon the cement content, water cement ratio, and size of the member. The historical records of the heat of hydration are presented in Figure 4 for the 11,000 psi (75.9 MPa) and the 14,000 psi (96.6 MPa) concrete. In both cases the peak of the heat of hydration occurs at about two days after the concrete is placed. No special curing is used for the columns where high-strength concrete is used. The removal of the forms is usually done the following working day after placing of the concrete.

4.4 Ductility

Recent investigations at the University of Illinois at Chicago on member ductility tested in flexure were found to be higher for the beams made of high-strength concrete compared to the beams made of low-strength concrete. The ratio of tension steel area to balance steel area was found to be the most important parameter governing the ductility of the members tested. For the same concrete strength, ductility decreased drastically as the above ratio increased. These results are available in the form of a Ph.D thesis and are being summarized for publication. Consequently, the concern about the lack of ductility caused by the use of high-strength concrete appears to be unfounded.

4.5 Modulus of Elasticity

Research conducted at Cornell University suggests Ec = 40,000 · f'c + 1,000,000 (Wc/145)E1.5 psi instead of the traditional ACI formula.

5. MIXTURES DESIGN

The mixture designs for the 9,000 psi(62.1 MPa) and 11,000 psi (75.9 MPa) columns of concretes used in the Water Tower Place and the River Plaza projects are as follows:

Material	9,000 psi(62.1Mp.)	11,000 psi(75.9Mp.)
Cement	846 lbs.	850 lbs.
Sand	1025 lbs.	1040 lbs.
5/8 in. Stone	1800 lbs.	--
1/2 in. Stone	--	1730 lbs.
Water	300 lbs.	330 lbs.
Water Reducer	25.4 fl.oz.	43.0 fl.oz.
Fly Ash	100 lbs.	100 lbs.
Slump	4-1/2 in.	4-1/2 in.

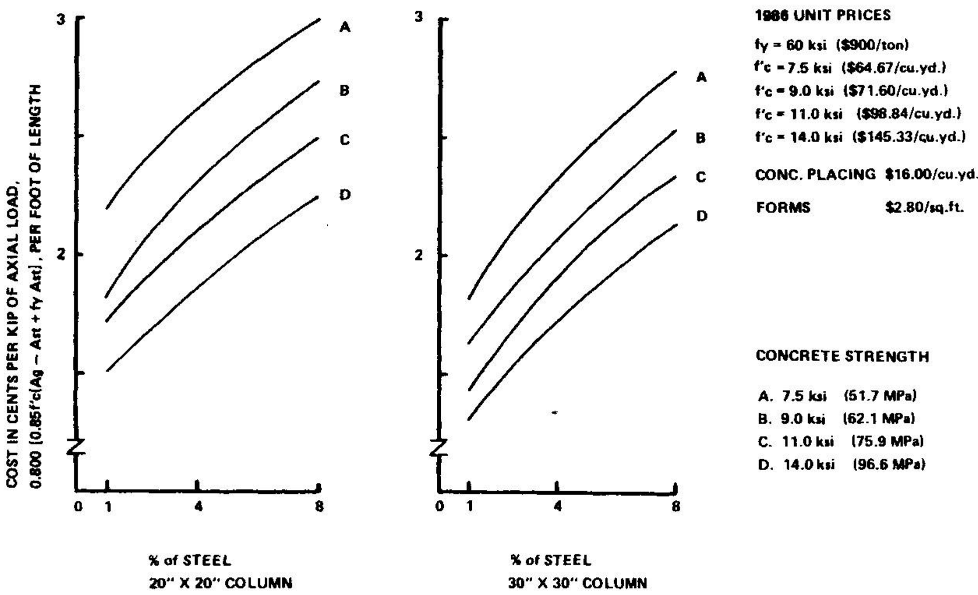
Recent developments in concrete technology have allowed the ready-mix industry to improve and optimize the mixtures shown in this table. The new mixtures for high-strength concrete are proprietary and this type of concrete is being sold as perfomance concrete.

6. CONSTRUCTION CONSIDERATIONS

For the contractor, the use of high-strength concrete implies some special considerations to comply with (10.13) ACI Code requirements regarding transmission of column loads through the floor slabs. When the specified concrete strength of the column is greater than 1.4 times that specified for the floor system, the transmission of load through the floor system is accomplished by placing concrete of strength specified for the column in the floor system for an area four times the column area. For the contractor, it implies special coordination since two different concretes are placed at the same time in the slab.

7. CONCLUSIONS

1. The production of high-strength concrete in excess of 6,000 psi places the primary responsibility for performance on the ready-mix supplier. 2. To produce high-strength concrete, a ready mix supplier must possess a complete infrastructure consisting of research and development, quality control, promotion, sales, and equipment. 3. Using Chicago prices, the cost per unit load carried by concrete decreases when the concrete strength increases. 4. High-strength concrete is a minimum percentage of the ready-mix supply, which may not make it commercially appealing for the ready-mix supplier. 5. The production of high-strength concrete improves the quality of lower strength concretes. 6. High-strength concrete helps the marketing and sales of lower strength concretes. 7. The technology of high strength concrete has exceeded the present market requirements for high-rise buildings in Chicago. 8. There is sufficient information on concretes up to 14,000 psi (96.6 MPa) for the safe design of structures. 9. High-strength concrete research has led to the development of new products like highly fluid concrete and other specialty concretes.



COLUMN COST

FIGURE 1

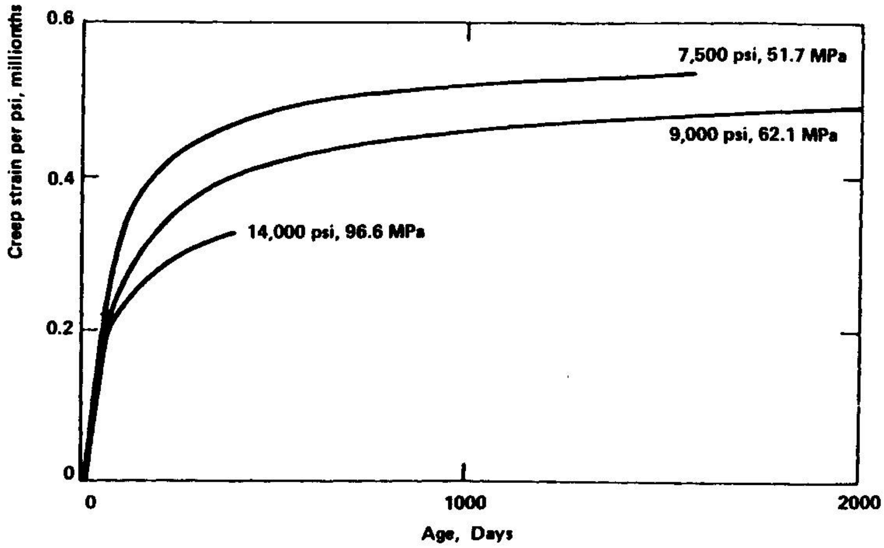


FIGURE 2

MEASURED CREEP IN 6 X 12 INCH CYLINDERS

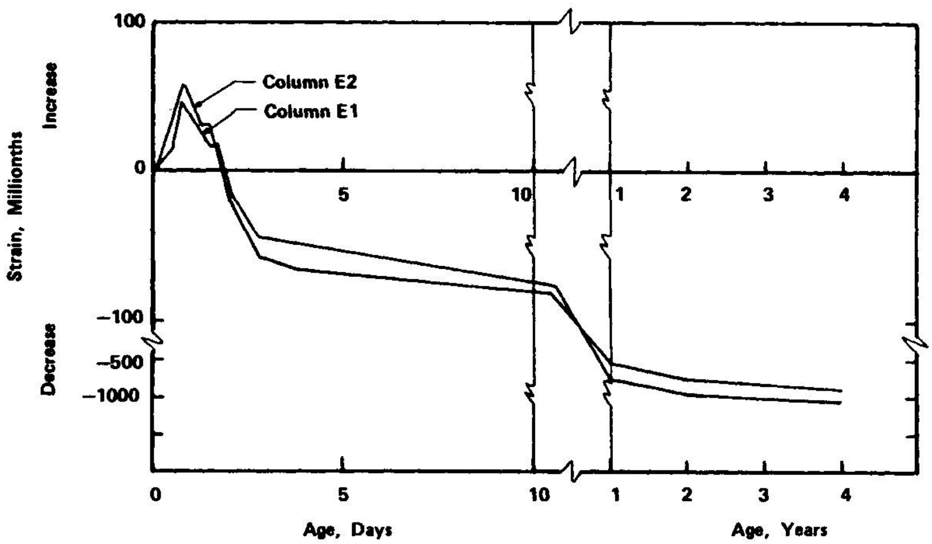


FIGURE 3

STRAINS AT THE CHICAGO MERCANTILE EXCHANGE
14,000 psi (96.6 MPa) COLUMNS

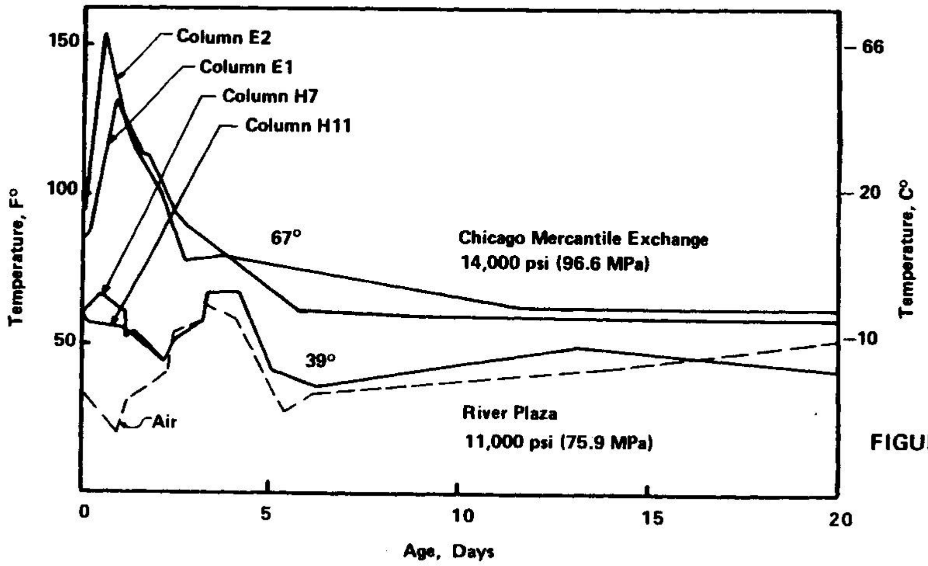


FIGURE 4

MEASURED TEMPERATURES

New Material for Reinforced Concrete in Place of Reinforcing Steel Bars

Nouveau matériau remplaçant l'acier dans le béton armé

Ein neues Material an Stelle von Stahl für die Bewehrung von Beton

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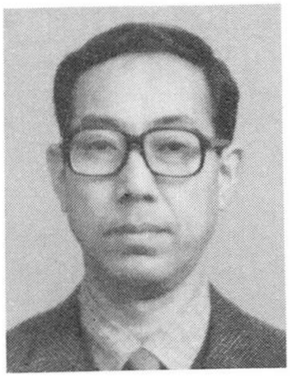

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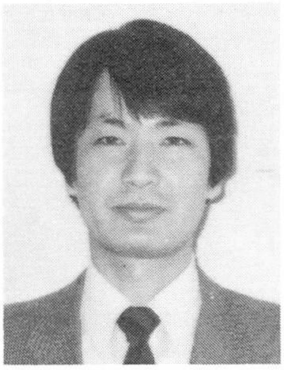
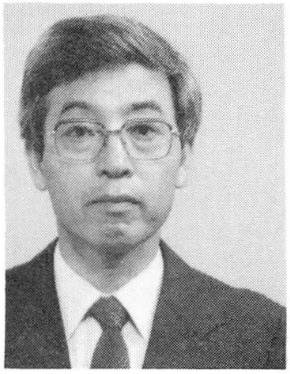




SUMMARY
The characteristics of a new material developed to replace reinforcing steel bars are introduced together with experimental results. This non-corrosive, lightweight new material is made of fiber reinforced plastics. It was formed in two – or three-dimensional grid shape and was developed for the purpose of improving the durability of reinforced concrete structures. Examples of applications to actual structures are also described.

RÉSUMÉ
Les caractéristiques du nouveau matériau mis au point à la place des barres d'armature sont présentées, de même que les résultats expérimentaux. Ce nouveau matériau léger et non corrosif est une fibre de plastique renforcée, produite sous forme de grille en deux ou trois dimensions. Il a été mis au point dans le but d'améliorer la résistance des structures en béton armé. Des exemples d'applications pour les structures réelles sont également donnés.

ZUSAMMENFASSUNG
Die Eigenschaften dieses neuen Materials werden unter Verwendung von Versuchsergebnissen näher behandelt. Das aus faserverstärktem Kunststoff bestehende Material zeichnet sich vor allem dadurch aus, dass es nicht korrodiert und leicht ist. Es können zwei – und dreidimensionale Gitterstrukturen hergestellt werden. Das Material wurde mit dem Ziel entwickelt, dem Beton eine bessere Dauerhaftigkeit zu geben. Einige Beispiele praktischer Anwendung werden beschrieben.

1. INTRODUCTION

It is said that the decline in the durability of steel reinforced concrete structures is caused mainly by the rust generated on the reinforcing steel bars. There are two new methods to cope with the rust generation: one is to prevent the concrete from cracking by improving its tensile strength such as fiber reinforced concrete, and the other is to replace the steel reinforced bars themselves with new material. The latter method is adopted, and developed Neo Fiber Material for Concrete (NEFMAC) to replace reinforcing steel bars is introduced in this paper.

2. CHARACTERISTICS OF NEFMAC

This newly developed NEFMAC is made of fiber reinforced plastics (FRP) produced by the filament winding method. Its characteristics are as follows:

(1) Desired strength, modulus and elongation can be obtained by changing the kind and the quantity of the fibers. Phenomena similar to those of reinforcing steel bars at the yield point are observed by combining different kinds of fibers.
(2) Sufficient anchorage to concrete is secured and lapped splice joints are made possible by making NEFMAC in grid shape and obtaining enough strength at the cross points.
(3) It is possible to form NEFMAC into curved surfaces and three dimensional shapes as well as flat surfaces thus making it unnecessary to process and assemble it in the field. And also re-bar arrangement such as diameters and intervals can easily be changed (See Fig. 1).
(4) Fatigue strength is equal to or greater than that of reinforcing deformed bars.
(5) Shear strength is about 50% of tensile strength.

Details of the results are not described in this paper, but it is known that the coefficient of linear expansion of NEFMAC is similar to that of concrete and that NEFMAC is light in weight (specific gravity is approximately 2.0). In addition, using vinyl ester resin as matrix, resistance to alkaline, acid and chemical products were tested and good results were obtained.

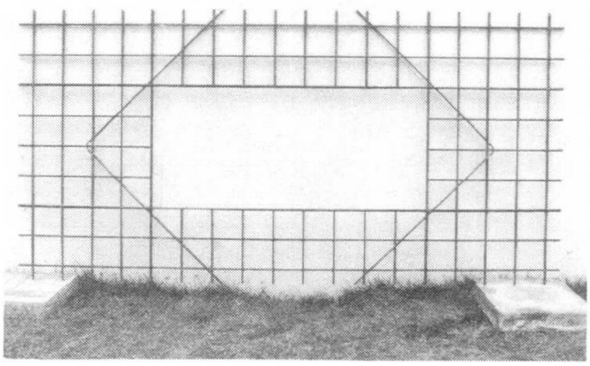


Fig. 1 Sample of NEFMAC

3. EXPERIMENTAL RESEARCH

3.1 Properties of Material

Fig. 2 shows schematically the stress-strain relationship of NEFMAC in cases where various fibers were employed. In this figure, the volume fraction of fibers Vf is taken as 40% for calculation. Examples of NEFMAC using high strength carbon fiber (HSCF), high modulus carbon fiber (HMCF), aramid fiber (AF), glass fiber (GF) and mixtures of the above fibers are shown in the figure. Phenomena similar to those when a reinforcing steel bar yields take place by combining various fibers. Fig. 3 gives an example confirming the above.

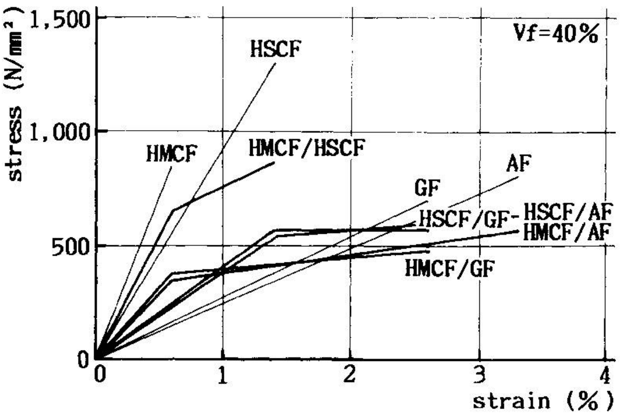
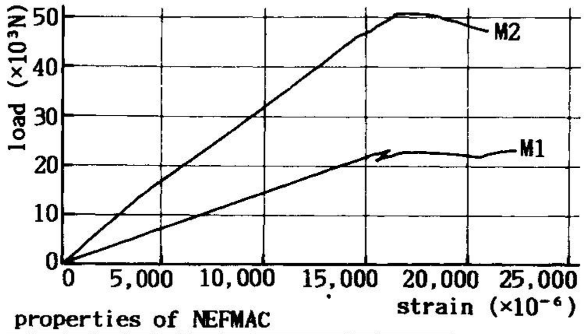


Fig. 2 Schematic stress - strain relationship of NEFMAC



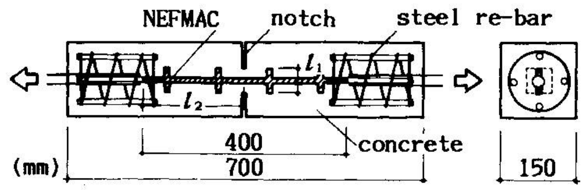
properties of NEFMAC

test piece	fiber	Vf (%)	sectional area (mm²)
M1	HSCF/GF	43.2	36.5
M2	HSCF/GF	42.2	84.1

Fig. 3 Actual load - strain relationship of NEFMAC

3.2 Properties of Anchorage and Lapped Splice Joint

The anchorage properties between NEFMAC and concrete were investigated through tensile tests by using the specimens shown in Fig. 4. In the experiment, the length of transverse reinforcement was taken as a parameter, and the development length of NEFMAC was kept as a constant at 2 intervals of the transverse reinforcement. The experimental results are given in Fig. 5. From Fig. 5 it became clear that anchorage of NEFMAC greater than the tensile strength of the longitudinal reinforcement can be secured by taking the NEFMAC transverse reinforcement as equal to or greater than 30mm and developing it as much as 2 intervals of transverse reinforcement into the concrete.



parameters of specimens

specimen	l_1 (mm)	l_2 (mm)
T1-1,2	30	200
T2-1,2	50	200
T3-1,2	70	200

properties of NEFMAC

fiber	sectional area (mm²)	Vf (%)	tensile load (N)	modulus (N/mm²)
HSCF/GF	84.1	42.2	49.9×10^3	39.6×10^3

Fig. 4 Specimens for the tensile tests

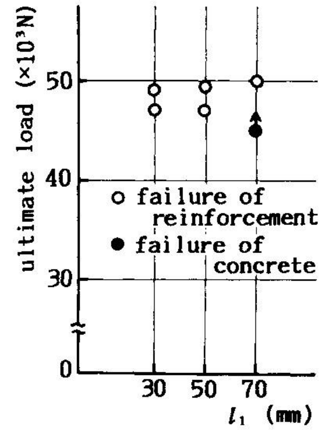
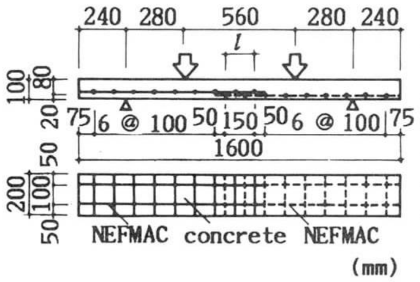


Fig. 5 Ultimate loads of the tensile tests

After the anchorage properties of NEFMAC were confirmed, bending tests of specimens having lapped splice joint were conducted. The shapes of the specimens and the test results are shown in Fig. 6 and Fig. 7 respectively. From these results it was confirmed that with the length of lapped splice greater than 1.0 times the grid size (more than 2 intervals of transverse reinforcements are overlapped), NEFMAC longitudinal reinforcements are broken and therefore lapped splice joints become possible.



parameters of specimens

speci-men	l (mm)	NEFMAC
B-1	50	GF
B-2	100	GF
B-3	150	GF
B-4	50	HMCF
B-5	100	HMCF
B-6	150	HMCF

properties of NEFMAC

fiber	sectional area (mm^2)	Vf (%)	tensile load (N)	modulus (N/mm^2)
GF	70.8	39.4	45.5×10^3	32.6×10^3
HMCF	88.4	22.6	33.4×10^3	78.1×10^3

Fig. 6 Specimens for the bending tests

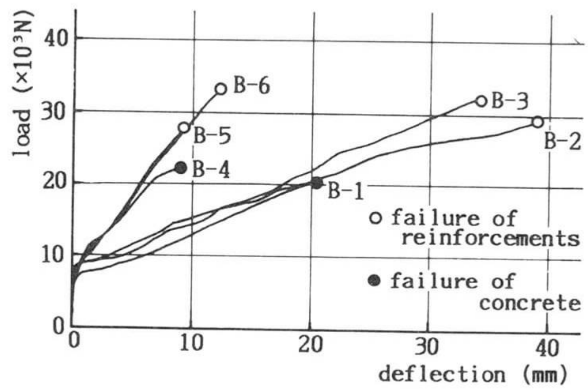
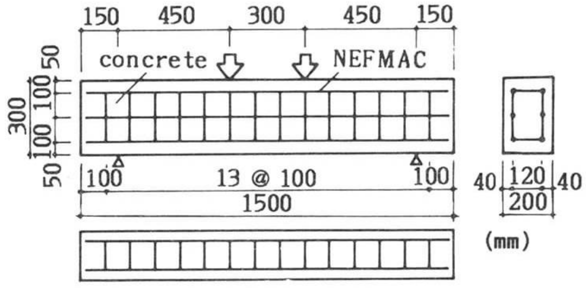


Fig. 7 Load - deflection curves of the bending tests

3.3 Fatigue Properties

Main reinforcements and stirrups of test specimens illustrated in Fig. 8 were formed in three-dimensional shape as shown in Fig. 9. Test specimens were subjected to two-point repeated loading. The test results are shown in Fig. 10. Many specimens were failed due to fatigue failure of main reinforcements at the cross points of stirrups. The fatigue strength of the deformed bars which have the same ultimate tensile load as NEFMAC are also shown in solid and dotted lines [1]. It became clear that the fatigue strength of NEFMAC is equal to or greater than that of deformed bars.



properties of NEFMAC

NEFMAC	fiber	sectional area (mm^2)	Vf (%)	tensile load (N)	modulus (N/mm^2)
main	HSCF/GF	207.1	25.5	78.6×10^3	27.1×10^3
stirrup	HSCF/GF	112.2	25.5	—	—

Fig. 8 Specimens for the fatigue tests

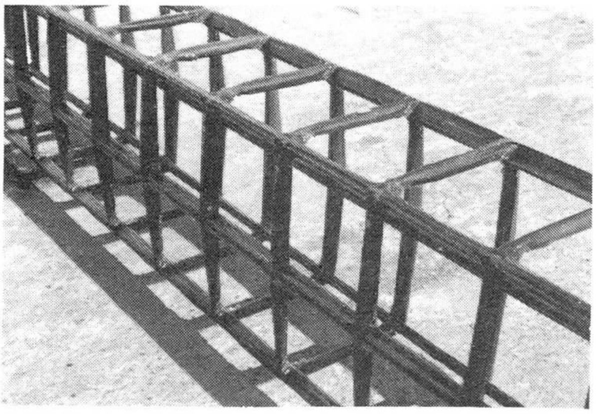


Fig. 9 Sample of three-dimensional shaped NEFMAC

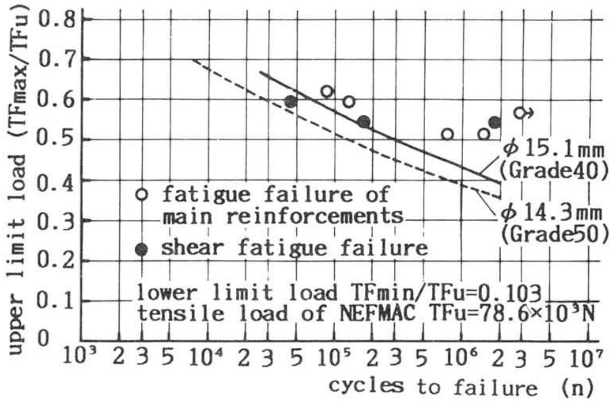
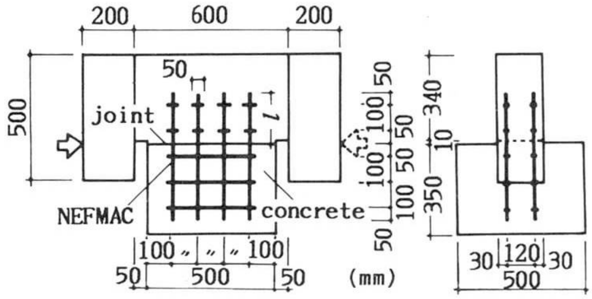


Fig.10 Fatigue strength of NEFMAC

3.4 Shearing Properties

As illustrated in Fig. 11 specimens with NEFMAC arranged in the concrete were produced and shearing tests were conducted. As for parameters in the tests, the development length of NEFMAC and loading program - monotonic or cyclic - were selected (See Table 1). In cyclic loading, the maximum displacements of specimens were controlled equal to the displacement at the 75% load of the maximum monotonic load, and they were forced to fail after 5 cycles. The test results are shown in Table 1. As listed in Table 1, it became clear that the shear strength of NEFMAC is about 50% of the tensile strength under monotonic loading. It also became clear that the development length of NEFMAC is enough to transmit the shearing force when it is 1 interval of the transverse reinforcement and that the shear strength becomes about 80% compared with that under monotonic loading when the specimens are subjected to cyclic loading.



properties of NEFMAC

fiber	sectional area (mm^2)	Vf (%)	tensile load (N)	modulus (N/mm^2)
HSCF/GF	84.1	42.2	49.9×10^3	39.6×10^3

Fig.11 Specimens for the shearing tests

specimen	l (mm)	loading program	Pmax (N)	Pmax/8 (N)
S-1	200	monotonic	196.1×10^3	24.5×10^3
S-2	200	cyclic	156.9×10^3	19.6×10^3
S-3	100	monotonic	194.2×10^3	24.3×10^3
S-4	100	cyclic	156.9×10^3	19.6×10^3

Table 1 Test results of the shearing tests

4. APPLICATION EXAMPLES

Application of NEFMAC to actual structures started in 1986 and there are 7 application cases for tunnel structures in particular. NEFMAC was used as reinforcing grids for shotcrete (See Fig. 12), and as reinforcements for arch and invert (See Fig. 13 and Fig. 14). It was adopted because of following reasons: a corrosion free material was required for a water-conveyance tunnel with flowing high acid water. And even if trouble with corrosion had not existed, NEFMAC, light in weight and requiring no processing or assembling, was best suited for improving the work productivity in a small space such as a tunnel where works depend on manpower.
In addition, application to inshore structures, underground structures and slope protection structures is now being planned.

Fig.12 Reinforcing grids for shotcrete

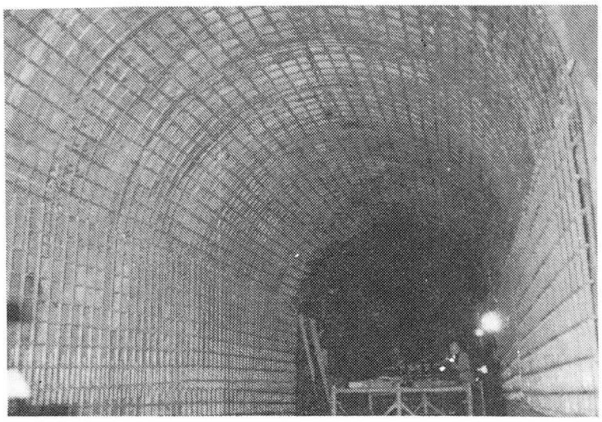


Fig.13 Reinforcements for arch of tunnel

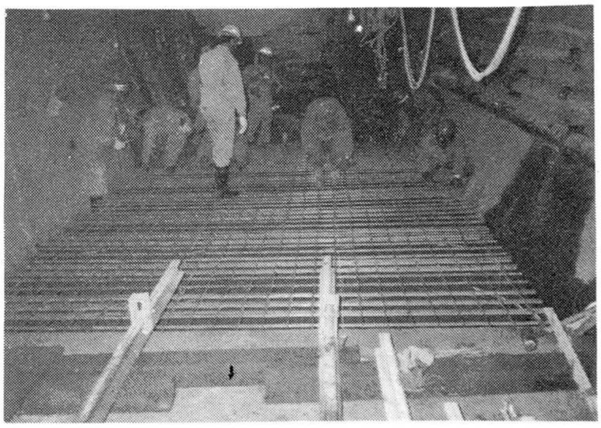


Fig.14 Reinforcements for invert of tunnel

5. CONCLUSIONS

The new non-corrosive, lightweight material --Neo Fiber Material for Concrete (NEFMAC)-- was introduced above and various test results were presented. NEFMAC, developed to replace reinforcing steel bars, is made of FRP that was formed in two- or three-dimensional grid shape to improve the durability of reinforced concrete structures. Confirmation tests on fire resistance are presently underway and it is expected that NEFMAC applications will be further widened in view of the test results.

ACKNOWLEDGEMENTS

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Hochleistungs – Verbundwerkstoff für die Vorspannung von Betonbauwerken

Heavy Duty Composite Material for Prestressing of Concrete Structures

Matériau composite à haute résistance pour la précontrainte d'ouvrages en béton

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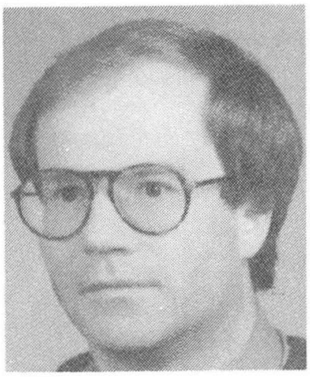

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ZUSAMMENFASSUNG
Faserverbundwerkstoffe, die bisher fast ausschließlich in der Luft- und Raumfahrt eingesetzt wurden, haben jetzt auch Anwendung in der Bauindustrie gefunden, z.B. bei der Vorspannung von Betonbauwerken. Durch Kombination von Glasfasern und Polyesterharzen wurde, als korrosionsbeständige Alternative zum herkömmlichen Spannstahl, ein Werkstoff hergestellt, der sehr hohe Medienbeständigkeit besitzt und Festigkeiten aufweist, die in der Größenordnung hochfester Spannstähle liegen.

SUMMARY
Fibre composite materials, previously employed almost exclusively in the air and space industries, are now being used in the construction industry, e.g. for prestressing of concrete structures. A material with high resistance to corrosive media, and with material strengths comparing with the strongest prestressing steels, has been manufactured as a highly corrosion resistant alternative to commonly used prestressing steels by correct choice of glass fibres and polyester resin.

RÉSUMÉ
Des matériaux composites renforcés par des fibres, qui, jusqu'à présent, ont été utilisés presque uniquement dans le domaine aéronautique et spatial, sont également mis en application dans le génie civil, par exemple dans la précontrainte d'ouvrages en béton. Par la combinaison des fibres de verre et de résine polyester on a obtenu un matériau anti-corrosif qui présente une alternative à l'acier de précontrainte traditionnel et dispose d'une résistance très élevée aux milieux corrosifs, et des caractéristiques comparables à celles des plus forts aciers de précontrainte.

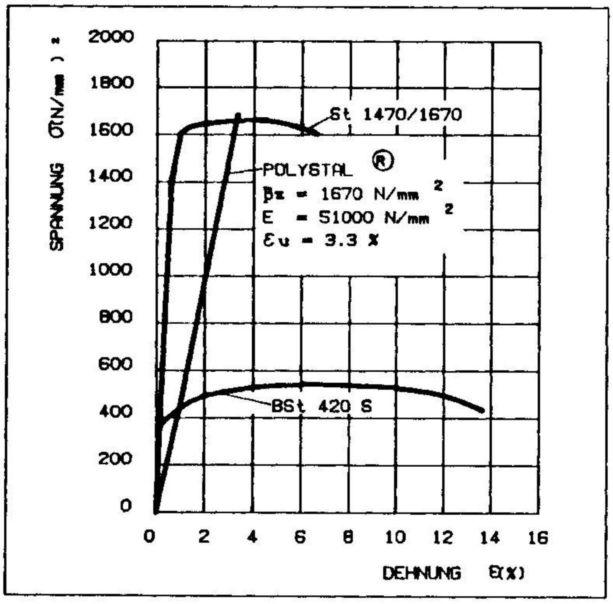
1. STABMATERIAL

Die Arbeitsgemeinschaft HLV-Elemente, bestehend aus den Firmen Strabag Bau-AG, Köln (Federführung) und Bayer AG, Leverkusen, entwickelte in einem vom Bundesministerium für Forschung und Technologie geförderten Vorhaben Glasfaserverbundstäbe bis zur Anwendungsreife als hochfeste Zugbewehrung für vorgespannte Konstruktionen.

Diese, unter dem Markennamen Polystal (R), von der Bayer AG hergestellten Glasfaserstäbe haben einen Durchmesser von 7,5 mm und bestehen aus 60.000 Glasfasern von 10 - 25 μm Dicke. Der Querschnitt enthält 68 % Glasfasern und32 % ungesättigtes Polyesterharz. Stränge aus je 2000 Glasfasern werden in einem Tauchbad mit flüssigem Polyesterharz imprägniert, zu einem Rundstab geformt und unter Wärmezufuhr ausgehärtet. Gegen chemische Einflüsse (wie Chloride und Alkalien) sowie gegen mechanische Beschädigungen erhält der Glasfaserstab eine ca. 0,5 mm starke Polyamidummantelung, die in einem "on-line" Extrusionsverfahren aufgebracht wird.

2. WERKSTOFFVERHALTEN

Die Längszugfestigkeit des Werkstoffes von 1670 N/mm^{2}ist Folge des hohen Glasfaseranteils mit streng unidirektionaler Orientierung. Die Zeitstandfestigkeit erreicht ca. 70 % des Endwertes der Zugkraft bei einem nahezu konstanten E-Modul von 51.000 N/mm^2(Bild 1).



<u>Bild 1</u> Spannungsdehnungsdiagramm von POLYSTAL-Stäben im Vergleich mit Betonstahl und Spannstahl

Die wesentlichen Unterschiede der HLV-Spannglieder im Vergleich zum Spannstahl sind:
- Der E-Modul der HLV-Spannglieder hat nur 1/4 der Größe des E-Moduls der Stahlspannglieder.
- Die HLV-Spannglieder zeigen nahezu einen linearen Zusammenhang zwischen Dehnung und Spannung bei fehlendem Fließvermögen.
- Die Dauerstandfestigkeit ist kleiner als die Kurzzeitfestigkeit.

Als weitere nennenswerte Werkstoffeigenschaften sind genannt:
- gute Medienbeständigkeit
- sehr gute Hitzebeständigkeit der tragenden Glasfasern
- elektromagnetische Neutralität
- geringes Gewicht von 2 g/cm^3.

Es wurde ein Bemessungskonzept entwickelt, das den vom Stahl abweichenden Eigenschaften Rechnung trägt.

Das fehlende Fließvermögen läßt vermuten, daß mit HLV-Spanngliedern vorgespannte Tragwerke nur eine geringe Systemzähigkeit aufweisen. Andererseits ist zu erwarten, daß der kleine Elastizitätsmodul der HLV-Spannglieder in Verbindung mit dem Aufreißen des Querschnitts ein duktiles Systemverhalten ermöglicht. Am Beispiel eines Durchlaufträgers, der zu einen mit HLV-Spanngliedern, zum anderen mit Stahlspanngliedern vorgespannt ist, wird das Verhalten bei Überlastung eines Feldes gezeigt.

Als Querschnitt für den zu untersuchenden Träger wurde ein Hohlkasten mit 3 m Bauhöhe und 45 m Spannweite gewählt. Die Querschnitte bei Spannbewehrung wurden so festgelegt, daß sich für Spannstahl und HLV die gleiche zulässige Vorspannkraft ergab. Die Belastung bestand aus den ständigen Lasten in allen Feldern und einer Verkehrslast als Gleichstreckenlast im mittleren Feld. Diese Gleichstreckenlast wurde schrittweise solange gesteigert, bis Systemversagen auftrat.

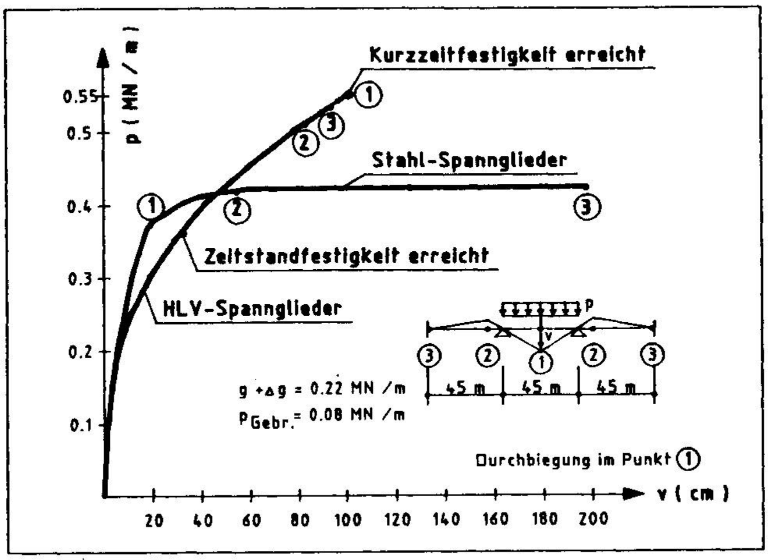


Bild 2 Last-Verformungs-Verhalten bei Überlastung eines Innenfeldes eines Durchlaufträgers

Aus dem Vergleich der beiden Kurven ist erkennbar, daß auch HLV-bewehrte Tragwerke große Verformungen ertragen können. Obwohl die HLV-Spannglieder kein Fließvermögen besitzen, sorgt ein "Systemfließen" für diese relativ großen Verformungen. Wegen des bei der Bemessung einzuhaltenden großen Sicherheitsabstandes gegenüber der Kurzzeitfestigkeit ist bei kurzfristigen Beanspruchungen eine höhere Traglast vorhanden als bei Tragwerken mit Stahlspanngliedern (Bild 2).

3. SPANNGLIEDER

Durch die nur ca. 10 % der Längszugfestigkeit betragende Querdruckfestigkeit mußten für die Einleitung der hohen Längszugkräfte an den Stabenden der Faserverbundwerkstoffe von der Strabag Bau-AG, die im Rahmen der Arbeitsgemeinschaft für die Verankerungstechnologie zuständig war, neue Wege beschritten werden:

- Die Möglichkeit einer Kaltverformung fehlt.
- Im Bereich der Verankerung muß ein gleichmäßig verteilter Querdruck aufgebracht werden.
- Die Verankerungslänge wird von der Schubfestigkeit der Staboberfläche bestimmt.

Die Lösung ist eine spezielle Vergußverankerung mit einem Kunstharzmörtel. Bisher wurden drei verschiedene Spanngliedervarianten mit 8, 14 und 19 Stäben bis zur Anwendungsreife entwickelt, mit Gebrauchslasten von 278 kN, 486 kN und 660 kN.

Die Spannkanäle werden bei Vorspannung mit nachträglichem Verbund nach dem Vorspannen mit einem von Bayer entwickelten Kunstharzmörtel verpreßt. Die wesentlichen Vorteile dieses Mörtels sind

- kein Ausfiltern des Zuschlags an Umlenkstellen,
- Möglichkeit des Nachverpressens,
- kein Vermischen mit Hüllrohrwasser,
- gutes Fließvermögen.

Für das Stabmaterial, die Spannglieder und den Verpreßmörtel wird z.Z. eine allgemein bauaufsichtliche Zulassung bei dem Institut für Bautechnik in Berlin beantragt.

Die Integration von Lichtwellenleitern und leitenden Metalldrähten als Sensoren in den Faserverbundwerkstoffen ermöglicht künftig den Einblick in das Spannungs-Dehnungsverhalten im Bauteil. Damit ist der Weg zur Verwirklichung des "intelligenten" Spanngliedes vorgezeichnet. Die Sensoren lassen dann bei Änderung ihrer physikalischen Eigenschaften (Dämpfung bei Lichtleitern, Kapazitätsänderung bei Drähten) z.B. Rückschlüsse auf Veränderungen des Spannungszustandes und deren Lokalisierung zu.

4. BAUTEILVERSUCHE

Zur Untermauerung der theoretischen Ergebnisse wurden am Otto-Graf-Institut in Stuttgart Bauteilversuche durchgeführt. Zum Vergleichist in Bild 3 die Last-Verformungskurve des Biegeversuches der theoretisch ermittelten Kurve eines spannstahlbewehrten Versuchskörpers gegenübergestellt. Bemerkenswert ist hierbei, daß die Verformung bei einer Entlastung vor dem Bruch nahezu vollständig reversibel ist. Die Bauteilversuche bestätigten, daß der Bruch auch bei HLV-bewehrten Konstruktionen durch große Verformung wie bei spannstahlbewehrten Konstruktionen angekündigt wird.

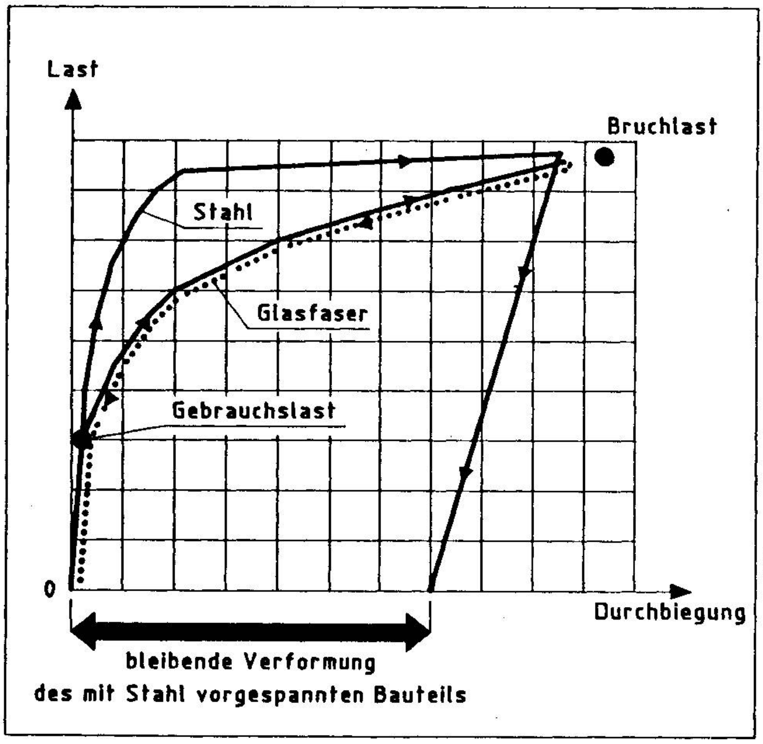


Bild 3 Last-Durchbiegungsbeziehung eines mit POLYSTAL vorgespannten Bauteils zu einem mit Stahl vorgespannten Bauteil.

5. BRÜCKE ULENBERGSTRASSE

Die erste großmaßstäbliche Anwendung der HLV-Spannglieder im Brückenbau erfolgte bei der Brücke Ulenbergstrasse in Düsseldorf. Diese Brücke der Brückenklasse 60/30 ist eine zweifeldrige massive Plattenbrücke, die in Längsrichtung mit nachträglichem Verbund beschränkt vorgespannt ist. Die Feldweiten betragen 21,30 m und 25,60 m bei einer Plattendicke von 1,44 m (Bild 4 und 5).

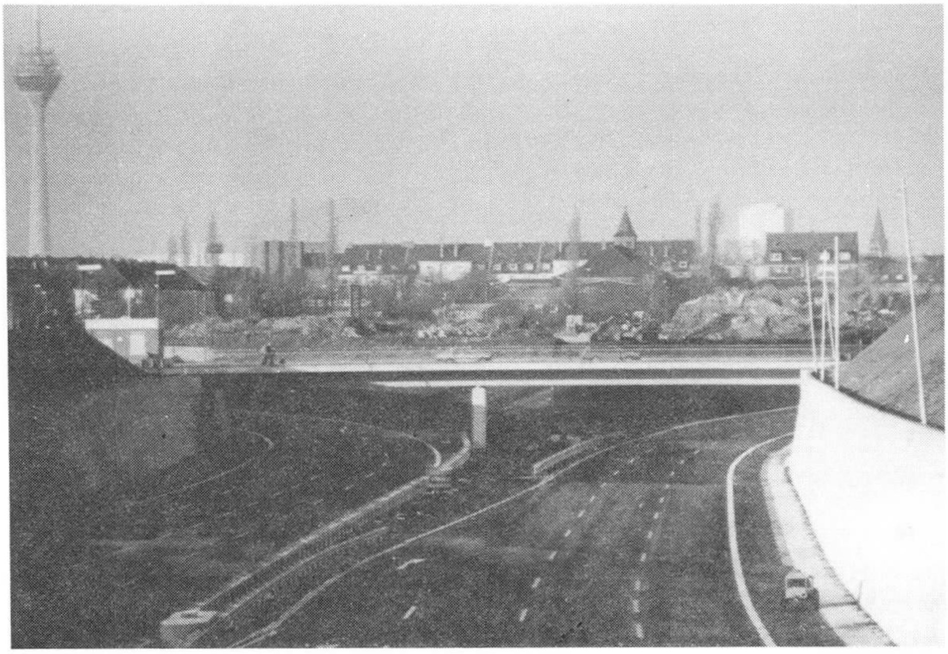


Bild 4 Gesamtaufnahme Brücke Ulenbergstraße, Düsseldorf

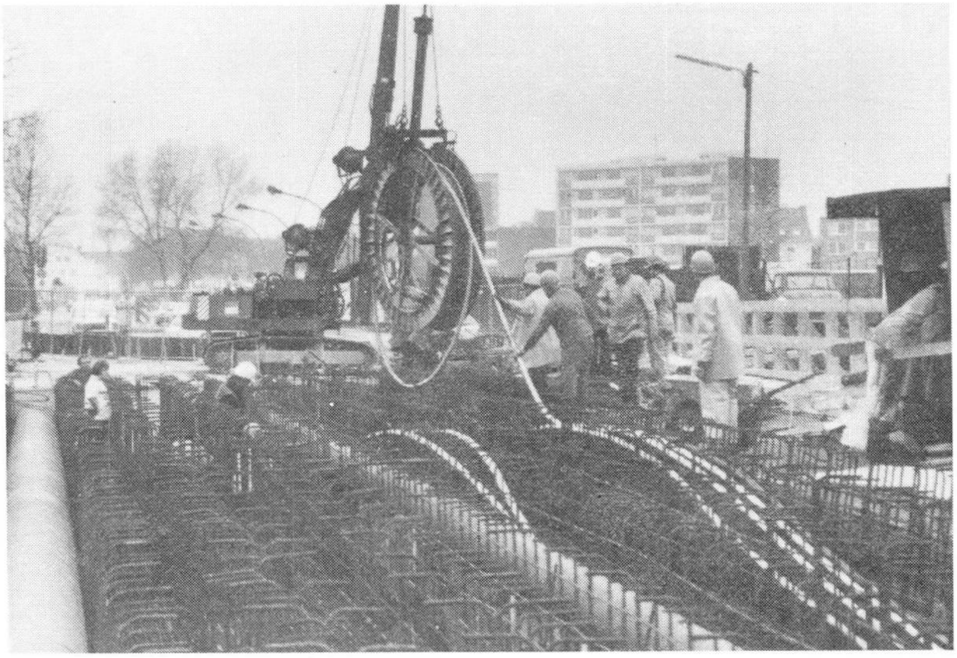


Bild 5 Verlegen der HLV-Spannglieder

Der Ausführung der Brücke Ulenbergstrasse lagen Befürwortungen durch gutachtliche Stellungnahmen zugrunde über "Beurteilung der Standsicherheit und Gebrauchsfähigkeit" (Prof. König, Frankfurt), "Material- und Verbundeigenschaften der HLV-Elemente im Hinblick auf einen Einsatz als Spannbewehrung" (Prof. Rehm, Stuttgart), "Beurteilung von Konstruktion und Tragverhalten der Verankerungen" (Prof. Rostasy, Braunschweig).

6. AUSBLICK

Der Bau der Brücke Ulenbergstrasse, der weltweit ersten großen Anwendung von Glasfaserverbundwerkstoffen für den Spannbetonbau, steht am Anfang einer Serie weiterer Anwendungsmöglichkeiten, z.B. der Einsatz von Hochleistungs-Faserverbund-Werkstoffen für alle Grade von Vorspannung mit und ohne Verbund, Erd- und Felsanker sowie Antennen- und sonstige Abspannungen.

Für die zunächst noch teureren HLV-Spannglieder werden sich die ersten Marktchancen bei korrosionsgefährdeten Bauwerken ergeben. Durch Befriedigung einer wachsenden Nachfrage wird dann bei Erreichen vergleichbarer Produktionsserien auch die Konkurrenzfähigkeit der HLV-Vorspannung mit den herkömmlichen Stahlspanngliedern erwartet.

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Prestressed Concrete Structures with High Strength Fibres

Ouvrages en béton précontraint, avec des fibres à haute résistance

Vorgespannte Bauwerke mit hochfesten Chemiefasein

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SUMMARY

Tendons of man-made high strength fibres – in this case aramids – are appreciated in environments which are aggressive to prestressing steel. The main advantages are : non-corrosive, insensitive to chlorides and to electro-magnetic currents. Characteristics and structure of the material as such differ from steel experience. Understanding of material behaviour, especially in alkaline environment and under stress is essential. Emphasis is laid on the differences in relaxation behaviour between steel and Arapree tendons.

RÉSUMÉ

Les éléments de précontrainte en fibres chimiques haute performance – ici en aramide – peuvent être utilisés avantageusement dans des environnements agressifs pour les aciers de précontrainte. Les avantages principaux sont les suivants : inoxydabilité, résistance aux chlorures, non-conductibilité électrique. La connaissance du comportement du matériau, notamment dans un environnement alcalin et sous contrainte est un facteur essentiel pour son application. Les différences de comportement en relaxation seront traitées particulièrement.

ZUSAMMENFASSUNG

Zugelemente aus hochfesten Chemiefasem – hier Aramiden – können bevorzugt eingesetzt werden in Umgebungen, die Bewehrungsstähle angreifen. Hauptvorteile sind : nichtrostend, unempfindlich gegen Chloride, elektrisch nichtleitend. Voraussetzung für den Einsatz ist die Kenntnis des Materialverhaltens, insbesondere in alkalischer Umgebung und unter Spannung. Vertieft werden die Unterschiede im Relaxationsverhalten.

1. INTRODUCTION

The properties generating the interest in the use of high-strength man-made fibres as an alternative to steel in reinforced and presstressed concrete structures are:

- high strength; up to 3000 N/mm² (fig. 1).
- non-corrosive; not attacked in carbonated concrete
- resistant to aggressive environments like chlorides
- insensitive to electro-magnetic currents

Practical use, however, is still restricted by:

- lack of experience and hesitation to use non-proven materials;
- relative low E-modulus; therefore preferably to be used in prestressing;
- brittleness; no deformation due to yield;

Not withstanding these restrictions it seems to be worthwhile to investigate the advantageous properties in view of the improvement of the durability of concrete structures in "exposed" conditions. Several developments of tendons based on high-strength man-made fibres have recently been made public [1] to [4]

For practical use as a prestressing tendon in concrete structures at least the following (very) long term aspects must be considered:

- creep/relaxation behaviour
- behaviour in different environments (e.g. alkaline and carbonated)
- stress rupture/stress corrosion behaviour
- residual strength under sustained loading.

For each of these the different nature of polymer materials (long chains of molecules) versus steel (atom-rostrum) could lead to significant and even surprising differences. This paper will extend the information given in [1] and [2] with special emphasis on current investigations about creep and relaxation of Arapree*). With respect to the other items only short indications are given.

2. NON-CORROSIVE TENDONS

2.1 Continuous tensile elements for structural application

In general polymer fibres are excluded owing to their low modulus of elasticity (fig. 1), high creep and temperature sensitivity.

Carbon fibres are very insensitive to aggressive environments. They may in general be disregarded on account of their low strain at failure (insufficient warning behaviour). Low E-modulus carbon fibres (pitch based types) may become a possibility in the future. As yet they are not considered. That leaves glass fibres and aramid fibres.

2.2 Glass-fibres

The first development which found its way into actual service is "Polystal", which consists of circular rods of E-glass bonded by a polyester resin. [3].

2.3 Aramid fibres

Aramid is an organic man-made fibre with a high degree of crystallinity. Two grades of stiffness are generally available; E-moduli in the range of 70 kN/mm² and 130 kN/mm². In the case of Twaron they are denoted Twaron and Twaron-High Modulus (HM). At the Imperial College of London Parafil ropes containing aramids to be used in unbonded tendons are being investigated (4). Enka and HBG jointly develop Twaron based tendons and stressing devices suitable to prestressed concrete. These tendons are named Arapree.®

*) Arapree®: a composite of Twaron® fibres and epoxy resin.
Twaron® : the aramid fibre produced by Aramide Maatschappij v.o.f.

2.4 Arapree tendons

For practical reasons like handling, good adherence, stability and resistance to many chemicals, epoxy resin was selected as the bonding matrix to produce tensile elements. A strip-like shape proved to be effective in continious bond with the cement matrix, using pretensioning and avoiding the need to insert permanent ducts or (metallic) terminations. In this approach effective use of the non-corrosive character can be made. A cover of only a few millimetres is sufficient. In addition, the strip-like shape enabled simple anchorage devices to be developed. The characteristics determining the use of EP impregnated Twaron HM as tendons in prestressed concrete are given in table 1.

3. DURABILITY

To simulate ageing, investigations are carried out at elevated temperatures to evaluate the retention of properties over 50 to 100 years. The results available, from accelerated testing [1], give a number of preliminary conclusions:

- chemical resistance: outstanding with regard to practically all hazards that can be assumed to exist in or around concrete structures; for instance:
- alkaline attack: lifetime predictions from extrapolations based on Arrhenius plots are fully satisfactory. As a preliminary guidance strength retention of over 80% after 180 days in a saturated $Ca(OH)_2$ solution of 80°C is assumed to be adequate.
- chlorides: no problem at all, which clearly indicates suitability for use in exposed concrete structures (marine environments);

4. CREEP AND RELAXATION

4.1 General

Relaxation and creep are interrelated material properties. Both describe a relation between stress, strain and time. Creep describes change in strain as a function of time at constant stress. Relaxation describes the change in stress as a function of time at a constant length. The former usually expressed in direct strain figures indicating the increase and the latter in percentages of initial stress.
The reaction of the structure of the materials on being stretched is specific. No general law gives a fixed relation between creep and relaxation. A significant difference between prestressing steel and polymeric materials becomes apparent from tests at different stresses.

4.2 Comparison between creep and relaxation of steel and aramid

Three possible relationships between creep and relaxation can be assumed. If relaxation is simulated by a creep test whereby, after a certain time interval, the measured creep is compensated by a decrease in load, then the obtained relaxation will be according to a, b, or c in fig. 2.
From steel it is known,[8][9] that a higher choice of the initial stress-level gives a more than proportional increase in creep strain. ($\varepsilon_{cr2}/\varepsilon_{cr1} > \sigma_2/\sigma_1$; fig. 3a.) Relaxation data show comparable results.[10]
Investigations on Aramids however show a different behaviour. With an increase of stress -as a percentage of ultimate strength- creep strain increases proportionally or even less than proportionally, from ε_{cr1} to ε_{cr2}
It can be concluded that -contrary to prestressing steel- increase of initial stress produces constant or even decreasing relaxation percentages for aramids for the same time interval. $\Delta\sigma_2/\Delta\sigma_1 \leq \sigma_2/\sigma_1$; fig. 3 b/c.
In the following discussion a constant relaxation percentage is assumed to be valid (fig. 3b). Consequently relaxation at t_x can be obtained from any creep test in the practical stress range. The apparent long term modulus E_{tx} giving the relation.

Even at a higher initial stress Arapree shows losses in stress, which are significantly less than prestressing steel type I and in the same range as type II.

5. LONG TERM BEHAVIOUR/SAFETY PROGNOSES.

5.1 Long term behaviour under stress

Practical use in concrete and in the building industry requires reliable long term behaviour under continuously stressed conditions. Creep is one of the properties investigated (see 4). Others, still under investigation, will deserve a -more extensive- future discussion ot the results. Also in these cases differences, due to the inherent behaviour of these fibres, from steel experience do occur.

5.2 Safety prognosis

Based on the above given indications from preliminary results and on [12] a safety prognosis on longterm behaviour under prestress in alkaline environment is given in fig 7.

6. APPLICATION

6.1 Experiments

To investigate the behaviour in actual practice experiments are being conducted on concrete elements. One of which is shown in fig.8. [14]

6.2 Fields of application

Notwithstanding the present price level -as compared to steel- an effective and economic long term use of these aramid/epoxy tendons for reinforcement or prestressing may be expected where:
- concrete is exposed to aggressive atmospheric attack;
- aggressive liquids and gases are to be stored;
- chlorides are present (seawater/de-icing salts);
- use of $CaCl_2$ can increase productivity;
- thin and light elements are required;
- large deformation capacity is required (impact, explosions, earthquakes).
- high fatigue requirements are to be met;
- electro-magnetic currents must be prevented.

Table 1: Comparison of properties (based on cross-sectional area of the fibres)

Properties	Units	Prestressing Steel	Glass (Polystal)	Arapree (Twaron HM)
Density	kg/m³	7850	2650	1450
(in resin)			(2000)	(1250)
Youngs Modulus	kN/mm²	200	70	130
Tensile Strength	N/mm²	1750	>2000	>3000
Initial level of prestress	N/mm²	1300	1000	1500
Elongation of break	%	>3.5	3	2.4
Relaxation (0.1-1000h)	%	(type II) 2-3	4	7-9
Chemical Resistance:				
pH>12		++	-	+
pH<10		--	+	++
Cl-ions		-	-	+
Temp.high	°C	400	500*)	300*)
low		+	-	++
Fatigue $2x10^6$		+	-	++

*) Not valid for resin

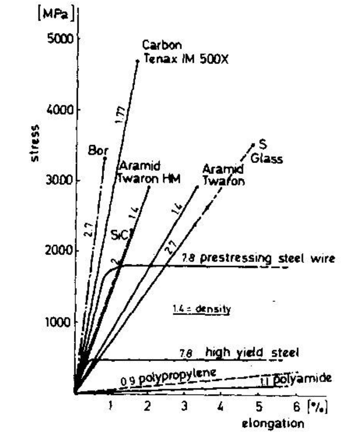

Fig.1 Stress-strain diagram of reinforcing materials

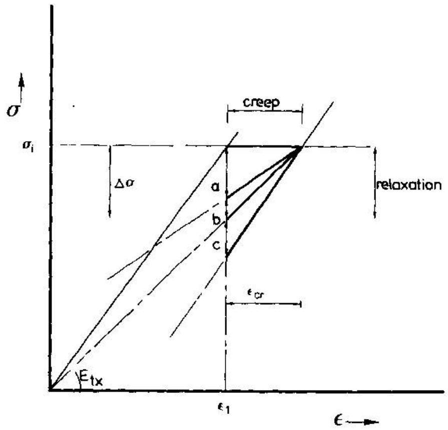

Fig.2 Possible relations between creep and relaxation at a chosen level

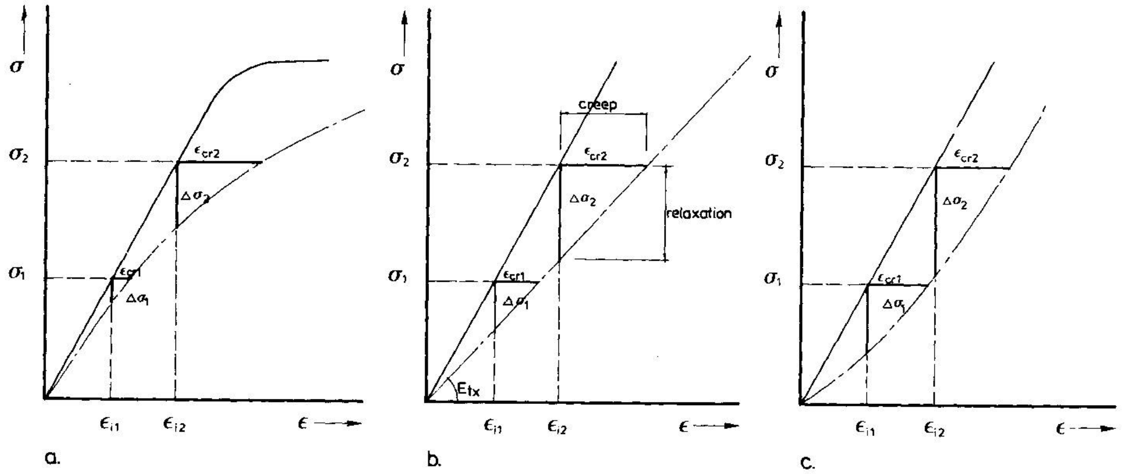

Fig.3 Schematic creep/relaxation behaviour at different initial stress-levels

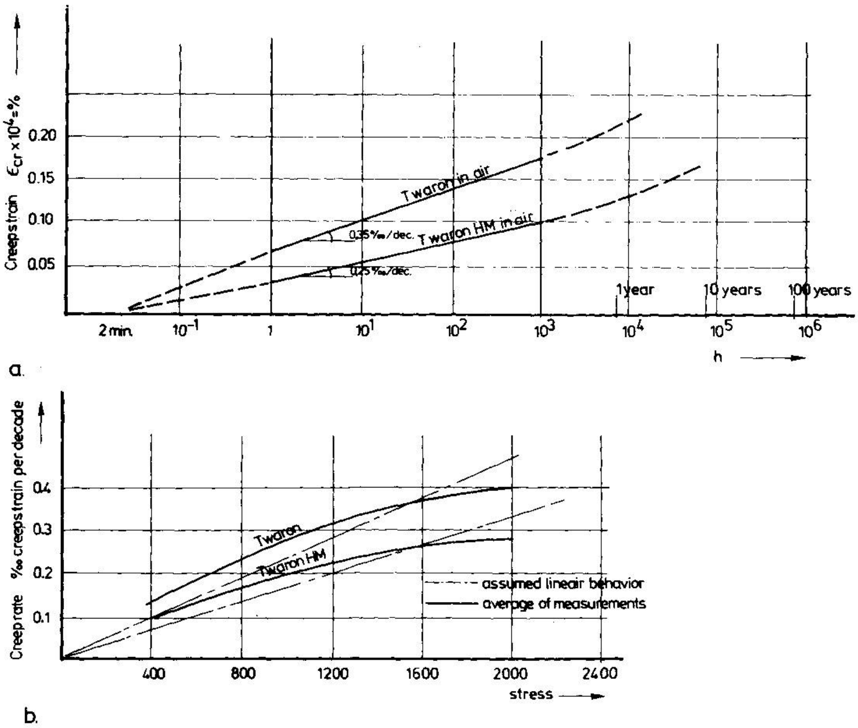

Fig.4 a Creep data of Arapree based on Twaron HM and Twaron in air, measured at $0.5f_u$ (ca. 1500 N/mm^2)

b Literature survey of average creep rates at increasing stresses

Preliminary data indicate a creepstrain in air of about 0,025% per decade at 50% of f_u (1500 N/mm²) for Arapree based on Twaron HM and about 0,035% per decade for Twaron based Arapree. These figures do increase slightly above 10^3 h. (fig. 4). From [5] corresponding data can be derived, as well as from [6] and [7] The former gives measurements even up to 1000 days. Fig 4b gives a survey of test results described in more than 20 sources [13].

4.3 Relation Creep/Relaxation

For HM: $\Delta\sigma$ per decade $\cong 0{,}025 \times 10^{-2} \times 130 \times 10^3 \cong 33$ N/mm² $\cong$ 2.1% of 1500 N/mm². Starting at 2 min after stressing this leads to a relaxation of about 10% up to 1000 h; independent of stress level (< 5 decades). For Twaron based Arapree a relaxation of about 8% at 1000 h can be calculated accordingly. Ongoing tests roughly confirm these calculations (fig. 5). The important difference with the behaviour of prestressing steel can also be derived from fig. 5. Final relaxation of prestressing steel is commonly calculated from the 1000 h results multiplying it by a factor of 3 (factor n in formula (1), see 4.4) to accommodate for the time from 10^3 to 10^6 h. This corresponds to a strong upward trend on the log/lin plot. Relaxation of Arapree starts at a higher rate per decade but remains more or less constant although a slight upward trend has to be considered. It seems that for final relaxation of Arapree a multiplication n = 2 will do. In fig. 6a an indicative comparison of relaxation at increasing stresses is given. Fig. 6b gives an extrapolation for 10^6 h. An essential difference in behaviour becomes apparent.

4.4 Example of losses in prestressed concrete

An example may provide indications on the implications of the above.

Losses du to relaxation $\Delta\sigma_{p,1} = n \cdot \Delta\sigma_{p.1000} \left(1 - m \dfrac{\Delta\sigma_{p,r+\phi}}{\sigma_{po}}\right)$ (1)

A pretensioned prestressed column 220 mm square ($A_c = 484 \times 10^2$ mm²) is subjected to a uniform initial prestress of 10 N/mm² (484 kN).
Initial stress level in Arapree 1500 N/mm² $\cong 0{,}5\ f_u$.
and in prestressing steel 1350 N/mm² $\cong 0{,}8\ f_u$.
$f_{ck} = 35$ [N/mm²] $\rightarrow$ E-mod $= 33{,}5 \times 10^3$ N/mm² $\rightarrow \varepsilon_{elast.} = \dfrac{10}{33.5 \times 10^3} = 0.33 \times 10^{-3}$
assume ϕ creep $= 2 \rightarrow \varepsilon_\phi = 0.66 \times 10^{-3}$
assume shrinkage $\varepsilon_{cs} = 0.25 \times 10^{-3}$ $\quad \dfrac{0.91 \times 10^{-3}}{1.24 \times 10^{-3}}$
Concrete deformation:

$\Delta\sigma_{p,r+\phi}$ becomes:
- Twaron HM $\rightarrow 0.91 \times 10^{-3} \times 130 \times 10^3 = 118$ N/mm²
- Twaron $\rightarrow 0.91 \times 10^{-3} \times 70 \times 10^3 = 64$ N/mm²
- Steel $\rightarrow 0.91 \times 10^{-3} \times 200 \times 10^3 = 182$ N/mm²

		Twaron HM	Twaron	Steel (type I)	Steel (type II)	
Losses due to relaxation:						
Twaron HM	$\Delta\sigma_{p,1} = 2 \times 10 (1 - 2 \times \frac{118}{1500}) =$	16.6				%
Twaron	" $= 2 \times 8 (1 - 2 \times \frac{64}{1500}) =$		14.5			%
Steel (type I)	" $= 3 \times 8 (1 - 2 \times \frac{182}{1350}) =$			17.3		%
Steel (type II)	" $= 3 \times 3 (1 - 2 \times \frac{182}{1350}) =$				6.5	%
Losses due to concrete deformations						
Twaron HM	$\Delta\sigma_{p,2} = 1.24 \times 10^{-3} \times 130 \times 10^3 = 161$ N/mm²	11.5				%
Twaron	" $= 1.24 \times 10^{-3} \times 70 \times 10^3 = 87$ N/mm²		6.2			%
Steel	" $= 1.24 \times 10^{-3} \times 200 \times 10^3 = 248$ N/mm²			18.2	18.2	%
Total losses	Twaron HM	28.1				%
	Twaron		20.7			%
	Steel (type I)			35.5		%
	Steel (type II)				24.7	%

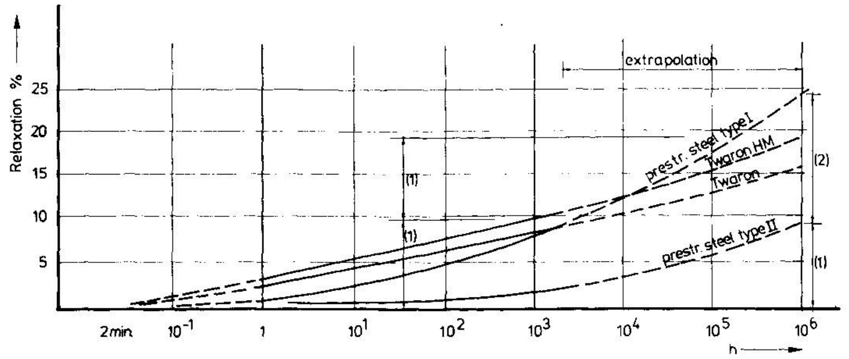

Fig.5 Relaxation behaviour
- Arapree (based on Twaron HM and Twaron), independent of initial stress
- Prestressing steel (type I and II), initial stress ca. $0.7f_u$

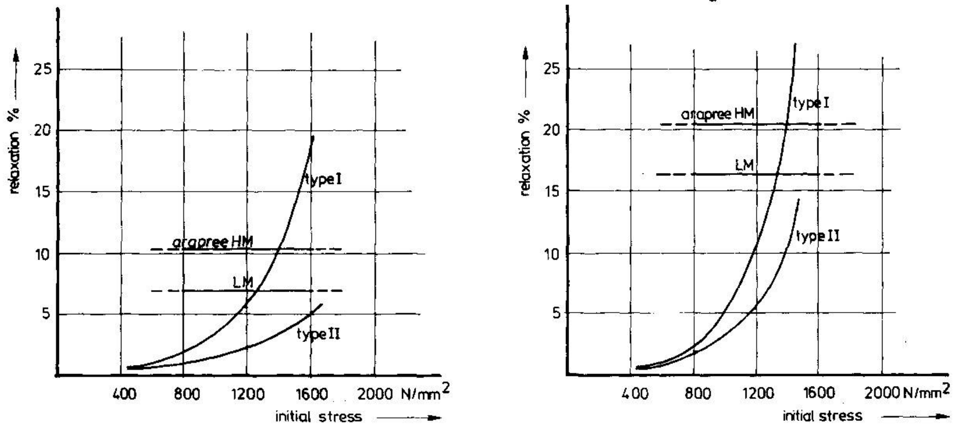

Fig.6 Comparative relaxation behaviour at different initial stress-levels
a Measured at 10^3 h
b Expected at 10^6 h

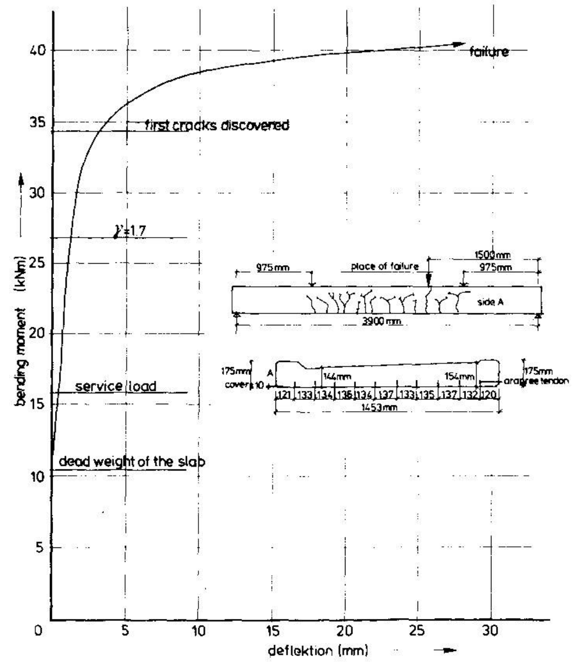

Fig.8 Graphical representation of a load-test on a balcony slab

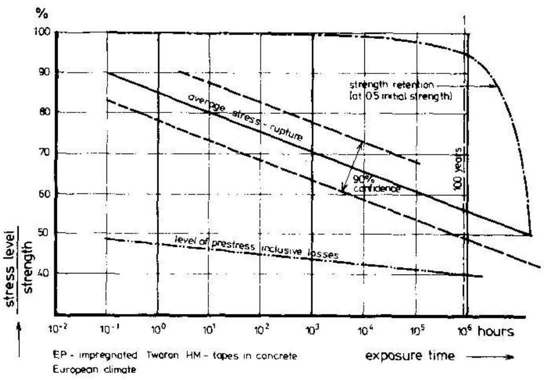

Fig.7 Safety prognosis

7. CONCLUSIONS

1. High strength man-made fibres in general and especially Arapree can be assumed to be a valid and satisfactory alternative for longterm use under stressed conditions in structural concrete.
2. With Arapree the final losses of stress to take into account for relaxation and concrete shortening have the same order -or even less- then those of prestressing steel.
3. With prestressing steel stress relaxation increases progressively at higher levels. Arapree shows a relaxation behaviour, which is roughly independent of the stress level applied.
4. The multiplication factor n -to calculate final relaxation losses from 1000 h measurents can be chosen equal to n = 2 (prestressing steel n = 3).
5. Arapree based on Twaron HM (with the higher E modulus) exhibits a lower creep rate than the type based on Twaron, but the relaxation of the Twaron based type is lower as a result of the different modulus of elasticity.

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SOUS-THÈME 4.2

Structures for Hazard Protection and Composite Structures

Structures pour la protection contre les phénomènes accidentels et structures mixtes

Bauwerke für den Schutz von Unfallphenomenen, und Verbundbauwerke

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Concrete Structures as a Safe Engineering Response to Environmental Catastrophes

Structures en béton : réponse technologique sûre aux catastrophes écologiques

Betonstrukturen als sicherheitstechnische Antwort auf Umweltkatastrophen

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Helmut Bomhard, born 1930, joined Dywidag in 1955 after studying Civil Engineering at the Munich Institute of Technology. In 1973 he was appointed Director for Structural Design and Construction. He designed several internationally well-known buildings and published 70 papers on different structural and scientific subjects. Since 1982 he chairs the FIP Commission on Concrete Storage Vessel Systems.

SUMMARY
Innovations must not be limited to the classical areas of concrete construction. As a matter of fact, they are needed in numerous new fields. One of these is the protection of the environment against catastrophes caused by technological failures. The article outlines the concept of a protective technology and of new concrete structures to guarantee a safe enclosure of raw materials, products and waste materials in an industrial cycle. The objective to be achieved is to strictly limit any damage caused by accidents.

RÉSUMÉ
Les innovations ne doivent pas se limiter aux domaines classiques d'utilisation du béton. Le besoin s'en fait sentir dans nombre de domaines nouveaux, en particulier celui de la protection de l'environnement contre les catastrophes d'origine technique. L'article expose les principes de base d'une technique de protection. Il indique selon quels principes les nouvelles structures en béton doivent être conçues en vue de constituer pour les matières premières, les produits ou les déchets impliqués dans le circuit industriel une enceinte offrant toutes garanties de sécurité. L'objectif visé est, quelle que soit la probabilité des accidents, de limiter strictement les dommages qui leur sont dûs.

ZUSAMMENFASSUNG
Innovationen dürfen sich nicht auf die klassischen Arbeitsgebiete des Betonbaus beschränken. Gefragt sind sie vielmehr auf zahlreichen neuen Gebieten. Dazu zählt der Schutz der Umwelt vor technikverursachten Katastrophen. Skizziert werden das Konzept einer Schutztechnik und neue Betonstrukturen zur Gewährleistung der sicheren Umschließung von Rohstoffen, Produkten und Abfallstoffen im industriellen Kreislauf. Ziel dabei ist es, unabhängig von der Wahrscheinlichkeit von Unfällen, den mit Unfällen verbundenen Schaden streng zu begrenzen.

1. INTRODUCTION

Modern industrial society has been thrown into a state of extreme uncertainty by spectacular cases of technological failures with catastrophic consequences. Names such as Seveso, Mexico City, Bhopal or Schweizerhalle - and last but not least Chernobyl - may be regarded as typical of this uncertainty. The consequence is that society welcomes a product but at the same time rejects the production process it involves. The public continues to expect general affluence and general safety. Affluence and safety are also political aims, as indeed they surely must be, but they can only be realized - if at all - by the continuing and increasing use of large-scale industrial technology. Any technology however has potential risks that may result in damage - indeed must do so at some time or other. Only a technology that we decide to do without it is devoid of any risks.

But even when the damage potential is catastrophically high, accidents need not necessarily result in disasters. This report aims to demonstrate this for a number of non-nuclear technologies and at the same time will attempt to answer the question: "How safe is safe enough?" It is based on development work carried out by Dyckerhoff & Widmann AG. Limitations of space do not allow a description of individual projects and the solutions found, but instead we will outline a preventive engineering concept for the entire industrial cycle: raw material, product, waste material, in which all these solutions are integrated and from which they were developed. What is technically feasible, and what is economically acceptable - both questions are dealt with here.

Applications of this kind for concrete construction are extremely new and constitute both an opportunity and a challenge.

2. MEXICO CITY AND BHOPAL - TWO CATASTROPHES

Most events with catastrophic consequences outside what may be termed "the industrial fence" - and this is what we mean by environmental disasters - have their origin in technical disturbances "inside the fence" which have got out of control.

In Mexico City, after a leakage in a PEMEX liquid gas plant, several explosions and devastating fires occurred, destroying the plant almost completely and devastating the surrounding housing estates. According to official figures more than 500 people were killed, with more than 7,000 seriously injured. Material damage is put at several hundred million US $. The area affected was 2.5 km in diameter.

In Bhopal, India, approximately 25 tons of highly toxic methylisocyanate were released from one of several storage tanks in a chemical factory belonging to Union Carbide. In this disaster about 3,000 people who came into contact with this toxic substance were killed, and about 200,000 suffered injuries to health. The amount of subsequent damage is incalculable. Lethal levels of poisonous gas were exceeded over an area of 40 km^2, with a diameter of approximately 7 km. India demanded more than US $ 3 billion compensation from Union Carbide for damage to property, health and environment.

3. SAFETY AIMS AND SAFETY MEASURES

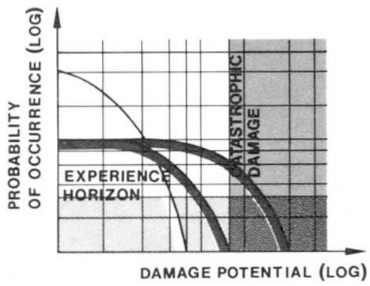

Fig. 1. Risk spectrum

Catastrophes are "infrequent events" - cases of technical failure with extreme consequences, as shown by the disasters of Mexico City and Bhopal, but the probability of their occurrence is really small. In the risk spectrum these are represented by the dark curves (Figure 1). A risk is usually defined as the product of the probability of occurrence and the potential amount of damage. Below a limit delineated by the relevant practical experience all indications of probability have a purely hypothetical character. For facilities involving a high level of risk - these are the ones we are dealing with - there is a gap between the reliability factor obtained by calculations and that dependent on operational experience. This gap can never be closed, however exact our calculations. Chernobyl has once again brought home this fact.

The necessary conclusion to be drawn from this is that a safety aim that only attempts to progressively reduce the probability of occurrence of "infrequent events" is insufficient. What is needed additionally is the restriction of the amount of damage to an acceptable, no longer catastrophic level. This has to determine our entire course of action. Only when we demand that the dark area representing catastrophic damage in the risk spectrum should also be avoided will our safety aims be comprehensive enough.

After Bhopal Union Carbide equipped several plants with a computer-controlled safety system connected to a meteorological station. Based on the dispersion values of any escaped chemical and the weather conditions it aimed at the rapid calculation of the direction and speed of a toxic cloud. Thus, at least in theory, makes it possible for a catastrophe alarm warning to be given in good time and for adjoining housing to be evacuated. This is undoubtedly one way of restricting the damage inherent in any given risk. Active safety systems such as this, however, require an extremely high standard of quality and high redundancy levels, but even then they are still not safe from "computer bugs".

Both active and other additional, i.e. passive safety systems are aiming at a technology that can fail without catastrophic damage. Only a technology safe within such limits is acceptable in the long term.

4. CONCRETE COMPONENTS AS PASSIVE SAFETY SYSTEMS

The aim in a safety context of this kind is the introduction of a passive safety system which, independent of the probability of accident consequences, rigorously restricts the volume of damage caused by such events. This can be achieved by structural safety measures using concrete components. What makes this passive safety systems so valuable is their almost deterministic reliability when compared to active safety systems. They are independent of accidents caused by human error, which for 80 % of all accidents is given either as the only cause or together with technical failure. A typical example of passive safety is the well-known firewall, which prevents fire from spreading not by starting any process or by automatic control technology, but simply by its sheer presence.

The facilities in Mexico City and Bhopal had nothing of this sort. In Chernobyl too safety relied to a great extent on "active safety" principles.

5. THE AIM OF SAFETY MEASURES: ENSURING ENCLOSURE

In all safety measures the main aim is nothing else but to ensure that hazardous materials are enclosed against external and internal influences over relatively long periods.

6. BURST-PROOF ENCLOSURE SYSTEMS

A small leakage in Mexico City caused a disaster because steel pressure vessels were able to explode like bombs. Explosions of this kind are termed "BLEVEs", which stands for "Boiling Liquid Expanding Vapour Explosion". A BLEVE is a physical explosion of the vessel, which "bursts" and is the centre of a more or less devastating wave of pressure, and - if the contents are flammable - of a wave of heat as well. The result is then a fireball. The waves of pressure and heat together with container fragments acting as missiles may also cause adjacent containers to undergo a BLEVE. Any facility can be completely destroyed by such a "domino effect", as happened in Mexico City.

Similar events can and will happen again elsewhere, because only very few containers today are likely to be sufficiently safe against BLEVEs.

What are needed are pressure vessels with a structure that is not only unaffected by BLEVEs but is also as far as humanly possible BLEVE-proof.

These containers are now available and can be built. Made of reinforced and prestressed concrete of varying shapes, they form an entire family of containers: for cryogenic and non-cryogenic liquid gases, and for flammable or toxic liquids [1].

All containers in this family are:

- safe against the effects of catastrophes, as far as humanly possible

and

- technically and economically efficient.

When operating pressures are high the container is in the shape of a spherical shell (Figure 2). The dimensions in Figure 2c relate to pressure liquefied propane. Figure 2d represents the construction process in which the sphere, consisting of several parts, is expanded and thus prestressed, then being closed to form a monolithic shell.

In contrast to Mexico City the containers in Bhopal are all very small and cylindrical. A concrete enclosing shell in Bhopal could also be cylindrical in shape. The ideal solution would be to combine them into larger units again spherical in shape.

Concrete pressure vessels, owing to their structure, have high redundancy levels. They cannot burst and are resistant to perforation and fire, e.g. [1]. In steel pressure vessels, on the other hand, the safety elements form a tandem connection, which is why once failure begins the process cannot be stopped. This was proved - not for the first time - by Mexico City.

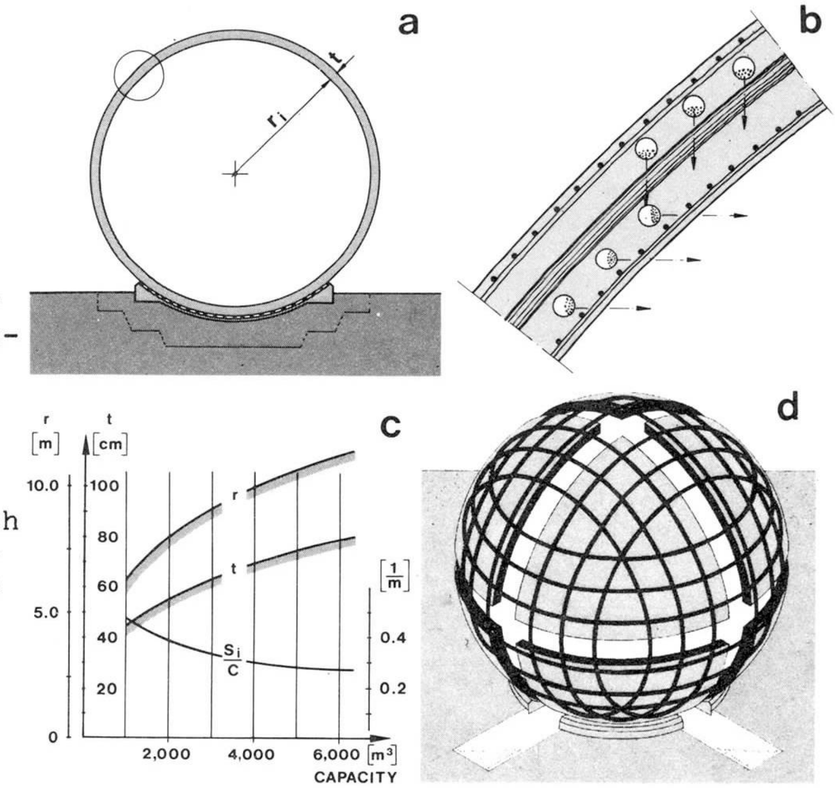

Fig. 2. Spherical concrete pressure vessel

Safety is expensive and more safety is even more expensive. We are used to this and are ready to accept it. The spherical concrete vessel - to restrict ourselves to this shape - should therefore cost more than one made of steel. In fact the reverse is true (Figure 3). The concrete container will cost less. This applies to the entire range of different capacities. Figure 3 shows the results of a cost comparison based on the pressure storage of liquid propane - in other words, one suitable for Mexico City. In such a case concrete containers can cost up to 40 % less than steel containers designed on the basis of the ASME Code. So we have good reason to assume that the new quality of safety we are discussing here and the higher availability connected with it will cost considerably less than what is still being built world-wide today.

The full economic advantage will and can display itself only if an entire facility, including all smaller safety distances and all other safety elements, is designed on the basis of concrete containers.

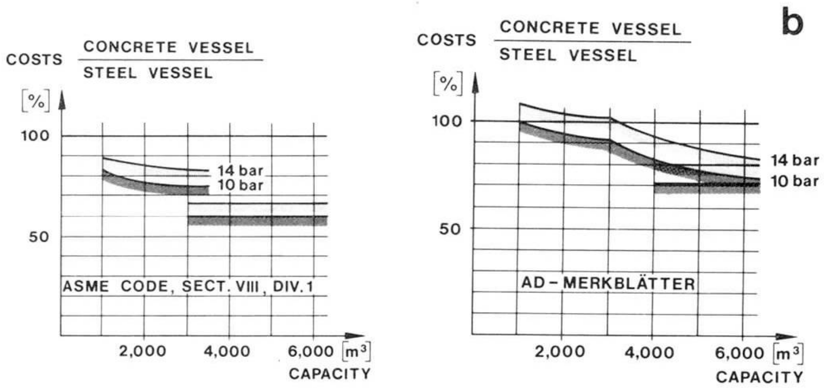

Fig. 3. Costs ratio of spherical pressure vessels

7. "MATERIAL TECHNOLOGY" AND "SYSTEM TECHNOLOGY" IN THEIR SIGNIFICANCE FOR ENCLOSURE SYSTEMS

So far we have discussed the enclosure of environmentally hazardous raw materials and products. However, waste materials are also involved in the material cycle. They too require to be reliably enclosed, frequently over very long periods of time, unless they can be recycled or their eluates are not harmful.

While the composition of raw materials and products - and thus how they will affect the enclosing system - are known, this does not as a rule apply to waste materials. If this is so, then material technology will lose its dominating role in the design of reliable enclosure systems, because we do not know which enclosing material - or whether there is one at all - can resist the effects of the waste material. System technology must now take the place of the material technology as the dominant technology. Only with its help will we be able to counter at the earliest stage any material-induced failure in the enclosing system.

8. SAFETY AIMS AND SAFETY MEASURES FOR MATERIALS WHICH ATTACK THE ENCLOSING SYSTEM IN A NON-DIFFERENTIABLE MANNER

The concept of risk is no longer applicable to material systems that can destroy the system enclosing them, so to speak as a result of their inherent nature. In such cases the risk of failure can no longer be limited. The safety aims detailed in Section 3 above must therefore be extended accordingly. From a structural point of view this means that the enclosing system can be guaranteed safe only if, and only then, it is designed in all its parts to be accessible to checks, to be repairable in a way that is accessible to checks - repeatedly repaired if necessary - and to be renewed. This is a new, entirely unusual design aim.

From this point of view enclosing systems for waste materials must differ markedly from those designed for raw materials and products, with regard to attention paid to safety and thus also to construction. Existing sytems do so - unfortunately in the reverse direction. Many waste storage dumps existing today are preprogrammed to becoming future liabilities, because the reliability of the enclosing systems continues to be - and is continually - wrongly assessed.

Waste material storage facilities for substances representing an environmental risk must be in the form of buildings - there is no way of avoiding this if we are serious about environmental protection.

9. ENCLOSING SYSTEMS THAT CAN BE CHECKED, REPAIRED AND RENEWED

We will describe a number of solutions to problems involving the storage of waste materials, these being based consistently on the safety aim of an enclosing system that can be checked, repaired and renewed [2]. This enclosing system has, as we shall see, a modular construction. This means that all solutions are adaptable, adaptable to very different requirements depending on whether - and how - the waste substances have been pre-treated.

All shapes are possible for the enclosing system, examples being mound-shaped dumps (Figure 4a) and container dumps (Figure 4b).

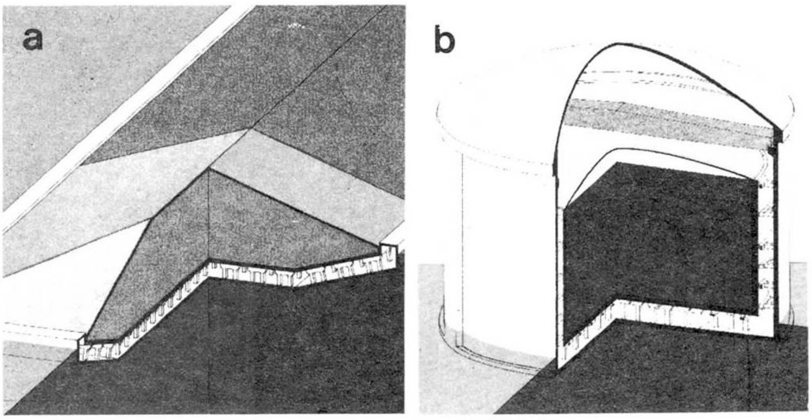

Fig. 4. Controllable, repairable and renewable enclosure systems for wastes

In the detailed cross-section of a container dump here considered as an example the modular construction can be easily recognized (Figure 5a). The elements from inside to outside are: Plate filter, plate lining, inside wall parts with supports to outside wall. As far as possible filter and lining material will be adapted to suit the task in question - in this example we dealing with the storage of untreated waste. Inside and outside walls are made of concrete.

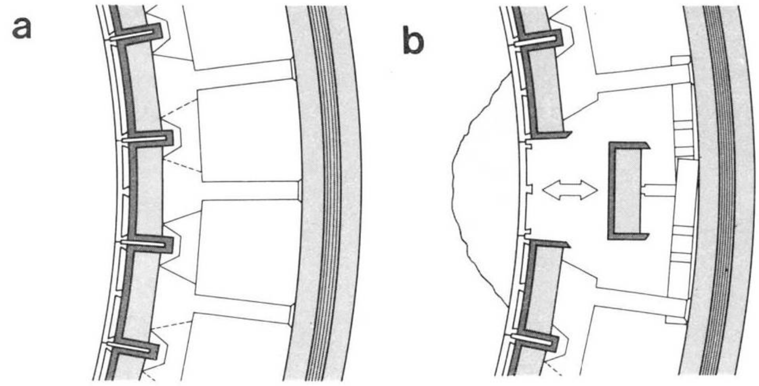

Fig. 5. Container-dump. Details of cross-section

How is repair work to be performed? How are parts to be replaced? This is also shown in the cross-section details given in Figure 5b for the container dump. The working operation is roughly as follows: Compact the waste locally, with the container remaining filled. Separate the lining seams, move the enclosing element hydraulically into the annular space - into the working position - repair and renew defective elements, e.g. plate filters, plate linings or even the concrete of the enclosing elements, return the element into its normal position, weld the seams of the lining together again - and the entire operation is completed. In the base area of the enclosing system work is performed in a similar manner. The same applies to the mound-shaped-dump.

This enclosing system is quite consciously based primarily on the principle of passive safety.

The design of the system allows for errors. The system is quite intentionally designed to accommodate a phase of trial and error. This means that technical and human errors can be kept under control and corrected.

The design however also contains redundancy elements, with all parts of the system being completely accessible. All working operations - the carrying out of checks, repairs, renewal work, including functionality tests, can be repeated as often as required.

Reliability in waste storage enclosing systems does cost more (Figure 6). Since the construction of earth waste dumps with a combined sealing enclosure and temporary roofing is still relatively frequently considered as a solution to the waste dump problem, let us use a dump of this kind as a comparison. The mineral sealing course should have a thickness of 1.5 m and the plastic lining course should be 3 mm thick. For non-pretreated waste - in this case the dump is then a biological and chemical reactor - the container dump costs 2 to 3.5 times as much as for this simple earth dump - for dumps with small capacities twice as much, for those with larger capacities 3.5 times. If the waste has been mineralized costs for the container dump will be halved, those 2 to 3.5 times being reduced to 1 to 1.7 times. Markedly cheaper than the container waste dump is the mound-shaped dump.

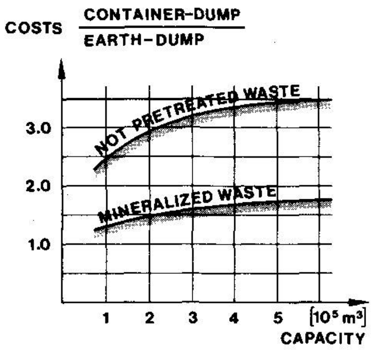

Fig. 6. Costs ratio of waste dumps

These comparisons show that expenditure on reliability does not increase exponentially as is sometimes assumed, but what does rise exponentially is the limitation of damage to the environment. Earth waste dumps with combined sealing enclosures permit neither checks nor repairs, and, since the effects caused by wastes are for the most part non-differentiable, they are with great probability condemned sooner or later to their own "ruin" from within themselves, and to the failure of the enclosing system. Claiming that reliable leakage rates for such a failure can be defined is an illusion.

10. CLOSING REMARKS

Today's trial and error procedures are no longer appropriate when environmental catastrophes caused by technology are concerned. The danger potential is too great for this, and a detrimental interaction of materials and factors which increase uncontrollably through their effect on each other is too probable. We can no longer live with this situation, and we do not have to. This report attempts to show this.

The task facing us all is to bring concrete structures as a preventive answer to environmental catastrophes into the public discussion of safety - a discussion that up to now has not been given a direction and which is stumbling from one disaster to the next. Used in the right way technology can and also must serve to protect the environment, and thus to conserve the quality of life. The better this principle is understood and put into practice, the less society will be tempted to reject technology.

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Design and Construction of Tall Reinforced Concrete Buildings in a Seismic Country

Conception et réalisation de bâtiments de grande hauteur en béton armé dans un pays sujet aux tremblements de terre

Entwurf und Ausführung von hohen Stahlbetonbauten in einem erdbebengefährdeten Land

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SUMMARY
This paper outlines several innovations in the conception of tall reinforced concrete buildings in Japan. Due to the high seismicity, the structures must be designed and built to withstand large alternating forces and preserve large ductility. Taking the case of a 30-storied residential building as an example, design procedures and safety confirmation methods as well as construction practices are described.

RÉSUMÉ
Cette contribution présente plusieurs innovations concernant les bâtiments de grande hauteur en béton armé au Japon. Ductilité et résistance à de grandes forces alternées doivent être absolument respectées en zone de haute séismicité. Le cas d'un bâtiment résidentiel de 30 étages sert à illustrer la conception, les moyens utilisés pour assurer la sécurité ainsi que les méthodes de la construction proprement dite.

ZUSAMMENFASSUNG
Der Beitrag stellt verschiedene Innovationen in der Konzeption von hohen Stahlbetonbauten in Japan vor. Infolge der grossen Erdbebenbeanspruchung müssen die Bauten grosse wechselnde Beanspruchungen ertragen können und eine grosse Duktilität ausweisen. Anhand eines 30-stöckigen Hochhauses werden die Projektierung, die Ausführung und die Sicherheitsaspekte beschrieben.

1. INTRODUCTION

It had been considered until a decade ago that, due to occurences of frequent severe earthquakes in Japan and brittle behavior of concrete structure in general, the construction of a tall reinforced concrete building was impossible. Learning lessons from many damages by earthquakes such as Tokachioki(1968,Japan) and San Fernando (1971,USA), and driven by economic demands the authors developed a new design and construction method for tall reinforced concrete buildings after solving problems on aseismic capabilities. The method called HiRC was already successfully applied to several tall buildings as shown in the references. [1,2,3]

As of March 1987, the completed reinforced concrete buildings over 50 meters high in Japan are limited to those by HiRC design and construction method. This paper is a general report on the innovative method for design and construction of other 30 storied concrete buildings which are now under construction in the City of Kawasaki near Tokyo.

2. OUTLINE OF THE BUILDING

The building is of 30 stories with the eaves height of 87.2 meters and has 230 housing units. Typical framing plan is approximately 30 meters square with recessed central portions on the four exteriors as shown in Fig.1 - 3. Building structure is composed of moment resisting frames by pure reinforced concrete structurally separated from walls, which makes possible the flexible housing layout at every floor.

In order to resist severe earthquake stresses especially induced in exterior columns on the lower story, following three items are, although unprecedented in Japan, introduced in the design practice.
- to use high strength concrete up to 420kgf/cm. (Concrete up to the 360kgf/cm was used for previous buildings.)
- to provide additional core-reinforcing bars in exterior columns on the lower story.(Prestressing tendons were used in case of previous buildings in order to reduce high tensil stresses)
- to use large subsoil diaphram walls in addition to piles with enlarged base in consideration of potential liquefactions as well as vertical and horizontal loadings.

3. DESIGN PRINCIPLE

Seismic design of the building structure is conducted through two-stage designing. The first one is based on so called allowable stress design method, while the second is on ultimate strength design method. Design earthquake force applied in the former stage is defined by the preliminary earthquake response analysis, against which the stresses induced in beams and columns should be within allowable limits.

The second stage of design is to check the stresses of the column and joining portions on condition that all the beams have already yielded, which implies that strong-column and weak-beam design principle is indispensable and the design procedure should take a form of feedback system. Typical cross sections of columns and beams by the final design are shown in Fig.4. High strength concrete and additional core-reinforcing bars make the unit structural costs reasonable compared with those of the previous lower buildings.

Fig.1 Building under construction

construction

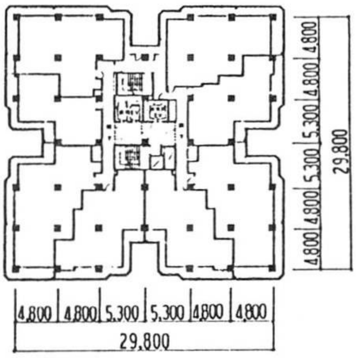


Fig.2 Typical floor plan

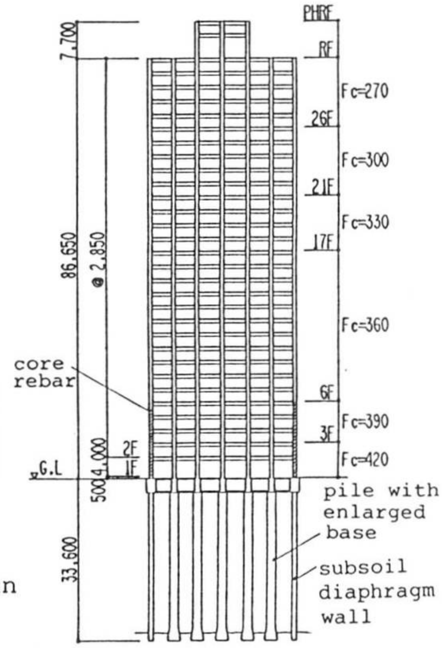


Fig.3 Framing elevation

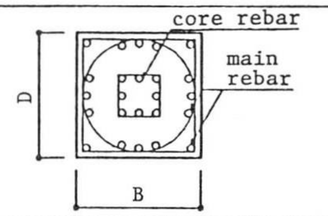
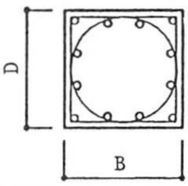
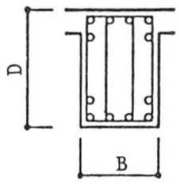
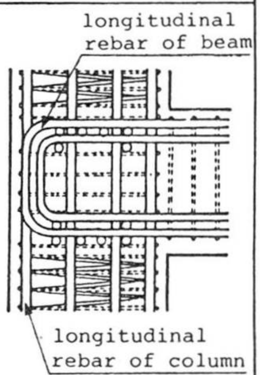
	Exterior column				Interior column			Beam			U-shaped anchorage
Cross section	 core rebar, main rebar, D, B				 D, B			 D, B			 longitudinal rebar of beam; longitudinal rebar of column
floor	B x D	main rebar	core rebar	hoop	B x D	main rebar	hoop	B x D	main rebar	stirrup	
30 ↓ 2	800×800 ↓ 850×850	12-D32 ↓ 16-D41	— ↓ 8-D41	D13 @150 13φ @150 ↓ D16 @100 16φ @100	750×750 ↓ 800×800	12-D32 ↓ 16-D41	D13 @150 13φ @150 ↓ D16 @100 16φ @100	500×750 ↓ 600×1000	4-D25 2-D22 ↓ 4-D41 2-D38	4-D13@175 ↓ 4-D16@125	

Fig.4 Typical cross sections

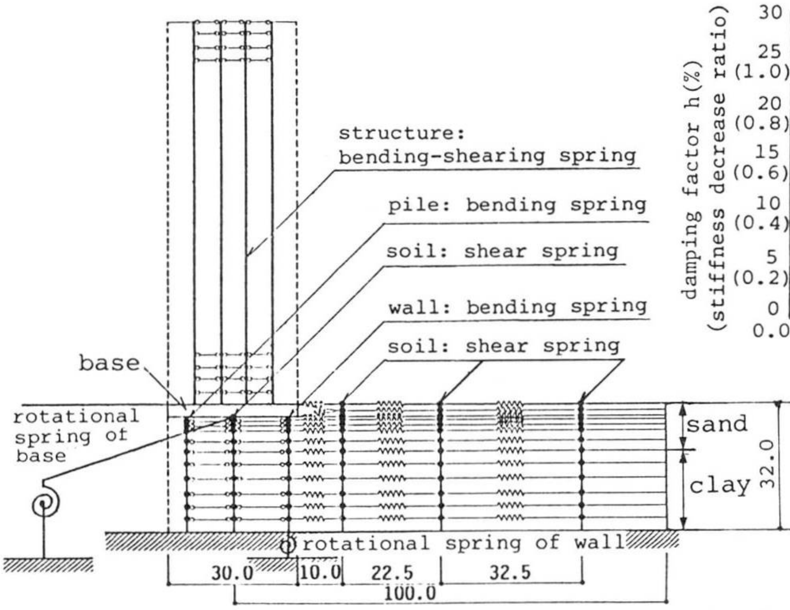


Fig.5 Modelling in earthquake response analysis

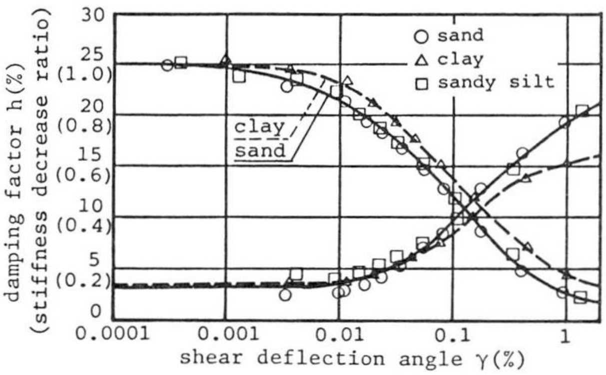


Fig.6 Dynamic characteristics of sub-soil

4. SAFETY CONFIRMATION

Safeness of this building against earthquakes are confirmed by the response analysis considering soil-pile-structure interaction as well as by the structural test of the component members.

4.1 EARTHQUAKE RESPONSE ANALYSIS

The vibration model used in the earthquake response analysis is shown in Fig.5, where bending-shearing springs equivalent to superstructure's framings are linked with each other using rigid floor assumption while shearing springs representing subsoil, are also connected by axial soil springs. Stiffness and damping factor of these subsoil elements are evaluated as shown in Fig.6 from experimental data using triaxial testing on boring samples from the construction site. Subsoil diaphragm and piles are also taken into account in the modelling, where rotational and horizontal movements of the foundation are defined by the bending and axial movements of these substructures.

The input earthquake wave is the EW component of the SENDAI TH038-1 which is one of the severest earthquake records in Japan. Intensity of the motion is defined to be 250 gals and 400 gals in maximum accelerations at the ground surface.

By the eigenvalue analysis on this model, it is recognized that the fundamental vibration periods are 1.85 second and 0.75 second which correspond to those of building and subsoil respectively. From the results of nonlinear response analysis in case of the earthquake with maximum acceleration of 250 gal, maximum responses are shown in Fig.7 and 8 as examples in the form of envelopes of shearing force and overturning moment that occured on each mass point. Stresses of members induced by these dynamic forces are recognized to be within allowable limits.

Nonlinear dynamic response analysis against the earthquake with maximum intensity of 400 gals is also conducted and the sufficient safeness of the designed building structure is confirmed in view of both strength and ductility.

4.2 Structural Test

Although previous testings on more than 100 specimens of structural members confirmed the appropriateness of design method [4], columns with additional core-reinforcing bars newly introduced in this building should be also confirmed using high strength concrete. Two specimens representing exterior columns on 2nd story are approximately 1/3 scale of actual members as shown in Fig.9. and are subjected to both horizontal and vertical earthquake forces after the axial fixed loading. No.1 specimen is loaded until final horizontal distortion under tensile field, while No.2 under compressive one.

Result of test shown in Fig.10 clarifies that the tensile column preserves its horizontal strength up to 2.5 times of the design shear stress with the deformation angle of 0.04 radian. Compressive column shown in Fig.11 reaches its horizontal ultimate strength under the deformation angle of 0.01 radian and also preserve its vertical load carrying capacity even until the deformation of 0.02 radian, which both prove sufficient abilities for an aseismic structural member.

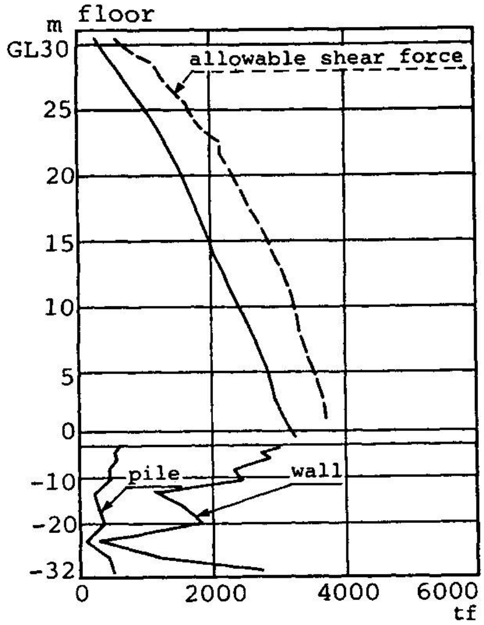


Fig.7 Maximum response shear force

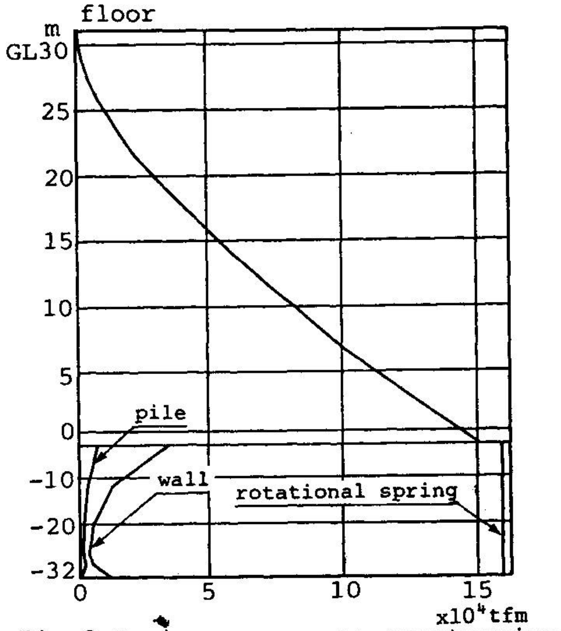


Fig.8 Maximum response overturning moment and bending moment

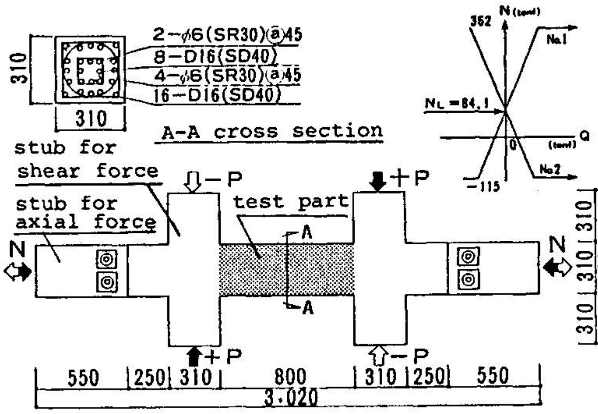


Fig.9 Specimen and loading pattern

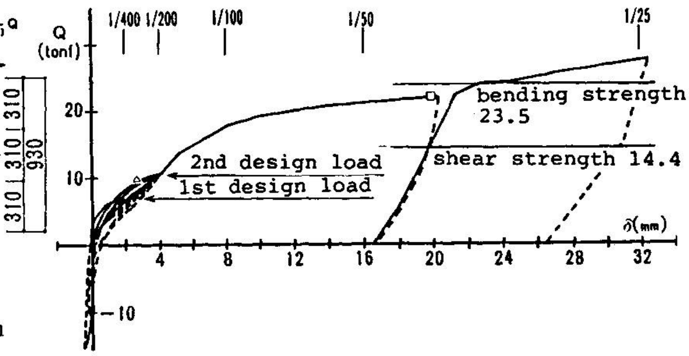


Fig.10 Load-deflection curve of No.1

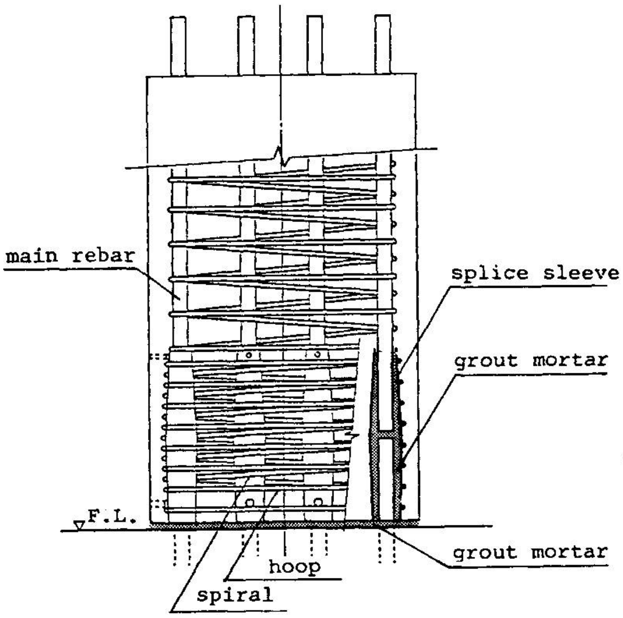


Fig.12 Pre-Cast Column

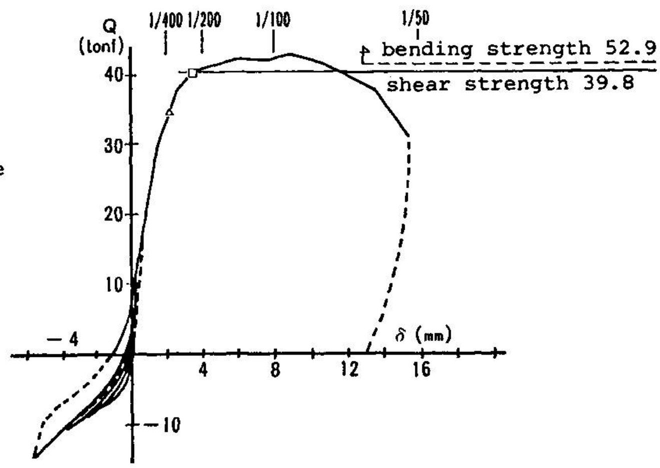


Fig.11 Load-deflection curve of No.2

5. INNOVATION IN CONSTRUCTION

Construction method in HiRC includes many sophiscated techniques such as prefabrication of reinforcing bars, U shaped anchorage, mechanical splices, integrated form work and so on. In this particular building, more over, upper storied column is to be made of pre-cast concrete as shown in Fig.12. The aim of this technique is to reduce the construction time from 8 days/floor to 7 days/floor as well as to conduct better quality control of whole works. As this application is also unprecedented in Japan, laboratory testings on structural behaviors are also conducted as well as on site testing on its practicabilities.

6. CONCLUDING REMARKS

Under very severe natural conditions, it needs long-term efforts in research and development to realization of building a tall reinforced concrete structure. Innovations in materials and methods are indispensable, but, they contribute fully to benefit not only to those concerned but also to general public. Using the methods described above, one of the urban renewal of the City of Kawasaki is to be fulfilled in coming year.

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Retrofitting of Reinforced Concrete Structures with Steel Bracing Systems

Restauration de structures en béton armé avec des contreventements métalliques

Verstärkung von Stahlbetonbauten mit Stahlverbänden

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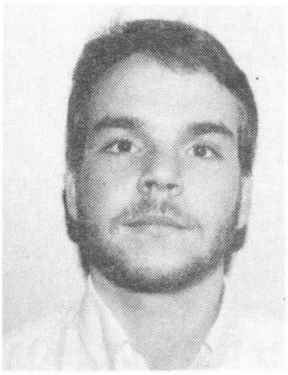

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SUMMARY
Steel bracing systems have been used for the retrofitting of reinforced concrete buildings with seismically inadequate lateral resistance. Applications of this retrofitting technique are presented and the main aspects of the design of the bracing system are discussed. The behavior under inelastic cyclic lateral deformations of a steel braced frame with weak columns is investigated with an analytical model.

RÉSUMÉ
Certaines structures en béton armé ayant présenté une mauvaise résistance latérale aux séismes ont été restaurées au moyen de contreventements métalliques. Trois applications pratiques de cette technique sont présentées et les principaux aspects du calcul du système de contreventement sont discutés. Le comportement d'un cadre contreventé soumis à des déformations latérales inélastiques est étudié à l'aide d'un modèle analytique.

ZUSAMMENFASSUNG
Für die Verstärkung von Stahlbetonbauten mit ungenügendem Horizontalwiderstand gegen Erdbebeneinwirkungen können Stahlverbände verwendet werden. Drei praktische Anwendungen dieser Technik werden vorgestellt und die prinzipiellen Aspekte der Bemessung solcher Verbände werden besprochen. Anhand eines analytischen Modells wird das Verhalten eines mit Stahlstreben verstärkten Rahmens unter nicht elastischen, zyklischen Verformungen untersucht.

1. INTRODUCTION

A large number of existing reinforced concrete frame structures are in need of seismic retrofitting because of inadequate lateral resistance. The inadequacy typically results from a poor design, a change in usage, or a change in design loads subsequent to the original construction. One promising retrofitting scheme uses diagonal steel bracing to strengthen and stiffen the structure. The bracing system is typically attached to the perimeter frames. By working on the exterior of the building, disruptions are minimized during and after construction.

2. APPLICATIONS

Steel bracing has been used for retrofitting inadequate structures as well as for repairing damaged structures. A well-publicized example is the five story Japanese school building shown in Fig. 1 [1]. The perimeter frames consist of deep spandrel beams and short columns damaged in the 1978 Tokachi-Oki earthquake. The damaged frames were retrofitted with a bracing system detailed to provide maximimum energy dissipation under cyclic inelastic loading. Retrofitting also included altering the deep spandrel beams in order to transform the brittle "weak column-strong beam" frame into a ductile "strong column-weak beam" frame.

Fig. 2 shows a twelve story medical building in Mexico City, which suffered structural damage to column and beams of the first three stories during the 1979 Petatlan earthquake and was subsequently repaired and strengthened with external steel trusses [2]. The trusses feature steel columns to resist high overturning moments. The slabs were strengthened to transmit the seismic shear forces to the new bracing system. The foundations of the perimeter frames were strengthened with steel piles. The project was completed in ten months at approximately 20 percent of the replacement cost of the building and with minimal disturbance to the users. Unlike many surrounding buildings, the retrofitted structure performed very well in the devastating 1985 earthquake.

The building of Fig. 4, the Zaragoza Hospital in Mexico City, suffered only minor damage in the 1985 earthquake. To reduce the possibility of damage in future events, diagonally braced rectangular steel frames are being added in the bays of the perimeter frames. The prefabricated steel bracing units are positioned and concrete is cast between the steel bracing unit and the existing concrete frame. Shear is transferred by dowels welded to the bracing unit and epoxy grouted into the concrete frame. The braces are square built-up sections designed to yield rather than buckle in compression.

The use of the bracing retrofit technique has also been considered in the U.S., particularly for cases similar to the Sendai School, which is typical of many buildings constructed in the 1950's in seismic zones. The behavior of a frame featuring weak short columns and strong beams retrofitted with a steel bracing system was investigated experimentally using a large scale model shown in Fig. 3 [3]. The experimental work was coupled with the analytical study presented in the next section.

3. ANALYTICAL STUDY OF A STEEL BRACED FRAME

3.1 Model

To further understanding seismic behavior of a steel braced frame, the lateral load-drift relationship under cyclic loading of a simple model was analyzed.

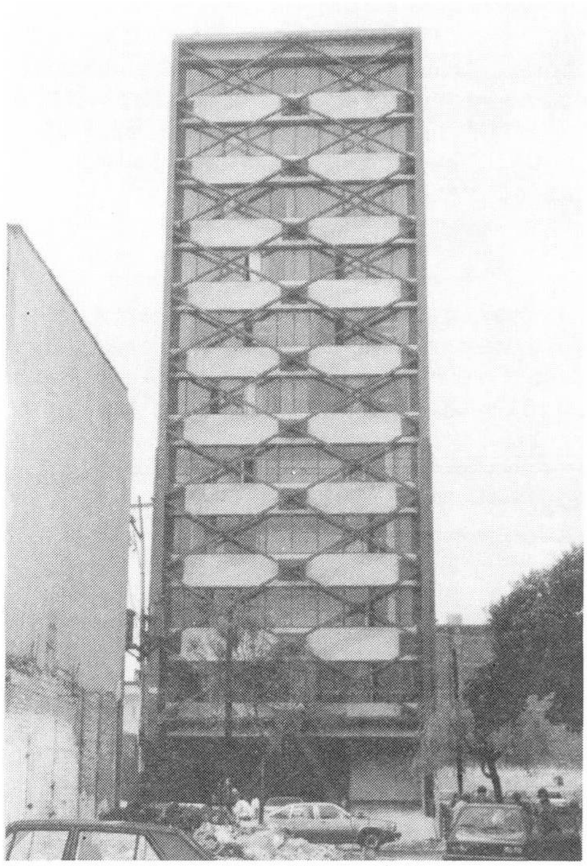
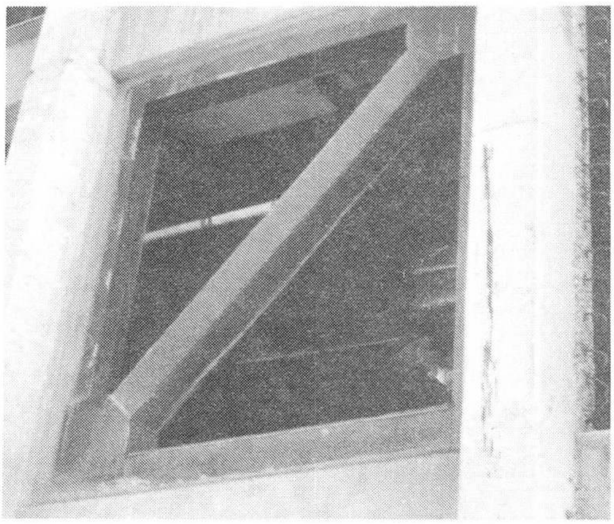


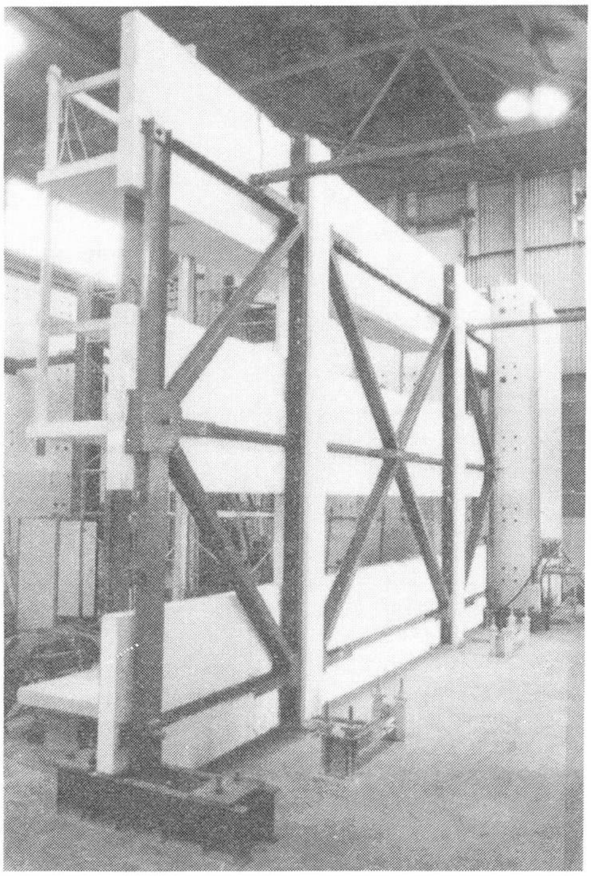
Fig. 1 Sendai School, Japan [1]

Fig. 2 Durango 49 Building, Mexico City [2,4]

Fig. 3 Experimental test of a steel braced frame [3]

Fig. 4 Zaragoza Hospital, Mex. C. [4]





The analytical model features two beams and a column simulating the frame of Fig. 5 braced as shown in the sketch in Fig. 6. The support conditions were chosen to reproduce boundary conditions of an interior column of the braced frame under lateral loading. The analytical models for the hysteretic behavior of the columns, beams, and braces were based on experimental results [5,6]. The computer program was checked using data from the experimental braced frame test.

3.2 Computed Response

In the analytical study, the slenderness ratio (kl/r) and the strength was varied and the model was analyzed for various cyclic loadings [4]. In the case described here, the braces have a slenderness of 80 and were designed to double the strength of the structure. The model was subjected to a single cycle of loading with reversal at large inelastic drift. The contribution of the frame and braces to the lateral resistance of the braced frame is shown in Fig. 5. The failure sequence is shown in Fig. 6. Important points in the response are numbered. First, the columns (1) and then the beams (3) crack. The bracing system remains elastic until the compression brace B2 buckles (4) at a drift of 0.20 percent and at 85 percent of the peak strength. The columns fail in shear (5) at a drift of 0.40 percent, the braced frame then loses strength until point 7, when the column has no lateral capacity in either loading direction. From point 7 onward, the frame no longer contributes to the lateral resistance and the bracing system controls behavior. Following the loading reversal (8), brace B1 is loaded in compression, buckles at point 9, and reaches post-buckling capacity (11) before brace B2 yields (12). When the load is reversed (13), brace B2 buckles for the second time when it reaches the post-buckling load level (14), and brace B1 yields (16) at a large drift level. The large reduction of strength and stiffness between first and

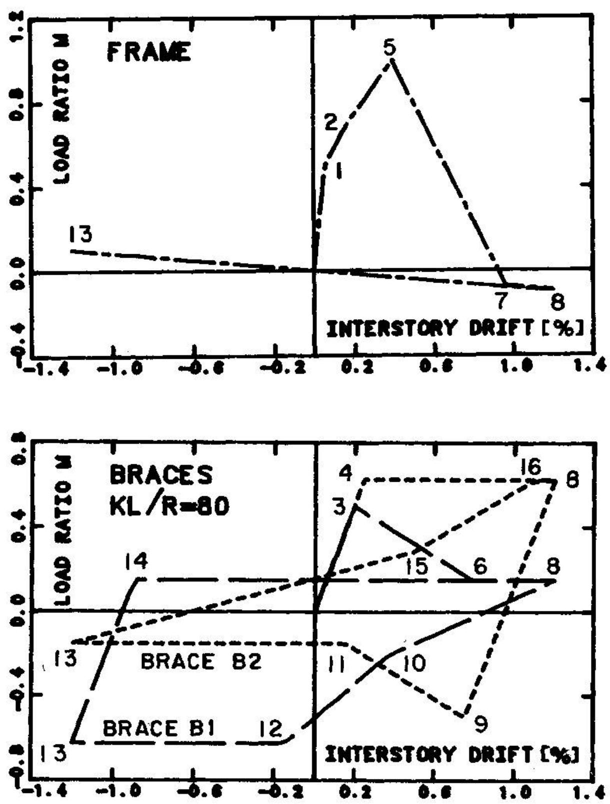


Fig. 5 Frame and bracing system under cyclic loading [4]

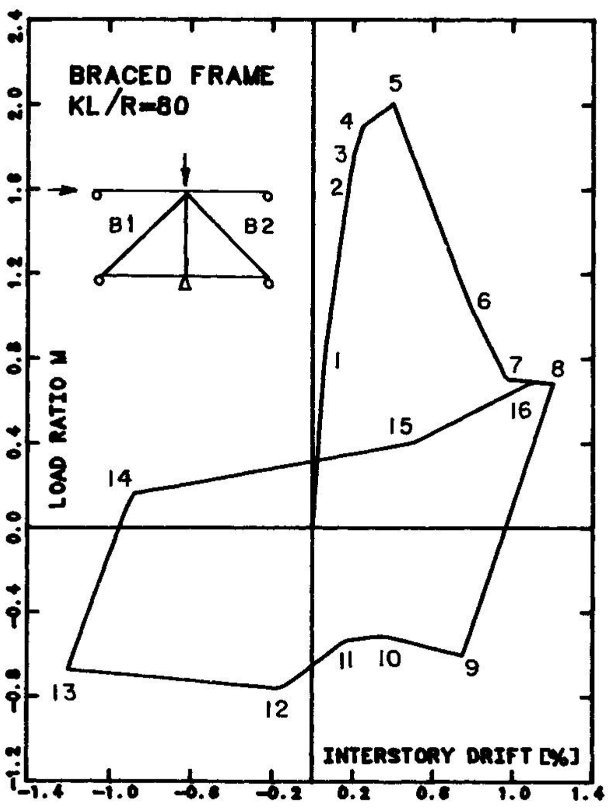


Fig. 6 Braced frame under cyclic loading [4]

second loading in the initial direction is due to loss of short column capacity and to buckling of the braces.

3.3 Behavior

Inelastic buckling of the braces is the main problem in designing a steel braced system. As was observed in experimental tests [3], the alternate buckling and yielding of the brace is linked with large local deformations at the brace connections which may lead to failure. Buckling also limits energy dissipation in the bracing system. One way to avoid inelastic buckling is to use very low slenderness ratios to guarantee that the braces yield rather than buckle in compression. Another way to prevent inelastic buckling is to use braces which buckle elastically, such as cables. The cables could be prestressed to improve the serviceability behavior. Alternatively, the bracing system may be designed to remain elastic which, in addition to preventing buckling, has the advantage of limiting drift during an earthquake. The steel bracing scheme is actually very well-suited for elastic design, since most of the added strength is in the elastic range for reasonable brace slenderness ($kl/r < 80$). The performance of the braced frame is optimized if the bracing system and the frame are well-matched in terms of their relative deformability. In Fig. 5 the two systems are well-matched, since the columns do not suffer substantial damage before they reach peak elastic strength.

4. THE RETROFITTING PROCESS

4.1 Decision to Brace a Structure

The main steps in the process leading to retrofitting a structure with a steel bracing scheme are outlined in the flowchart of Fig. 7. The evaluation of the seismic adequacy of the structure (step 1) consists of comparing performance requirements with expected behavior under seismic loads. If the structure is found inadequate (step 2), the owner must choose between retrofitting or replacement (step 3). The retrofitting scheme must be designed to correct deficiencies in the existing structure; that is, lack of strength, stiffness, or ductility (step 4). The retrofitting scheme should also be considered in terms of its impact on aesthetic qualities and on the usability of the building during and after construction. The rest of the flowchart is for the case where a steel bracing scheme best satisfies the requirements defined in step 4. Bracing may be combined with other retrofitting techniques. For example, bracing of perimeter frames may be used with column strengthening or infill shear walls in interior frames.

4.2 Design of the Bracing System

The choice of the bracing system configuration (step 6) includes selecting frames and bays to be braced and selecting bracing patterns.

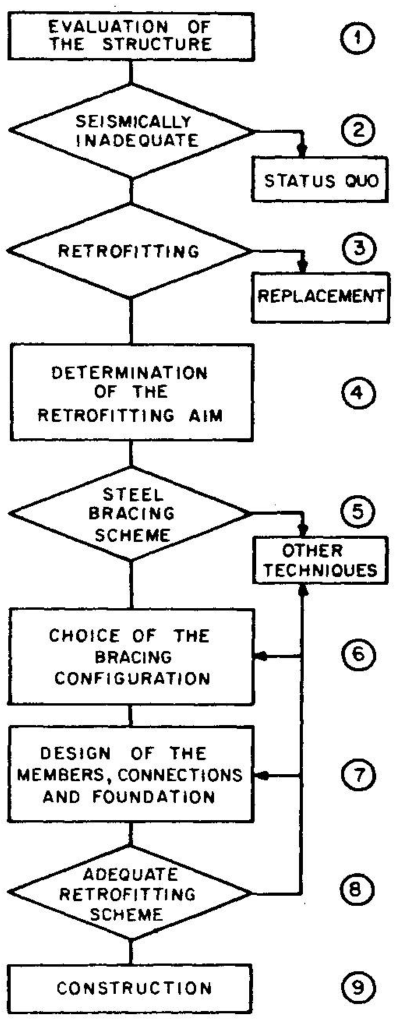


Fig. 7 Flowchart for the retrofitting process [4]

Changes in the force distribution in the existing structure must be considered to avoid overloading certain members, or introducing torsional eccentricities in the plan of the structure, or within the braced frames. Once a configuration has been chosen, the bracing system can be designed and detailed (step 7). To maximize the drift range in which the braced frame responds elastically, brace slenderness should be low and drift levels at which the frame and the bracing system suffer significant damage should be kept as similar as possible. If columns function as vertical elements of the bracing system, they must be able to carry the additional loads. Connections of the bracing system must be detailed (welds, bolted joints) carefully to avoid local failures under inelastic cyclic deformations. The foundations of the braced frames may need strengthening because the retrofitted structure typically imposes greater forces on foundations. In the construction phase of a retrofitting scheme (step 8), allowance should be made for higher fitting tolerances and for in-situ modifications.

6. CONCLUSION

Steel bracing systems are very well-suited for retrofitting operations aimed toward strengthening and/or stiffening reinforced concrete structures with inadequate lateral resistance. The main advantage of the technique is that strength and stiffness can be adjusted to achieve a variety of design objectives because the bracing system is independent of the existing frame. The retrofitted structure can be designed to respond primarily in the elastic range, thereby limiting damage which would occur under drift in the inelastic range. Problems associated with inelastic buckling may be alleviated by using braces which buckle elastically, such as cables.

ACKNOWLEDGMENT

Support of this study by the National Science Foundation is appreciated.

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Ouvrage à âmes plissées mis en place par poussage, Charolles, France

Viadukt mit gefalteter Stegbleche und dessen Ausführung im Taktschieben verfahren

Construction by the Incremental Launching Method of a Viaduct with Corrugated Steel Plate Webs

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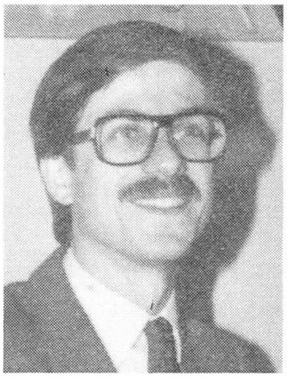

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RÉSUMÉ
Le viaduc de Maupré fait suite à la réalisation de trois ponts expérimentaux suscitée par la politique française d'innovation en génie civil. Son tablier est une poutre-caisson triangulaire dont les âmes, en tôle d'acier plissée, sont inclinées à 45 degrés pour converger vers un tube métallique rempli de béton qui constitue le hourdis inférieur. Une préflexion d'ensemble et une précontrainte centrée par câbles permettent le poussage du tablier avec ses superstructures. Des câbles extérieurs au tracé trapézoïdal complètent en service la précontrainte de l'ouvrage.

ZUSAMMENFASSUNG
Der "Viaduc de Maupré" wird als vierte Versuchsbrücke im Rahmen der französischen "Politique d'Innovation" erstellt, die Neuentwicklungen auf dem Gebiete des Bauwesens fördern soll. Den Brückenüberbau bildet ein dreieckförmiger Kastenträger, dessen Stege aus gefalteten Profilstahlblechen unter 45° geneigt sind. Diese werden auf den Untergurt, ein mit Beton gefülltes Stahlrohr, aufgeschweisst. Durch eine Gegenkrümmung des Stahlträgers vor dem Betonieren der Fahrbahnplatte und durch eine zentrische Vorspannung des Ueberbaus wird die Ausführung im Taktschiebeverfahren erleichtert Zusätzliche Spannkabel, die trapezoidal im Innern des Hohlkastens geführt sind, ergänzen die Vorspannung für die Einwirkungen im Gebrauchszustand.

SUMMARY
As part of the French innovative programme, three different solutions for bridge decks were recently investigated. One of these techniques was selected for the construction of the Maupré Viaduct. The bridge deck is a triangular box girder. The webs sloped at 45° are made of corrugated steel plate. These webs are welded on a steel tube acting as a bottom flange. This tube is grouted with concrete. Two kinds of longitudinal prestressing tendons, external to concrete, were used : straight tendons stressed span by span ; continuous tendons with a trapezoidal profile. The construction method was by incremental launching.

1. CONSTRUCTION DE PONTS EN OSSATURE MIXTE PRECONTRAINTE

Après un certain nombre d'études théoriques et expérimentales (3), on a construit en France plusieurs ponts dont les tabliers sont constitués d'une ossature mixte, comportant deux membrures en béton reliées par des âmes en acier et précontraintes par des câbles longitudinaux extérieurs.

Le passage supérieur N° 8 de l'Autoroute Orléans Clermont-Ferrand a un tablier de ce type, pour lequel les âmes sont des tôles planes de 12 mm d'épaisseur, raidies verticalement et longitudinalement. Le tablier est une travée isostatique de 38 mètres de portée. Il a été construit par Fougerolle pour la société concessionnaire de l'Autoroute : Cofiroute. L'ouvrage est en service depuis l'été 1985.

Le pont d'Arbois a un tablier dont les âmes sont remplacées par des treillis plans de type Warren à montants verticaux, constitués de profilés HEB 300 en acier. Cet ouvrage comporte trois travées de 29,85 m, 40,40 m et 29,85 m de portée. Il a été construit par Dragages et Travaux Publics et la Société Générale d'Entreprise, pour le compte de l'Etat; il est en service depuis l'été 1986.

Le pont de Cognac, lui, possède des âmes en acier constituées d'une tôle de 8 mm d'épaisseur, plissées selon un motif trapézoïdal. Cet ouvrage comporte trois travées de 31,00 m, 43,00 m et 31,00 m de portée. Il a été construit par Campenon Bernard BTP pour le compte de l'Etat. Il est en service depuis l'automne 1986.

Ces trois ouvrages, de dimensions modestes, ont permis de valider les hypothèses de calcul mises au point lors des études précédentes.

2. LE VIADUC DU VALLON DE MAUPRE, A CHAROLLES

Pour poursuivre sa politique d'incitation à l'innovation, la Direction des Routes a choisi de lancer un appel d'offres à variantes larges pour la construction du viaduc du Vallon de Maupré, à Charolles. Les caractéristiques fonctionnelles et le parti de l'ouvrage étaient imposés, et les entreprises devaient proposer uniquement des solutions "innovantes" à tablier mixte précontraint.

Ce viaduc a une longueur de 324,45 m, il comporte sept travées; la plus grande portée est de 53,55 m.

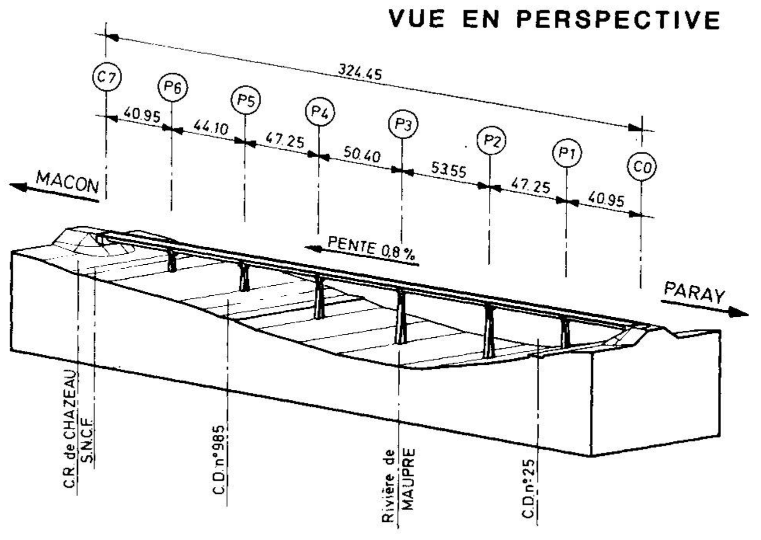

Fig. 1 : Vue perspective

Outre les deux solutions de base étudiées par l'administration, six solutions variantes ont été proposées par les entreprises. Le Maître d'Ouvrage a retenu la solution présentée par Campenon Bernard BTP : une poutre caisson triangulaire à âmes en tôle plissée.

3. LA STRUCTURE

La structure transversale du tablier du pont de Charolles est le résultat d'une évolution ayant pour origine celle du pont de Cognac. L'idée directrice a été de remplacer le hourdis inférieur en béton par une membrure en acier qui offre une résistance à la traction.

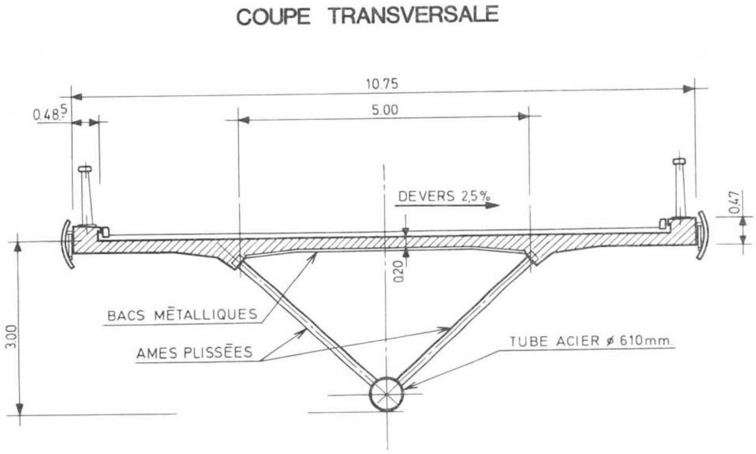

Fig. 2 : Coupe transversale

Le tube fut choisi pour son caractère de structure de révolution qui présentait une certaine souplesse du point de vue géométrique, et pour sa disponibilité directe sur le marché. Il est rempli de béton pour des raisons de déformabilité lors de l'application des réactions d'appui en cours de poussage.

La taille de ce tube, réduite parce qu'adaptée aux efforts exercés, conduit à faire converger les âmes plissées avec une inclinaison sur la verticale voisine de 45 degrés, constituant ainsi un triangle isocèle dont le hourdis supérieur en béton est la base. Cette forme a pour caractéristique de très bien fonctionner en torsion. La connexion entre âmes et hourdis supérieur est assurée par des tronçons de cornières disposées transversalement sur une platine; ces éléments sont assemblés par soudure. Le dimensionnement de cette connexion est déterminé pour la presque totalité par des dispositions de bonne construction vis à vis du comportement local.

Le V que forme la structure est soumis, sous l'action des charges verticales, à des efforts d'ouverture qui sont équilibrés par une précontrainte transversale, constituée de monotorons T15 GPE uniformément répartis dans le hourdis supérieur.

La totalité des éléments constitutifs du tablier représente, ramenée au mètre carré d'ouvrage, les quantités suivantes :

- béton : 0,27 m
- précontrainte : 19 kg
- acier passif de béton armé : 54 kg
- acier E36-4 de structure : 82 kg.

Cela correspond à un poids par mètre de structure brute courante de 7,7 t et à un poids par mètre de structure courante équipée de ses superstructures de 12 t.

4. LA MISE EN ŒUVRE

La structure métallique de l'ouvrage est préfabriquée et prépeinte en atelier par tronçons d'environ 12 mètres de longueur. Un bac métallique nervuré HI-BOND qui constituera le coffrage de la dalle centrale ferme le triangle structurel. Ces tronçons sont livrés sur chantier par camions semi-remorques et mis en position à la grue sur le banc de poussage.

Le banc est composé d'une poutre unique cintrée, à concavité vers le bas, suivant un rayon de 7 000 mètres permettant d'obtenir une préflexion du tablier lors de l'assemblage des tronçons d'ouvrage. Ces tronçons sont soudés par deux, puis par quatre pour

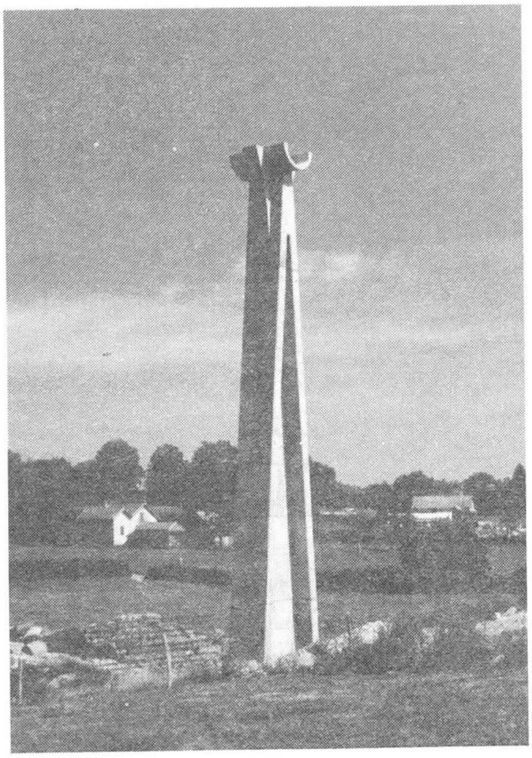

Fig. 3 : Vue d'une pile

constituer une travée. Le hourdis supérieur de cette travée est alors ferraillé et bétonné.

La précontrainte de poussage est ensuite mise en tension. On pousse la travée entière avec ses superstructures, hormis l'étanchéité et le revêtement. Il s'agit en fait d'un tirage à partir de la culée avec un effort maximum de 170 KN.

C'est la première travée de la structure qui constitue l'avant bec; pour cela le bac acier est remplacé par une tôle raidie épousant la forme du coffrage. Cet avant bec est complété par une prolongation sur 7 mètres du tube inférieur. Cet élément est équipé d'un dispositif d'accostage de grande amplitude susceptible de reprendre une flèche métrique.

En cours de poussage, la stabilisation transversale de la structure est assurée : sur le banc, par des étais roulant sur des chemins qui lui sont parallèles; sur les appuis définitifs, par des étais équipés de galets.

Les rouleurs sans cesse stabilisés transversalement supportent le tablier sur le banc. Les appuis définitifs sont équipés de profilés de 2,50 m de longueur, montés sur hydraulique, sur lesquels glissent les appareils d'appui de poussage qui épousent la forme du tube.

5. LA PRECONTRAINTE - LA PREFLEXION - LES ENTRETOISES

En plus du câblage de poussage composé de 12 à 14 câbles 6T13 centrés, régnant sur une travée d'appui à appui, l'ouvrage est précontraint longitudinalement par 4 câbles 19T15 tendus d'un bout à l'autre du tablier et ancrés dans les blocs d'about en béton.

Le câblage retenu, pseudo-funiculaire partiel des charges permanentes, permet d'obtenir pour une faible force de précontrainte des compressions suffisantes sur appui pour que le hourdis supérieur soit toujours comprimé en service.

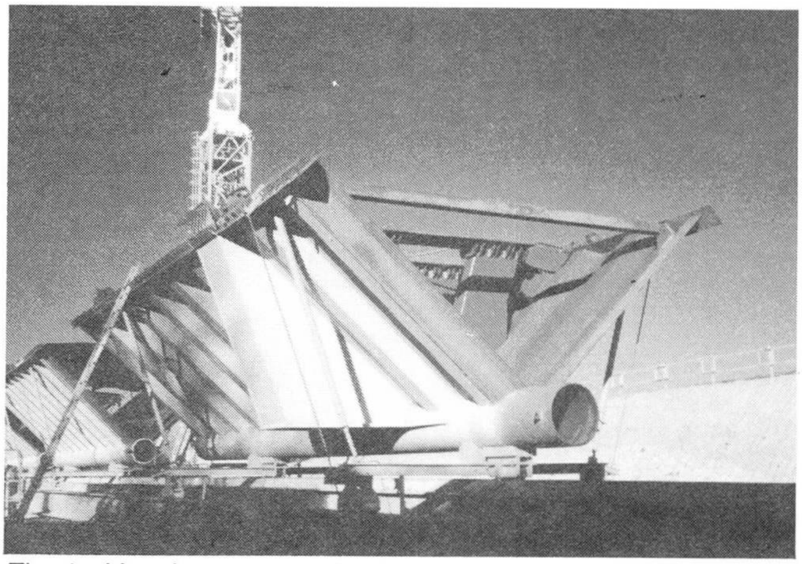

Fig. 4 : Vue d'un tronçon de charpente

Ces câbles 19T15 ont entre appuis un tracé trapézoïdal; ils changent de direction en travée, dans des déviateurs métalliques solidaires des âmes et du tube inférieur, par l'intermédiaire de tubes cintrés en acier. Les pertes de tension dues à leur grande longueur sont compensées par une opération de retension par modification du tracé. La selle de déviation sur appui central est rehaussée d'environ 55 centimètres par vérinage depuis le dessus du tablier.

L'ouvrage étant assez souple, ces câbles, considérés comme des haubans intérieurs au caisson, sont soumis à des variations de tension sous charges. Pour éliminer les risques de fatigue qui en découlent, on réalise des dispositifs qui empêchent le câble de glisser dans sa gaine au niveau des déviations.

La préflexion donnée au tablier, par la géométrie qui lui est imposée sur le banc, est une technique qui permet d'effacer les effets du retrait du hourdis supérieur en comprimant celui-ci d'environ 0,5 MPa, la traction concomitante du tube inférieur étant bénéfique sur appui, et sans conséquence en travée.

La structure triangulaire pose un problème de transmission des efforts au niveau des piles. La solution retenue consiste à adjoindre au caisson des entretoises métalliques en forme d'oreilles qui reconstituent localement l'allure d'un caisson classique à deux hourdis parallèles, transmettant ainsi les charges aux piles par l'intermédiaire d'appuis en élastomère fretté.

6. DISPOSITIF EXPERIMENTAL

L'ouvrage fait l'objet d'une expérimentation. Ce dispositif a pour objectifs le contrôle des paramètres suivants:

- répartition des contraintes de flexion longitudinale entre hourdis supérieur, âmes, tube métallique et béton du tube,
- fonctionnement du tube rempli de béton,
- déformabilité de la structure en cours de poussage et en service,
- étude des tensions dans les câbles de précontrainte de 325 mètres.

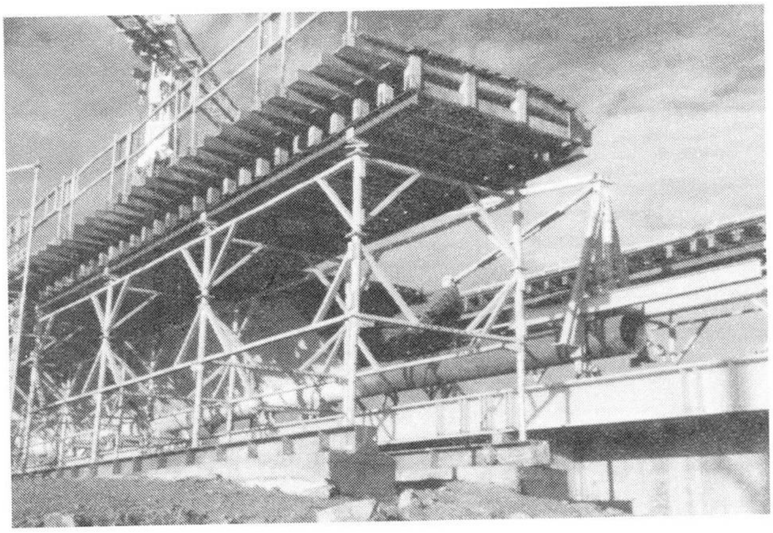

Fig. 5 : Vue du dispositif d'accostage

Dans ce but, une section a été complètement équipée de jauges de déformation collées sur les tôles ou sur les aciers passifs. L'un des appareils d'appui de poussage sera muni de pesons. Enfin des mesures de flèche de précision seront effectuées lors des essais de l'ouvrage.

7. LECONS ET PERSPECTIVES

Une opération paraissait délicate à mener à bien avant fabrication : l'accostage et le soudage de l'âme plissée sur le tube. Celle-ci s'est en fait déroulée dans d'excellentes conditions.

De manière plus générale, on peut dire que la forme triangulaire est assez coûteuse en matière, et que la méthode de construction approche de la portée limite pour ce type de structure.

Il convient néanmoins de rappeler que le poussage de la structure métallique a été réalisé avec la dalle en béton – ce qui n'est pas classique pour les ossatures mixtes traditionnelles – et de plus avec les superstructures hormis étanchéité et revêtement.

POUSSAGE : STABILITE SUR PILES

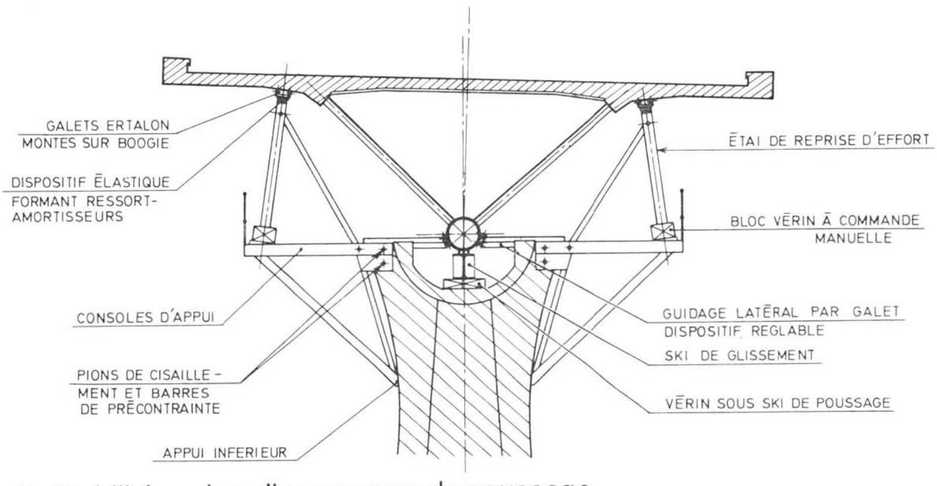

Fig. 6 : Stabilité sur les piles en cours de poussage

Les âmes plissées ont donc démontré, pour cet ouvrage poussé, comme pour le Pont de Cognac construit sur cintre, leur grande fiabilité. Il faudra mettre en œuvre de nouvelles méthodes de construction pour aborder les grandes portées, domaine naturel de ce type de structure.

Pour que cet avenir se concrétise, il est souhaitable que les maîtres d'ouvrage rendent possible la construction de ponts de grande portée de ce type, de façon à permettre aux âmes plissées de trouver leur domaine d'efficacité, et de démontrer dans ce cas leur compétitivité structurelle et économique, dans l'intérêt de tous les intervenants.

8. INTERVENANTS

- Maître d'Ouvrage : Etat, Ministère de l'Equipement, du Logement et des Transports
- Maître d'Oeuvre : Direction Départementale de l'Equipement de Saône et Loire
- Auteur de la conception et du projet d'exécution : Bureau d'Etude de Campenon Bernard BTP
- Contrôle du projet et assistance technique du Maître d'Oeuvre : SETRA et CETE de Lyon.
- Entreprise : Campenon Bernard BTP - Direction des Ouvrages d'Art Spéciaux
- Sous-traitant chargé de l'exécution des parties métalliques : Strasbourg Entreprises
- Sous-traitant chargé de l'exécution des piles et culées : Maillard et Duclos
- Sous-traitant chargé de l'exécution des pieux : SEPICOS
- Dispositif expérimental : LCPC et LRPC d'Autun

COUPE LONGITUDINALE-TYPE D'UNE DEMI-TRAVEE

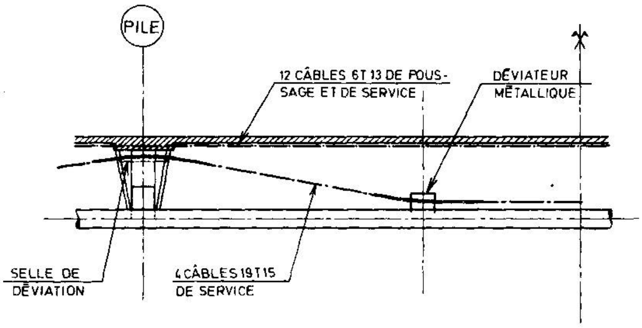

Fig. 7 : Coupe longitudinale type d'une demi-travée

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Composite Structure of Concrete and Steel Plate

Structure composite en béton et tôle d'acier

Verbundbauteile aus Beton und Stahlplatten

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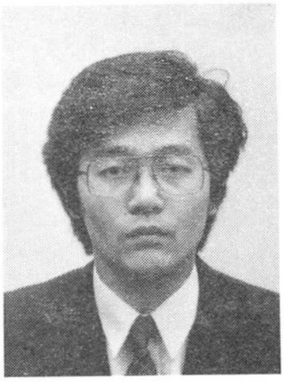

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SUMMARY
The composite structure of concrete and steel plate without reinforcing bar has various advantages compared with reinforced concrete. The steel plate, which acts also as the formwork, resists external forces together with the concrete. The rib used as the stiffener to ensure the rigidity of formwork acts as the shear connector. Use of a flanged rib improves the integrity of the concrete and steel plate. Experiments with composite beams using ribs as shear connectors revealed some fundamental factors relating to rib shape and rib arrangement in design of composite structure member, the results of which are presented.

RÉSUMÉ
La structure composite en béton et tôle d'acier, sans barres d'armature, présente quelques avantages par rapport au béton armé. La tôle d'acier, qui agit aussi comme coffrage, résiste aux forces extérieures en collaboration avec le béton. La nervure, utilisée comme raidisseur pour assurer la rigidité du coffrage, agit aussi comme goujon. L'emploi d'une aile nervurée améliore le comportement de l'élément mixte. Des expériences réalisées avec des poutres composites utilisant les nervures comme goujons ont montré l'importance fondamentale de l'emplacement et de la forme de ces nervures. Les résultats en sont présentés.

ZUSAMMENFASSUNG
Verbundbauteile bestehend aus einer Stahlplatte und unbewehrtem Beton haben verschiedene Vorteile gegenüber normal bewehrten Stahlbetonbauteilen. Die Stahlplatte dient auch als Schalung. Die aufgeschweissten Steifer geben der Stahlplatte eine grössere Steifigkeit für ihre Funktion als Schalung und dienen gleichzeitig als Schubdübel im Verbund mit dem Beton. Die Verwendung von Rippen mit Flansch verbessert das Verbundverhalten. Versuche mit solchen Trägern zeigten die fundamentale Bedeutung von Rippenanordnung und Rippenform auf. Die Ergebnisse dieser Versuche werden besprochen.

1. INTRODUCTION

In Japan, with the increase in urban population and to make better use of space, need of expanding cities toward coast or even to offshore, enlarging the unit of construction, and building larger structures by demolishing old ones. On the other hand, in the case of construction work in and around the city, not only usual safety and non-interference of urban lives are required but, especially in Japan, lack of on-site preparation area has become a matter of grave concern.

From these points of view, as well as the development of new efficient construction methods, the development and plans of new type structures have increased. For instance, lately, the submerged sewage treatment plant of reinforced concrete structure was constructed by improving the former technique of the submerged tunnel. The erection of steel parts and the concrete casting of bottom slab were carried out at a shipyard, and after flooding, the concrete of wall and top slab was cast with the body floating. The caisson thus completed was towed by tugboats from the shipyard to the project site, and was placed on the previously built seabed foundation.

The composite structure of concrete and steel plate concerned in this paper has various advantages and problems that are not seen in the reinforced concrete structure or the steel structure, as shown in Table 1. These advantages should make this kind of composite particularly suited for the following structures : (1) offshore structures, submerged structures, or bridge slabs, (2) tanks, vessels, or containers that needed tightness, and (3) structures carrying huge loads.

The greatest advantage of the composite structure over reinforced concrete is that the steel plate itself acts as the formwork, whose rigidity is further reinforced by the ribs, which act not only as shear connectors but also as stiffeners. All these combined make the structure economical.

The authors have carried out experiments on the composite beam using rib as the shear connector to create a composite action between concrete and steel plate. Also, some fundamental considerations related to rib shape and rib arrangement in design of composite structure member are presented.

2. DESIGN PROCEDURE

The design procedure of composite structure is shown in Fig. 1. Firstly, the steel shell (steel plate thickness, dimension and arrangement of stiffeners and web plates, etc.), is designed against the construction load, which consists in the external load, including the load from unhardened concrete, and the dead load. Of course, the construction load depends on the construction method. Nextly, the composite functions are reviewed with regard to the efficacy of compositioning, the efficiencies of stiffeners and the total strength against the service loads the composite is expected to carry, and the durability.

Should any of these be judged unsatisfactory, the design work is re-started from the steel shell designing again.

Table 1 Advantages and problems of composite structure compared with reinforced concrete structure

	Advantages		Problems
Construction	1. Formworks are not necessary. 2. Steel shell can be manufactured in a factory. 3. Bar arrangement is not necessary. 4. Concrete can be cast at the sea.	*Economy *Safety *High accuracy *Short of construction term *Reduction of land for construction	1. Stiffeners(ribs) are necessary to ensure the the rigidity of steel shell, making it rather difficult to fill up the space under the stiffener with concrete. 2. Calculation of stresses operating during construction is complicated. *Residual stress due to construction work must be considered as the permanent stress. *The order of concrete casting must be planned carefully.
Serviceability	1. The outer steel plate ensures the tightness 2. Concrete cover is not necessary. 3. Concrete does not fall off.	*Economy *Safety *Lightweight	1. There is possibility of increased deflection. 2. Buckling of the compression plate. 3. Corrosion of steel plate (durability).
Strength and Structural Behavior	1. High toughness with the sandwich type structure. 2. Concrete is confined by the steel plate.		1. The design of proper shape and arrangement of the stiffener (ribs) as shear connector is difficult. 2. The design method of the imperfect composite structure has not been established. 3. Influences of creep and shrinkage of concrete are not well known.

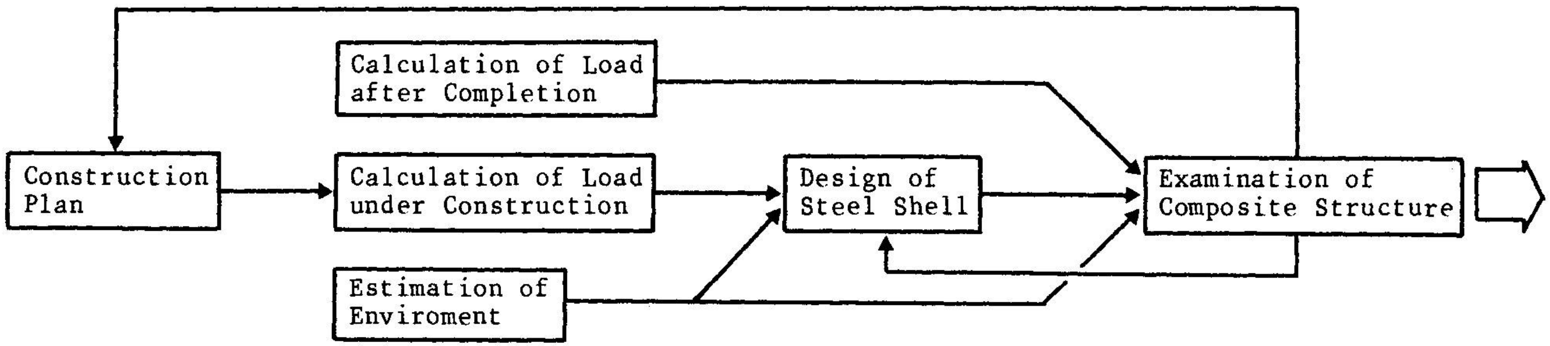


Fig. 1 Design Procedure

3. CONDITION FOR COMPOSITE FUNCTION AND STRUCTURAL BEHAVIOR

3.1 Shape and arrangement of rib

Since the shear is transferred between concrete and steel in the composite structure by the shear connector, the shape and the arrangement of shear connectors affect significantly the composite action. Since the stresses around the shear connector generally are highly concentrated, local copmressional failure or cracking of concrete may be likely. Therefore, it is realized that the shear strength may deteriorate markedly if the shape or arrangement of the shear connector is not correct.

3.1.1 Consideration on rib height for the composite action

Rib height should be greater than the largest aggregate in the concrete, and the compressive stress on the concrete enclosed in a rib should be smaller than the bearing strength.

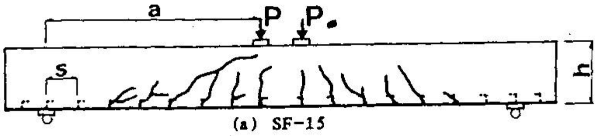
In the case that ribs are anchored in the tension zone of concrete, the rib height has no effect on the strength of the member as a whole and on the composite function (Fig. 2). But, when the rib height is too small, the concrete enclosed in a rib will fail under the bearing stress, and ribs will not work as the shear connector.

To demonstrate an extreme case, an experiment was carried out on a composite beam made up of concrete and checkered plate. The result obtained was that the beam failed simultaneously with occurrence of bending crack, and the concrete in contact with the checkered plate was pulverized at the places of stress concentration.

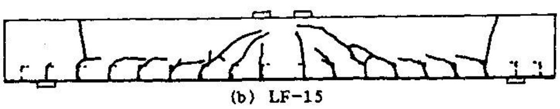
3.1.2 Necessity of rib-flange

The rib-flange is necessary for the rib to act as the shear connector. In general, the flange width should be larger than the size of the greatest aggregate.

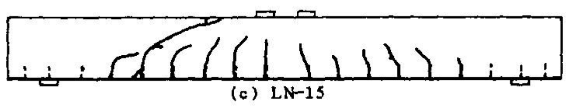
The shear connector should prevent the separation of concrete and steel plate, besides transferring the shearing force between them. If there is no flange on the rib, the bending crack from the top of rib is liable to occur early, and this bending crack will induce the diagonal tension crack in concrete (Fig. 2). From the experiments, the static shear strengths of beams with flanged ribs were higher than those of beams with unflanged ribs. On the other hand, however, the rib-flange is liable to induce almost horizontal cracks at the top of the flange, degrading the composite characteristic. And, when these horizontal cracks are linked together, the member will collapse (Fig. 2). Since the crack pattern is different with or without the rib-flange, the designer must judge which crack pattern is more harmful to the structure.



(a) SF-15



(b) LF-15



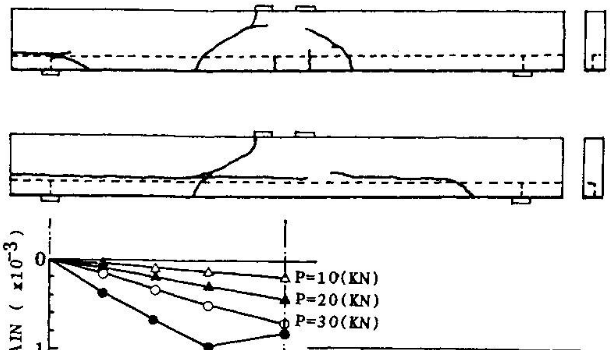
(c) LN-15

Specimen	Pmax (KN)	Rib (cm) Height	Flange	Space	a (cm)	h (cm)	f'c (MP)
SF-15	26.5	4.0	4.0	15.0	105	30	21.6
LF-15	29.4	7.5	4.0	15.0	105	30	22.1
LN-15	26.4	7.5	0	15.0	105	30	22.2

Fig.2 Effects of Rib height and Rib-flange on the strength and composite action

3.1.3 Effects of longitudinal rib

If the longitudinal rib has a proper flange, it functions as an excellent shear connector. However, as shown in Fig. 3, on account of tensile stress acting between concrete and rib-flange, it will give rise to a continuous weak plane along the rib-flange. When a crack propagates along this plane, the rib will lose its power.



Pmax (KN)	Rib (cm) Height	Flange	f'c (MP)
35.6	7.5	4.0	21.8

Fig. 3 Connection effects of longitudinal rib

3.1.4 Cares to be exercised on using longitudinal rib with transverse rib

If the longitudinal rib and the transverse rib are used together, concrete will be subjected to tensile forces from both the longitudinal and the transverse rib flanges. Therefore, their heights should not be the same, and the difference between the two need to be larger than the size of the greatest aggregate.

3.1.5 Shear reinforcement effects of transverse rib

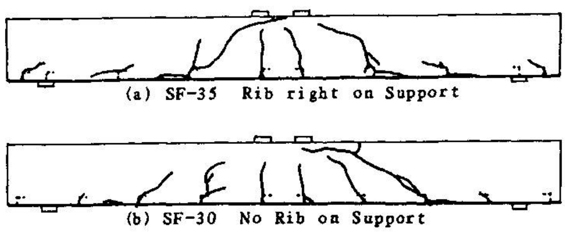
If the transverse ribs with flange are anchored in compressive zone of concrete, or the compression plate and the tension plate are connected by the transverse ribs (web), these ribs act as shear reinforcements as well as shear connectors. The shear mechanism in the member is like that in the truss action in which diagonal compressive struts equilibrate with transverse tensile ribs. Therefore, shear strength of this type of member is determined by the compressive strength of concrete or the tensile strength of transverse rib.

3.1.6 Shear reinforcement effects of longitudinal rib

If the longitudinal ribs with flange are anchored in compressive zone, or the compression plate and the tension plate are connected by the longitudinal ribs (web), these ribs also act as shear reinforcements. Vertical tensile stress in the rib varies like that of shear reinforcement (stirrup) of reinforced concrete with load increasing. But, in the case of longitudinal rib, the rib should carry the shear force (stress) from tension plate. Therefore, at the time of calculating the shear strength of longitudinal rib (Vs), the width of rib should be considered in assessing the influence of shear stress.

3.1.7 Effects of location and arrangement of transverse rib

In the case of large shear span members, the shear strength is equal to the diagonal tension crack load. And, in the composite member, cracking occurs from the top of all ribs. Therefore, when ribs are located so that the crack pattern is similar to that of reinforced concrete, the shear strength is equal to that of the reinforced concrete. As shown in Fig. 2, however, in the case the rib spacing is too short, the almost horizontal cracks are linked together easily, and the



Specimen	Pmax (KN)	Rib (cm) Height	Flange	Space	f'c (MP)
SF-35	35.2	7.5	4.0	35.0	25.4
SF-30	27.8	7.5	4.0	30.0	20.8

Fig. 4 Effects of rib arrangement on shear strength

composite action is lost early. On the other hand, in the case there exist a rib right on a support and few ribs are inside the shear span, compressive struts are formed in concrete from underneath of the load points to the supports, and the shear strength becomes larger than that of reinforced concrete, because the diagonal crack does not occur easily (Fig. 4). The mechanism of force transfer in this case is like in the tied-arch action with the concrete acting as the arch rib and the steel plate as the tie. But, as shown in 3.2, the deflection of member will be larger than that of reinforced concrete.

3.2 Deformation behavior

Deflection of composite member has been calculated by modifying the beam theory by rib spacing.

The bending stiffness of composite member under concentrated load will be $1/(1+s/a)$ as large as that calculated from the beam theory on the reinforced concrete (Fig. 5). Where, s is the rib space, and a the shear span.

From the experiments, the bond stress between concrete and steel plate becomes zero just about the time the flexure crack occur, and the axial strain of steel plate between two ribs becomes the same. Based on experimental results, the axial strain of steel plate can be assumed to be the solid line in Fig. 6 . The average (total) strain of steel plate in shear span is $(1+s/a)$ times the broken line value (perfect composite). Therefore, the deflection of this composite type member shall be $(1+s/a)$ as large as the perfect composite.

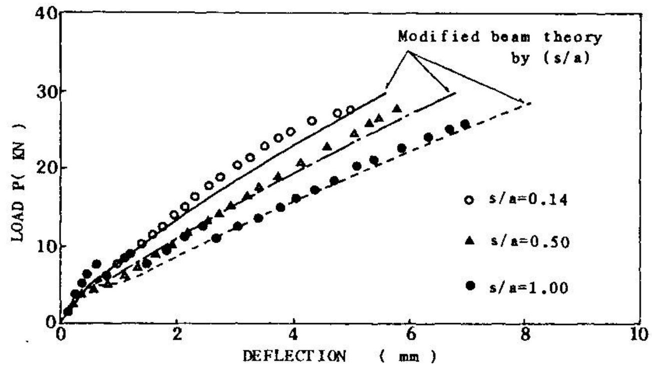


Fig. 5 Load - Deflection curve of composite beam

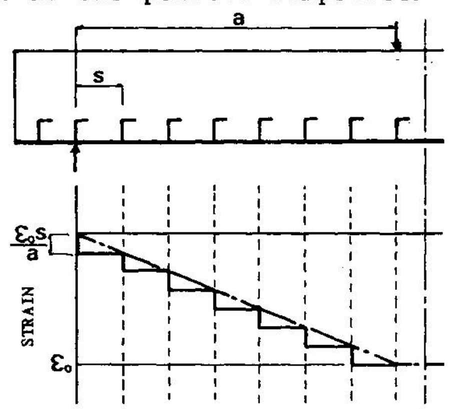


Fig. 6 Axial strain distribution of steel plate

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Concrete Filled Steel Bearing Walls

Murs porteurs constitués de parois d'acier et de béton

Tragwände in Stahlzellenverbundbauweise

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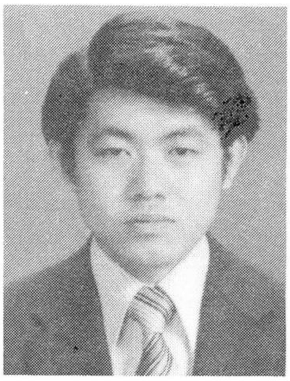

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SUMMARY
SC bearings walls are composed of box steel(S) and filled concrete (C). They are expected to reduce construction time and labor compared with reinforced concrete bearing walls with especially complicated re-bars. However, it is necessary to use comparatively thin steel plates for SC bearing walls from an economical view point. The objective of this investigation is to clarify the structural behavior of SC bearing walls with thin steel plates by axial and shear loading tests.

RÉSUMÉ
Les murs porteurs SC se composent du coffrage d'acier (S) rempli de béton (C). Comparés aux murs porteurs de béton armé avec des barres de répartition particulièrement compliquées, ils permettent de réduire le temps de construction et la main d'œuvre nécessaire. Mais pour des raisons économiques, il serait souhaitable de renforcer ces coffrages par de minces tôles d'acier. Le but de cette recherche est de déterminer le comportement structural de ces murs porteurs SC comportant de minces plaques d'acier par des tests de charges axiales et de cisaillement.

ZUSAMMENFASSUNG
Tragwände in Stahlzellenverbundbauweise bestehen aus einer mit Beton verfüllten Stahlzelle. Man erwartet von dieser Bauweise verkürzte Bauzeiten und eine Einsparung an Arbeitskräften im Vergleich zu den herkömmlichen Tragwänden aus Stahlbeton mit besonders komplizierter Bewehrung. Für eine Kostenersparnis ist allerdings die Verwendung verhältnismässig dünner Stahlplatten notwendig. Das Ziel dieser Untersuchung ist die Klärung des Tragverhaltens solcher Tragwände unter Druck- und Schubbeanspruchung.

1. INTRODUCTION

The concrete-filled steel (SC) bearing walls consist of elements of steel plate box units filled with concrete. Their structural behavior can be characterized by high strength and sufficient ductility owing to the composite effect of the steel plate and the concrete. By applying SC instead of reinforced concrete (RC) for structural constructions it can be foreseen that the arduous task of arranging and placing of rebars are eliminated, which should result in reducing construction time. This SC structural method should prove valuable to countries such as Japan which is located in high seismicity zone and is subjected to large seismic loads.

Regarding the SC bearing wall, the researches by Kato, Suzuki, et al.[1)2)] has confirmed, by basing on tests, that the relationship between loads and displacements show sufficient ductility when steel plates with width-thickness ratio of about 100 is used. They also show that the maximum shear strength could be estimated by an ultimate limit analysis.

The SC bearing walls proposed by the authors are constructed by assembling welded box steel units and filling the inside with concrete (see Fig. 1). The box units are composed of comparatively thin steel plates (width-thickness ratio of 200) in view of economical considerations. However, anxieties did exist that the composite effect to the SC bearing wall may not be adequate due to buckling of the thin steel plates at an early stress stage. In this paper, the authors describe the elasto-plastic structural behavior of the proposed SC bearing wall units by applying stiffening to the thin steel plates such as insert plates, L-shaped steel angle members and headed studs in loading tests. The tests conducted were compressive loadings and shear loadings.

2. OUTLINE OF THE PROPOSED SC BEARING WALLS

Process of the proposed SC bearing walls are 1) prefabricating a box unit consisting of two-surface plates and vertical and horinzontal insert plates, 2) assembling box units to the required length and width of the wall, 3) welding the joints of the boxes, and 4) filling the inside of the box units with concrete. The thickness of the proposed SC bearing wall is 1.25 m and the thickness of the steel plates are 12 mm. This possess equivalent strength and economy as a RC bearing wall which has wall thickness of 2 m and rebar reinforcement ratio of 1.2%. The outer dimensions of the box steel unit, which was determined in view of the convenience of construction and transportation, has the width of 2.5 m, and the height of 3 m (see Figs. 1 and 2). As a bearing wall, the proposed SC bearing wall has a smaller self-load than a RC bearing wall used in the same place because of its smaller cross section. Thus can contribute to reduce seismic loads.

The methods devised to prevent early stage buckling of the surface plate are as follows: 1) place one or two vertical insert plates in appropriate places

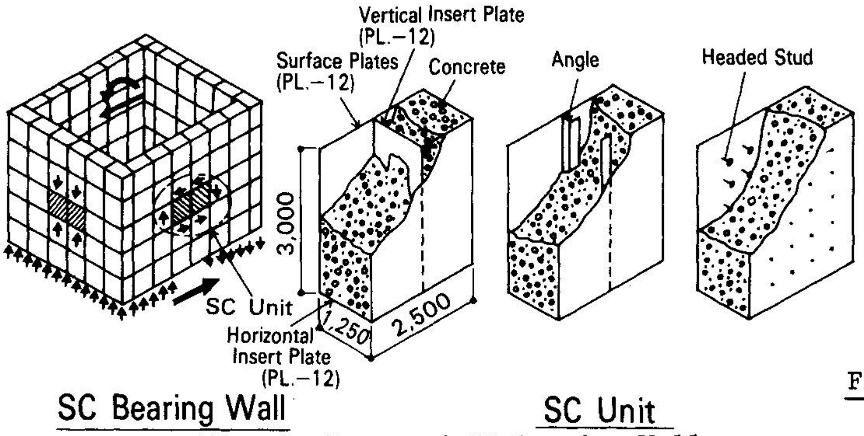


Fig. 1 Proposed SC Bearing Wall

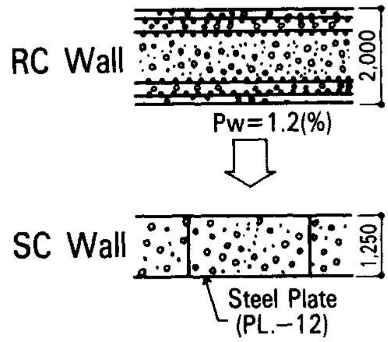


Fig. 2 Comparison of RC Wall and SC Wall

within the unit; 2) stiffen the surface plates by attaching four L-shaped steel angle members instead of the two vertical insert plates; 3) weld headed studs on the surface plates to conform with the AIJ standard for structural design of steel structures (width-thickness ratio ≤ 41)[3]; etc.

3. STRUCTURAL BEHAVIOR OF THIN STEEL PLATE SC BEARING WALLS

3.1 Outline of Loading Test Program

Table 1 and Figs. 3 and 4 show the specimens and their shapes. The specimens are approximately 1/4 scale-models of the proposed units of the SC wall. Seven respective specimens were tested each for compressive loadings and shear loadings to study the test parameters as follows.

- the composite effect of the steel plate and the concrete
- the effect of the insert plates interval (width-thickness ratio of the surface plates)
- the effect of the stiffening method to the surface plates

Table 2 and 3 show the mechanical properties of the materials.

3.2 Test Results

Figs. 5 and 6 show the compression and shear test results for each parameter. Table 4 show the final state of the surface plates and the filled concrete. Table 5 show the maximum strengths of the specimens.

Specimen	Cross Section	Structure	Width Thickness Ratio	Stiffening Method
Non Stiffening 200K		SC	200	Non Stiffening
One Insert Plate 100k		SC	100	Insert Plate (1-PL.-3.2)
Two Insert Plates 67K		SC	67	Inser Plate (2-PL.-3.2)
Angle 67A		SC	67	Angle (4-L50X 50X4)
Stud 35S		SC	35	Headed Stud (ϕ6, ℓ=40, @80)
Steel 100S		Steel	100	Insert Plate (1-PL.-3.2)
Concrete 0C(Com.) 1C(Shear)	0C / 1C	Concrete	—	—

※Concrete : Fc240 Surface Plate : PL.-3.2
Side Plate : PL.-22(Com.), PL-19(Shear)

Table 1 List of Specimens

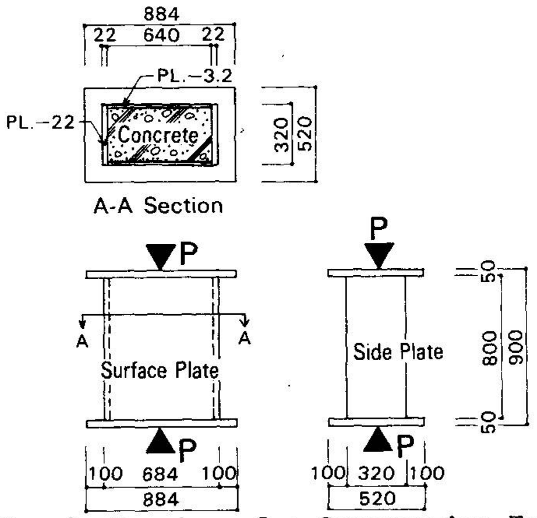


Fig. 3 Specimen for Compression Test

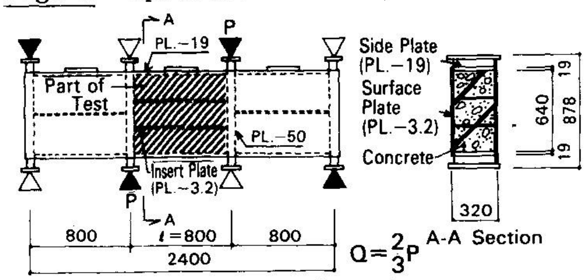


Fig. 4 Specimen for Shear Test

Thickness (mm)	Yield Stress σ_y(MPa)	Tensile Strength σ_u(MPa)	Elongation ε(%)
3.2	253	402	30.5
19	352	514	27.9
22	343	515	27.5

Table 2 Mechanical Properties of Steel

Test	Compressive Strength (MPa)	Tensile Strength (MPa)	Elastic Modulus (kN/mm²)	Age (days)
Compression	23.3	2.7	21.5	47
Shear	23.4	2.4	23.0	72

Table 3 Mechanical Properties of Concrete

3.2.1 Composite Effects of Steel Plate and Concrete (see Figs. 5(a) and 6(a))

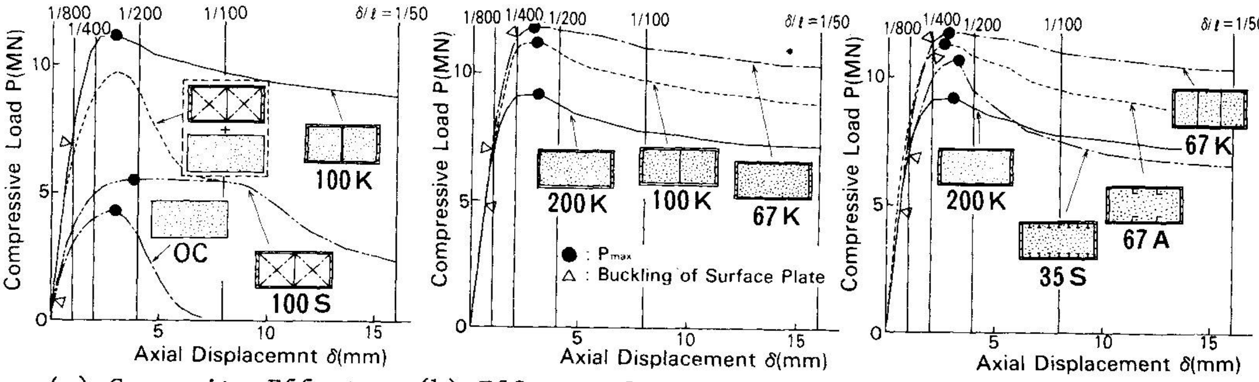
When compared with the superposed strength of the steel (100S) and the concrete (0C, 1C) specimens, the maximum strength of the SC specimen (100K) was stronger by 1.14 times in the compression test, and by 1.40 times in the shear test. The ductility was excellent in both tests.

3.2.2 Effects of Insert Plate (see Figs. 5(b) and 6(b), Table 4)

When three specimens with varying insert plate intervals (200K, 100K, 67K) were compared, their elastic rigidities were almost the same. In the compression tests, the maximum strengths were different due to the number of insert plates which support divided parts of compressive force, and due to the stiffening effects to the surface plates which increase the strengths against bucklings. In the shear tests, the maximum strengths were almost the same, but as the insert plate interval decreased, the rigidity beyond the elastic range decreased, and the displacement at the maximum strength increased. It was thought that each of the concrete blocks divided by the insert plate were deformed separately. In both tests, the early stage buckling of the surface plates had little affect on the relationship between the loads and the displacements.

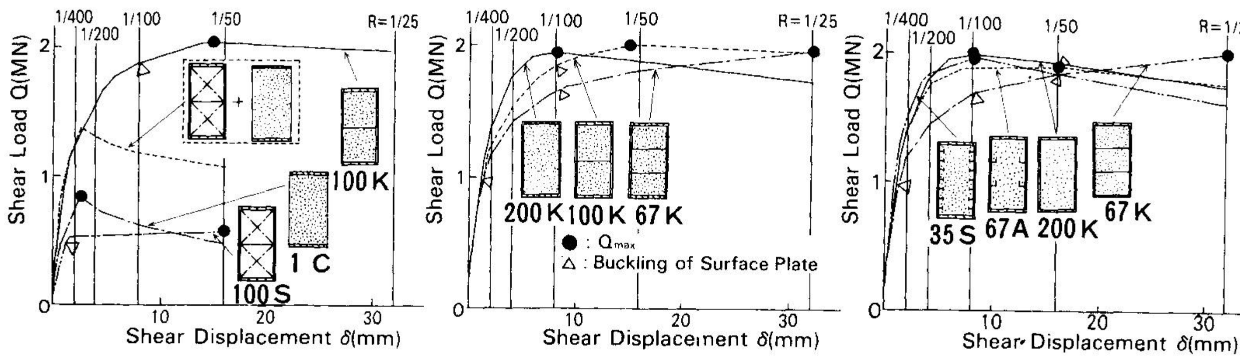
3.2.3 Effect of Stiffening Method (see Figs. 5(c) and 6(c))

In compression tests, the maximum strengths were different from aforementioned. In the relationship between loads and displacements of the shear tests, the non stiffening type (200K), the L-steel angle type (67A) and the stud type (35S) were about the same, but the two insert plates type (67K) showed a different behavior. In the case of stiffening by headed studs, the load carrying ability beyond the displacement at maximum strength was reduced earlier than others due to the failure of the concrete in between the stud head and the surface plate.



(a) Composite Effects (b) Effects of Insert Plate (c) Stiffening Methods

Fig. 5 Load-Displacement Relationship for Each Parameter in Compression Tests



(a) Composite Effects (b) Effects of Insert Plate (c) Stiffening Methods

Fig. 6 Load-Displacement Relationship for Each Parameter in Shear Tests

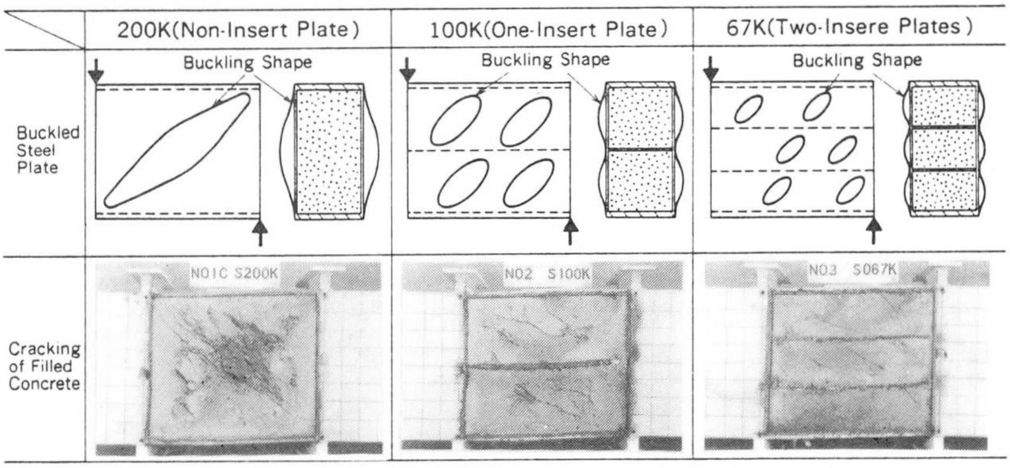


Table 4 Final State of the Steel Plate and the Filled Concrete

Specimen	Compression			Shear					
	Test	Calculated		Test	Calculated				
	Pe(MN)	Pc(MN)	Pe/Pc	Qe(MN)	Q_1(MN)	Qe/Q_1	Q_2(MN)	Qe/Q_2
200K	9.15	9.56	0.96	1.97	1.74	1.13	1.07	1.84
100K	11.30	10.20	1.11	2.04	1.74	1.17	1.07	1.91
67K	11.80	10.90	1.08	2.00	1.74	1.15	1.07	1.87
67A	11.40	10.80	1.06	1.89	1.74	1.09	1.07	1.77
35S	10.70	10.50	1.02	1.95	1.74	1.12	1.07	1.82
100S	5.44	5.00	1.09	0.58				
0C,1C	4.31	4.78	0.90	0.85				

$Pc = As \cdot \sigma_{cr} + (Ap + Ar)\sigma_Y + Ac \cdot Fc$, $\sigma_{cr} = k\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2$, k=6.98(SC Specimen), k=4.00(100S)

Q_1 : Kato and Suzuki's Analysis $Q_2 = As \cdot \tau_s + Ac \cdot \tau_c$, $\tau_s = \sigma_Y/\sqrt{3}$, $\tau_c = 0.1Fc$

As,Ap,Ar,Ac : Area of Surface Plate, Side Plate, Stiffening Member, Concrete

Table 5 Maximum Strength

3.2.4 Maximum Strength (see Table 5)

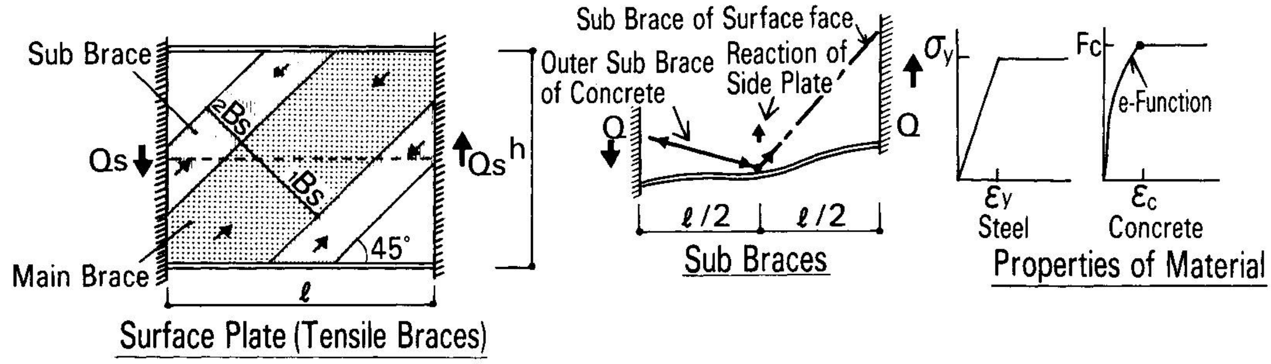
The calculated maximum strength (Pc, Q_2) of the SC bearing walls, obtained from superposed values of calculated steel strength and calculated concrete strength showed similar values with the rsults of the compression tests. The ratio of (Pe/Pc) were in the range of 0.96 ~ 1.11. But in shear tests, the ratio of (Qe/Q_2) showed safety margins of 1.77 ~ 1.91. The maximum shear strength Q_1 by Kato and Suzuki's theory showed that the (Qe/Q_1) ratios were 1.09 ~ 1.17 indicating good agreement with the shear test values.

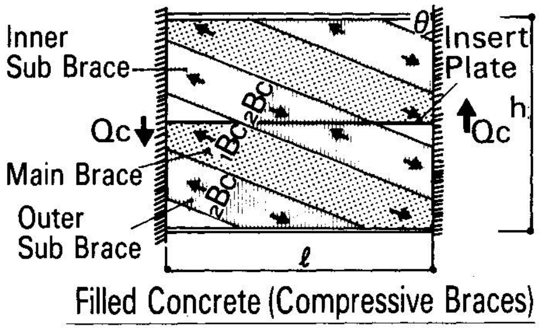
3.3 Analysis of Load-Displacement Relationship under Shear Load

Shear test results for differing insert plate intervals showed differences in the rigidity beyond the elastic range and displacement at the maximum strength. Therefore the analysis of relationship between load and displacement described below was carried out.

The analytical model was replaced by 45° direction tensile braces for surface plates, and diagonal direction compressive braces for separated concrete (in rectangle block formed by side plate and insert plates, see Fig. 7). The angle of each brace was determined by observation of the steel buckling pattern and the concrete cracking pattern after the tests (see Table 4).

The results of analysis agreed approximately with the test results (see Fig. 8). From the analyzed results, it can be seen that the maximum strength is dependent on the compressive strength of the main concrete braces, and the more the angle of the concrete braces increase due to the insert plate intervals, the more shear displacement at compressive strength increased. And also, the analytical elastic limit points were determined by the tensile yield of the surface plate. This phenomenon is the same as in the tests.





Shear Strength : $Q_{sc}=Q_s+Q_c$

$Q_s=t_s\{{}_1B_s\cdot\sigma_s(\varepsilon_s)+{}_2B_s\cdot\sigma_s(\varepsilon'_s)\}\cos 45°$

$Q_c=t_c\{(n+1)\cdot{}_1B_c\cdot\sigma_c(\varepsilon_c)+n\cdot{}_2B_c\cdot\sigma_c(\varepsilon_c)+{}_2B_c\cdot\sigma_c(\varepsilon'_c)\}\cos\theta$

Effective Width : ${}_1B_s=(h-\ell/4)\cos 45°$, ${}_2B_s=\frac{\ell}{4}\cos 45°$

${}_1B_c={}_2B_c=\frac{h\sin\theta}{2(n+1)}$

$\theta=\tan^{-1}\{(n+1)\ell/h\}$, n : Number of Insert Plate

t_s, t_c : Thickness of Steel Plate and Concrete

σ_s, σ_c : Stress of Steel Plate and Concrete

$\varepsilon_s, \varepsilon_c$: Strain of Main Brace of Steel Plates, Main Brace and Inner Sub Broce of Concrete

$\varepsilon'_s, \varepsilon'_c$: Strain of Outer Sub Brace of Steel Plates and Concrete

Fig. 7 Analytical Model of Load-Displacement Relationship under Shear Load

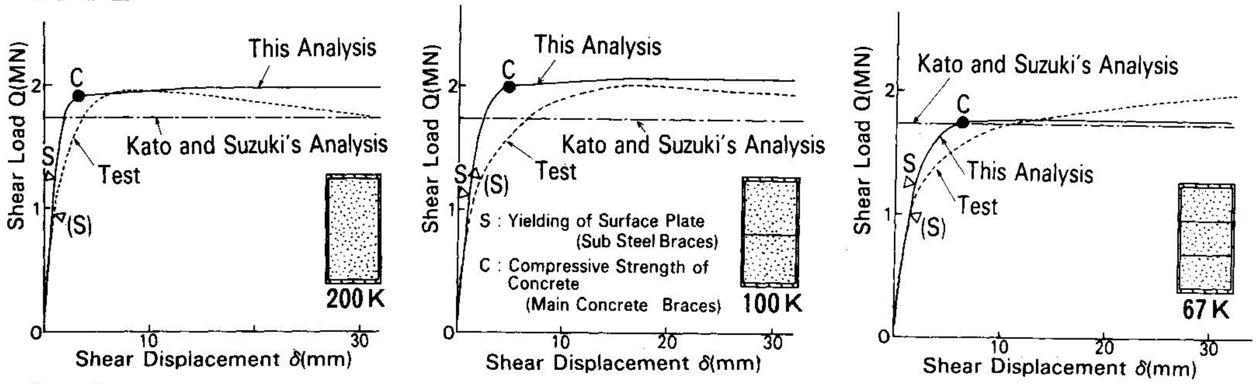


Fig. 8 Calculated Results of Load-Displacement Relationship under Shear Load

4. CONCLUSIONS

The thin steel plate SC bearing walls showed high strength and sufficient ductility due to the composite effect of the steel plate and the concrete. The buckling of the thin steel plate had little affect on the relationship between loads and displacements. The superposed values of the calculated steel strength and concrete strength is considered as a possible method to evaluate the strength of those structure. The calculated relationship between loads and displacements using a replaced brace model was in agreement with the test results.

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Poutre composite préfléchie précontrainte pour ponts et bâtiments

Composite Pre-Bent Prestressed Beam for Bridges and Buildings

Vorverformte, vorgespannte Verbundträger für den Brücken- und Hochbau

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RÉSUMÉ
Il s'agit d'une poutre mixte capable de supporter de très fortes charges pour de très grandes portées avec une hauteur de construction minimum et pour un coût acceptable. Le principe en est l'utilisation d'une poutrelle en acier préfléchie et précontrainte qui est ensuite enrobée de béton lui-même précontraint par adhérence et éventuellement par câbles. La performance s'explique par l'exploitation optimale en tout point de chaque section des caractéristiques des composants – tous à haute résistance – et n'est possible que par la maîtrise du calcul, et facile que grâce à l'informatique.

SUMMARY
This composite beam can carry heavy loads over large spans at a minimum building height and at a very reasonable cost. The composite beam is based on the following principle : it is composed of a pre-bent prestressed beam coated with concrete that is itself either prestressed through bonding or post-tensioned, whilst in all section points properties of the high strength components are put to an optimal use. Mastering of the calculation procedure and the use of a computer are indispensable elements for the realisation of this performance.

ZUSAMMENFASSUNG
Es handelt sich um Verbundträger für die Aufnahme sehr grosser Lasten bei grossen Spannweiten und minimaler Bauhöhe. Der Träger setzt sich aus einem vorverformten und vorgespannten Stahlträger, welcher mit Beton umhüllt wird, zusammen. Der Beton wird dabei durch Haftspannungen oder auch nachträglich durch Kabel vorgespannt. Die hohe Tragfähigkeit ergibt sich durch die optimale Ausnutzung der hochfesten Baustoffe in jedem Querschnitt des Trägers. Dies ist nur möglich dank der Beherrschung der Festigkeitsberechnung. Rechnerunterstützte Berechnungsmethoden sind dabei unentbehrlich.

1. CONCEPTION

Les idées de base du nouveau procédé sont les suivantes :
- Utiliser une poutrelle métallique précambrée et précontrainte posée sur 2 appuis d'extrémité.
- Ramener la fibre inférieure en situation horizontale par préflexion, enrober la poutrelle de béton armé, la semelle inférieure étant pourvue de torons de précontrainte par adhérence.
- Relâcher la préflexion mettant la semelle inférieure en compression, à laquelle s'ajoute encore la précontrainte par adhérence.
- Pour garder des contraintes admissibles dans la semelle supérieure, prévoir une précontrainte provisoire agissant sur cette semelle.

2. DESCRIPTION DU PROCEDE

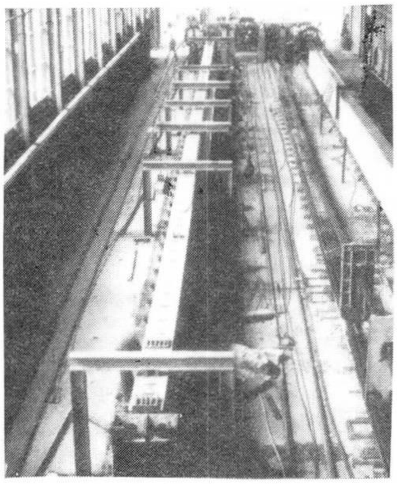

Fig.1 Poutrelle installée

2.1. Une poutrelle métallique est précambrée lors de sa fabrication par laminage ou par assemblage de plats soudés. Elle est sollicitée par préflexion et compression longitudinale excentrée de manière à éliminer les contraintes résiduelles de construction et à leur rendre un comportement élastique. La poutrelle est alors mise en place sur le fond de coffrage d'un banc de précontrainte à l'usine de préfabrication.
Des portiques métalliques la maintiennent dans la position voulue et empêchent tout déversement de la semelle supérieure. (fig.1)

2.2. Une précontrainte excentrée est appliquée à la semelle inférieure de la poutrelle sur tout ou partie de la longueur et vient augmenter la précambrure initiale en même temps que précomprimer la semelle inférieure.

2.3. Deux efforts verticaux sont appliqués à la poutrelle métallique et la mettent en préflexion, la ramenant dans une position horizontale.

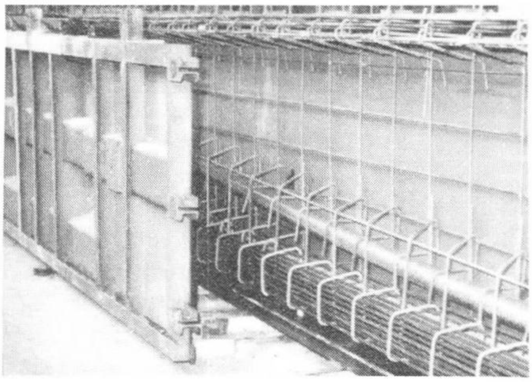

Fig.2 Poutre prête au bétonnage

2.4. Des armatures de précontrainte par adhérence du béton sont disposées autour de la semelle inférieure de la poutrelle et tendues entre les ancrages du banc. Des câbles de précontrainte à tendre sur chantier sont mis en place de même que les armatures complémentaires passives ou actives (armatures technologiques, étriers, armatures de renfort,...) (fig.2). Les coffrages latéraux sont fixés sur le fond et remplis de béton serré par vibration agissant sur les coffrages eux-mêmes. Le bétonnage est arrêté au niveau de la semelle supérieure de la poutrelle.

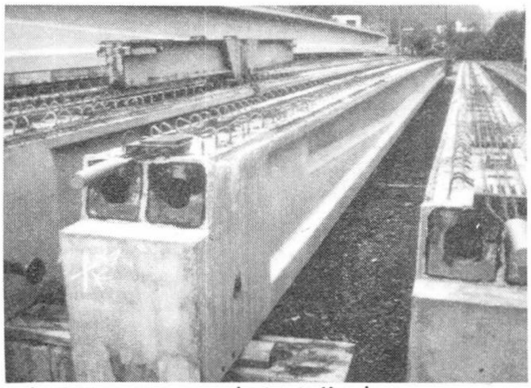

Fig.3 Poutre terminée à l'usine

2.5. Des armatures de précontrainte sont mises en place et tendues sur la semelle supérieure de la poutrelle restée apparente ("antiprécontrainte" à caractère provisoire).

2.6. Les efforts de préflexion sont relâchés mettant la poutre mixte en précontrainte et mobilisant simultanément son poids propre.

2.7. Les torons de précontrainte par adhérence sont relâchés apportant un complément de précontrainte à la poutre mixte. La poutre est mise au stock. (fig.3).

2.8. La poutre est transportée, mise en place sur chantier et reçoit le coffrage de la dalle, c'est-à-dire généralement des prédalles en béton.

2.9. Les efforts de précontrainte sur la semelle supérieure sont relâchés.

2.1O. La dalle est coulée.

2.11. Les câbles de précontrainte chantier sont mis en tension et injectés.

2.12. La poutre est apte à reprendre les charges permanentes et les charges mobiles.

3. EVOLUTION DES CONTRAINTES (σ)

Nous allons suivre cette étude dans le cas précis du viaduc de Lixhe qui est un bon exemple d'application du procédé dans son ensemble.

3.1. Caractéristiques du viaduc (fig.4) :

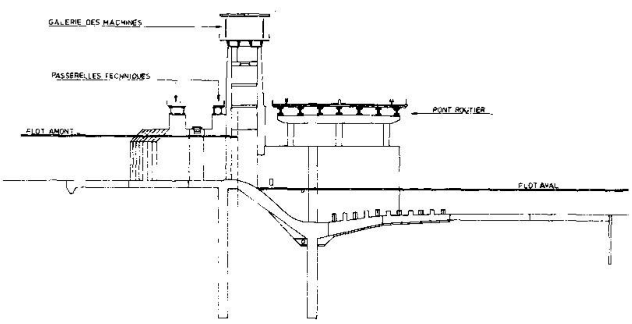

Fig.4 Coupe transversale de l'ouvrage.

Cet ouvrage permet le franchissement de la Meuse à 16 km en aval de Liège à hauteur d'un barrage, ce qui a conduit l'Administration à choisir une solution de tablier à poutres préfabriquées présentant une hauteur totale de 1m55. C'est un pont routier à 4 bandes de circulation qui doit permettre le passage de convois lourds de 240 tonnes.

Longueur totale : 324 m dont 4 travées de 47 m pour lesquelles les poutres faisant l'objet du présent exposé ont permis une solution économique avec une hauteur de 1m39 pour les poutres, soit 1/34 de la portée.

3.2. Caractéristiques de la poutre et de ses matériaux : (fig.5.)

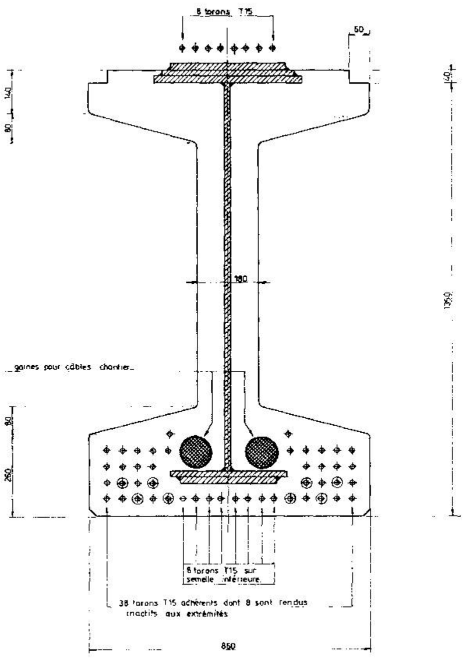

Fig.5 Coupe transversale de la poutre à mi-portée.

Longueur 46m90 - Portée 46m10 - Poids total 90 T

Poutrelle : Acier AE 355D - Ea = 215.000 N/mm^2
Tension max. service : 0,8 x 355 (= 288 N/mm^2)

T15.140.S : Charge de rupture 140 x 1,86 = 260,4 KN
Effort initial sur béton : 0,7 x 260,4 = 182,3 KN

Câbles 4T15.150 : Charge rupture 4x150x1,77=1062 KN
Effort initial sur béton : 0,7 x 1062 = 744 KN

Aciers BE 50 : Limite élastique 500 N/mm^2
Tension max. service : 0,6 x 500 (= 300 N/mm^2)

Béton poutre à la mise en précontrainte
Résistance moyenne R'b = 45 N/mm^2
Eb = 37000 N/mm^2 m_1 = Ea/Eb = 5,75

Béton poutre à 28 jours :
Résistance garantie R'wk = 54 N/mm^2
Eb = 42000 N/mm^2
m_2 = Ea/Eb = 5 (valeur instantanée)
m'_2 = 8,4 (valeur à la longue)

Béton dalle à 28 jours :
Résistance garantie R'wkd = 45 N/mm^2
Ebd = 37000 N/mm^2
m_3 = 5,75 (valeur instantanée)
m'_3 = 9,8 (valeur à la longue)

3.3. Calcul aux différentes phases d'exécution

3.3.1. Calcul élastique :

Les caractéristiques composées dans la section à mi-portée sont données au tableau 1.
Les distances des centres de gravité (G) sont données par rapport à la fibre inférieure .

Stade du procédé	Dénomination	Section m^2	Distance G m	Inertie m^4
[2.1]	Poutrelle métallique	0,056440	0,874	0,016313107
[2.2][2.3]	Poutrelle métallique précont.	0,057560	0,858	0,017058762
[2.5] à [2.7]	Poutre terminée au coupage(m_1)	0,152011	0,703	0,043032912
[2.6]à[2.10]	Poutre terminée à la longue(m'_2)	0,124576	0,721	0,036209845
[2.11]	Poutre+dalle à la longue(m'_2,m'_3)	0,180074	0,947	0,057152327
[2.12]	Idem+câbles+injection(m'_2,m'_3)	0,185718	0,925	0,060209387

Sollicitations propres à la poutre :
Préflexion : 250 KN, entre-axe 18,60 M.
Poids poutrelle métallique : 163,563 KN
Poids béton+torons+armatures : 717 KN.

Poids de dalle et surcharges :
Poids dalle + câbles + armat.: 12,68 KN/M.
Charges permanentes : 8,95 KN/M.
Charges mobiles : 26,97 KN/M.

Tableau 2 des contraintes en N/mm^2 dans la section à mi-portée : + = tract. - = compres.

Stade	Dénomination	Moment KN x M	Fib.inf acier	Fib.s. acier	Fib.sup béton	Fib.inf béton	Fib.sup dalle
[2.1]	Poids Poutrelle acier	+ 1057,26	+ 50,1	- 35,1			
[2.2]	Précontrainte poutrelle	- 1201,02	- 82,8	+ 14,0			
[2.3]	Préflexion poutrelle	+ 4650,00	+ 220,5	-154,3			
	Poutrelle avant bétonnage.		**+ 187,8**	**-175,4**			
[2.5]	"Antiprécontrainte" (ml)	+ 1730,79	+ 14,5	- 38,4	+ 3,23	- 6,50	
[2.6]	Lâchage préflexion (ml)	- 4650,00	- 65,2	+ 76,9	-13,02	+13,00	
	Poids propre béton (ml)	+ 3551,00	+ 49,8	- 58,7	+10,10	- 9,93	
[2.7]	Précontrainte adhér.(ml)	- 4773,53	-100,4	+ 19,1	-19,04	+ 2,90	
	Poutre terminée au coupage		**+ 86,5**	**-176,5**	**-18,93**	**- 0,53**	
[2.8]	Coffrage dalle (25% dalle)	+ 842	+ 14,4	- 16,2	+ 2	- 1,85	
[2.9]	Enlèvement antiprécontr.	-1730,79	- 14,5	+ 38,4	- 3,23	+ 6,50	
	Après coffrage dalle,pas pertes		**+ 86,4**	**+154,3**	**-20,16**	**+ 4,12**	
Valeurs à la longue							
[2.6]	Lâchage préflexion (m'2)	-4650	- 79,7	+ 89,2	-11	+10,2	
	Poids propre béton (m'2)	+3551	+ 60,9	- 68,1	+ 8,4	- 7,8	
[2.7]	Précontrainte (m'2)	-4773,53	-124,6	+ 21,6	-16,16	+ 2,24	
[2.10]	Coulée de la dalle (m'2)	+3367,49	+ 57,7	- 64,6	+ 7,98	- 7,41	
[2.11]	Précontr. chantier (m'3)	-4170	- 92,8	+ 3,1	-11,92	+ 0,16	+ 1,62
	Pertes précontrainte 10%	+1076	+ 22,3	- 1,2	+ 2,86	- 0,09	- 0,45
	Situation tablier terminé		**+ 31,6**	**-195,4**	**- 19,84**	**- 2,70**	**+ 1,17**
[2.12]	Surcharges fixes (m'2,m'3)	+ 2538,65	+ 34,8	- 20,7	+ 4,64	- 2,34	- 2,86
	Surch. mobiles (m'2,m'3)	+ 7163,96	+ 98,1	- 58,3	+13,10	- 6,59	- 8,08
	Poutre sous total charges		**+164,5**	**-274,4**	**- 2,10**	**- 11,63**	**- 9,77**

Quelles constatations peut-on faire à ce moment de l'étude ?

- La poutre est maintenue au cours des différentes opérations dans une situation de contraintes tout à fait confortable tant pour le béton que pour l'acier.
- Grâce à " l'antiprécontrainte " notamment, la poutre se trouve en compression à la fibre supérieure, ce qui permettra de lui donner certains porte-à-faux pour faciliter les manutentions de mise en place.
- Ce calcul réalisé dans toutes les sections de la poutre permet de connaître parfaitement la situation et donc d'exploiter au mieux les matériaux de la poutre composite.

3.3.2. Déformations

Le calcul des flèches aux différents stades du procédé et dans des sections entre-distantes de 1 m à partir de la section à mi-portée a été effectué. Nous résumons au tableau 4 les différentes valeurs dans la section à mi-portée.

Stade du procédé	Dénomination	Flèche : + = flèche - = contreflèche
[2.1][2.3]	Préflexion + poids propre poutrelle	+ 388 mm.
[2.2]	Précontrainte poutrelle	- 96 mm.
	Précontrainte poutrelle avant enrobage	+ 292 mm. fibre inf. acier
[2.6]	Relâchement préflexion	- 139 mm.
	Poids propre béton	+ 119 mm.
[2.7]	Précontrainte par adhérence	- 117 mm.
	Poutre complète au coupage	- 137 mm. fibre inf. béton
[2.6]	Relâchement préflexion	- 167 mm.
	Poids propre béton	+ 142 mm.
[2.7]	Précontrainte par adhérence	- 144 mm.
	Pertes de précontrainte	+ 20 mm.
	Poutre complète à la longue après pertes	**- 149 mm.**
[2.10]	Poids dalle à la longue	+ 104 mm.
[2.11]	Précontr. par câbles à la longue après pertes	- 66 mm.
	Tablier avant surcharges	**- 111 mm.**
[2.12]	Charges permanentes	+ 45 mm.
	Tablier sous charges permanentes	**- 66 mm.**
[2.12]	Charges mobiles (m_2 m_3)	+ 92 mm.
	Tablier sous charges mobiles	**+ 26 mm.**

Tab.3. Déformations.

3.3.3. Elastification de la poutrelle

Avant la mise en oeuvre de la poutrelle, il y a lieu de s'assurer de son comportement élastique et donc d'éliminer les contraintes résiduelles de fabrication. Pour ce faire, on soumet la poutrelle à des sollicitations qui créent en tous points des contraintes égales ou supérieures aux contraintes de service.
Pour ce faire, 2 poutrelles sont couchées sur des appuis à rouleaux, leur semelle inférieure mise au centre. Elles sont soumises :
- à flexion par tiges Dywidag agissant horizontalement pour les rapprocher.
- à un complément de compression introduit dans la semelle supérieure par une précontrainte à l'aide des torons " d'antiprécontrainte ". Ce complément est indispensable en raison de la dissymétrie de la poutrelle même et de l'importance des contraintes nécessaires en semelle supérieure.

Plusieurs mises en charge successives peuvent être faites jusqu'à obtention de flèches égales. En pratique, une seule opération a été nécessaire dans le cas présent.

3.4. Calcul à l'état limite ultime

Ce calcul a été fait suivant les méthodes du CEB en adoptant les coefficients de sécurité repris au tableau 4.

Dénomination	Béton poutre	Béton dalle	Barres BE50	Poutrelle	Précon-trainte	Charges permanentes	Charges mobiles
Coefficient	1,4	1,5	1,15	1,15	1,15	1,35	1,5

Tab.4. Coefficients de sécurité

On obtient : Moment Résitant/Moment sollicitant = 26482 KNm/24941 KNm = 1,062

3.5. Constations et mesures effectuées

3.5.1. Contrôles de contraintes et déformations : La fabrication des 28 poutres décrites ci-dessus a fait l'objet d'un autocontrôle de l'usine suivi journellement par la 2ème Division du Bureau des Ponts à Liège qui a par ailleurs établi un programme de mesures permettant de connaître le mieux possible les déformations et contraintes ( ) à mi-portée et de les comparer aux valeurs calculées. Pour ce faire, 9 jauges de contraintes et 2 extensiomètres à corde vibrante ont été placés en différents points de la section médiane d'une poutrelle. Les différentes mesures sont reprises au Tableau 5.

Stade procédé	Dénomination	σ calcul.	σ mesurées (N/mm^2)		Fibre
[2.1]	Elastification	- 244	- 233		Sup.
		+ 320	+ 278		Inf.
[2.1] à [2,3]	Poutrelle avant	- 175,4	- 172	- 168 (après 7 J)	Sup.
	bétonnage	+ 187,8	+ 171	+ 170 (après 7 J)	Inf.
[2.5] à [2,7]	Poutre au coupage	- 176,5	- 157	- 153 (à 21 J) -151 (à 132j)	Sup.
	et évolution	+ 86,6	+ 28	+ 66 (à 21 J) + 30 (à 132j)	Inf.
[2.8]	Dalle coffrée	- 154,2	- 149		Sup.
[2.9]	sans antipréc.	+ 86,5	+ 10	(sans pertes)	Inf.
[2.10]	Dalle bétonnée et	- 195,4	- 205		Sup.
[2.11]	précontr.chantier	+ 31,6	+ 43	(avec pertes)	Inf.

Tab.5 Comparaison des contraintes poutrelles calculées et mesurées.

La corrélation entre valeurs théoriques et valeurs mesurées est satisfaisante. Toutefois, on peut se demander quelles valeurs de m conduiraient à une concordance encore meilleure. On trouve : au coupage des torons adhérents, m = 8,5 ; à 21 jours , m = 13,5 ; à 132 jours, m = 18. Néanmoins, l'utilisation de semblables valeurs pour le calcul correspondant des contraintes est tout à fait aberrante et nous en avons, bien involontairement eu la preuve. Lors du transport par eau des poutres vers leur destination, un chargement a accidentellement basculé et des poutres se sont retrouvées au fond de la Meuse dans une position quelconque. Elles en ont été retirées de telle sorte que les contraintes en cours de manutention auraient atteint plus de 25 N/mm^2 dans certaines sections qui n'ont présenté aucune dégradation importante. Il y a donc bien eu une redistribution des contraintes entre béton et acier, l'acier ayant empêché le béton de prendre des allongements correspondant aux contraintes théoriques. Sans doute, faudrait-il trouver un moyen dans les calculs de plafonner la résistance du béton à la traction tout comme cela se fait à la compression dans les calculs aux états limites.

3.5.2. Mise en charge d'une poutre avant sortie d'usine : Un essai de mise en charge a été réalisé 21 jours après relâchement de la précontrainte par adhérence par application de 2 efforts de 250 KN à 4m45 de l'axe de la poutre (M = 4650 KNm). Les mesures de contraintes dans la poutrelle et les flèches élastiques mesurées correspondaient à des valeurs calculées pour $5 < m < 6$.

4. CONCLUSION ET COMPARAISON ECONOMIQUE : Les mesures effectuées confirment la fiabilité des méthodes et coefficients de calcul utilisés ainsi que des méthodes de fabrication. Economiquement, la comparaison entre la poutre nouvelle et ses consoeurs, poutre précontrainte classique et poutre préfléchie classique se résume comme indiqué ci-dessous à égalité de portée, charge et hauteur de construction.

	Poutre nouvelle	Poutre préfléchie	Poutre précontrainte
Nombre	7	7	15
Poids poutrelle	114,5 T	255,7 T	-
BE50	20,1 T	22,6 T	59,6 T
T15, 140S	18,1 T	-	30,3 T
T15, 150	11,4 T	-	28,8 T
m^3 béton	201,3 m^3	282,7 m^3	590,2 m^3
Rapport prix placé	1	1,60	1,45

Axial Shear Connectors for Wide Span Prefabricated Structures

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Axiale Schubdübel für vorfabrizierte Balken mit grosser Spannweite

Carlo MARIONI
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Carlo Marioni, born in 1939, graduated in civil engineering from Milan Polytechnicum in 1964. From 1974 through 1977 he lectured at Rome University. He has been responsible for a number of prefabricated and/or prestressed structures projects ranging from multi-storey buildings to highway bridges and nuclear power plants, partly based on his own patents.

SUMMARY
The purpose of achieving spans for bridges of up to 90 meters by means of prestressed simply supported structures adopting light composite truss beams supporting cast in situ floor decks is getting nearer and of real economical interest. Coupling of steel diagonals and struts to prefabricated prestressed concrete chords is facilitated by flexible-type shear connectors. The paper describes tests carried out during ten years and first realizations by the author.

RÉSUMÉ
Le franchissement de portées de 90 m, par des poutres librement appuyées, précontraintes, est devenu intéressant par l'utilisation de poutres-treillis légères en construction mixte acier-béton précontraint, soutenant des prédalles complétées in situ. Le couplage entre les barres métalliques du treillis et les éléments préfabriqués est obtenu au moyen de goujons flexibles. L'étude donne une description des expériences conduites pendant dix ans par l'auteur, et les premières réalisations.

ZUSAMMENFASSUNG
Die Möglichkeit Brückenspannweiten bis 90 m mit Hilfe frei aufgelagerter, vorgespannter Balken zu erreichen kommt näher und wird wirtschaftlich interessant, wenn vorgespannte Fachwerk-Verbundträger verwendet werden, welche mit einem an Ort gegossenen Ueberbeton zu ergänzen sind. Die Verbindung der Fachwerkstäbe aus Stahl mit den vorfabrizierten Betonelementen wird mit Hilfe flexibler Schubdübel erreicht. Der Beitrag beschreibt Versuchsergebnisse und erste Anwendungen.

1. FOREWORD

A careful investigation on shear connectors particularly fit to prefabricated structures and aimed at reaching wide spans, using composite truss beams completed by cast-in-situ floor decks, was conducted by the author during ten years. First realizations in Italy, ranging from highway bridges to floor decks for nuclear plants, display an ingrowing interest for this type of structural solution.

2. EXPERIMENTAL INVESTIGATION

2.1. Statical Tests (Asymmetric Specimens)

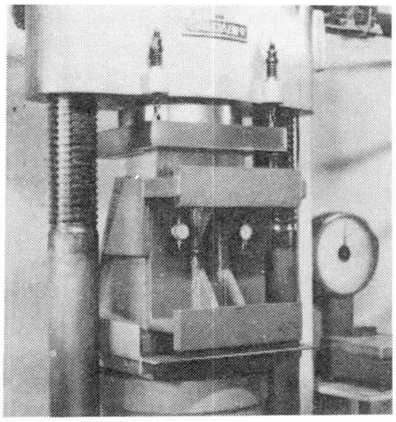

Fig.1 Tests on asymmetrical shear connector specimens

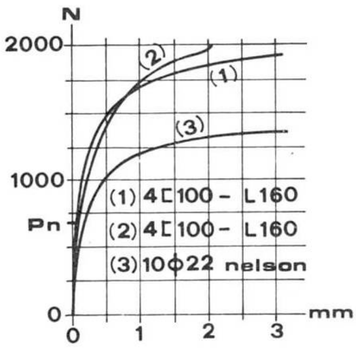

Fig.2 Relative displacement vs. load for different types of shear connectors

Early tests at the Laboratory of the Istituto di Scienza delle Costruzioni at Turin Polytecnicum during 1978 on four shear connectors specimens (2 specimens with channel and 2 specimens with Nelson headed stud connectors) persuaded the author to choose the light channel type connector, so much that he stopped testing the forth specimen equipped with Nelson-type connectors (curve three of Fig.2). Intensive and more carefully conducted tests by other authors (1),(2),(3), explain in detail the soft behaviour of the Nelson type,(4) compared with the rigid behaviour of the channel-type connector (curves 1 and 2 of Fig.2).

A second serial of tests was conducted in 1979 at the same Laboratory of Turin Polytechnicum upon 2 real scale specimens of truss beams spanning 6.36 meters (5). Location of the 149 measurement bases on two beams are shown in Fig.3 and the quite low relative displacements between steel plates and concrete are illustrated in Fig. 4, giving good accordance with early results.

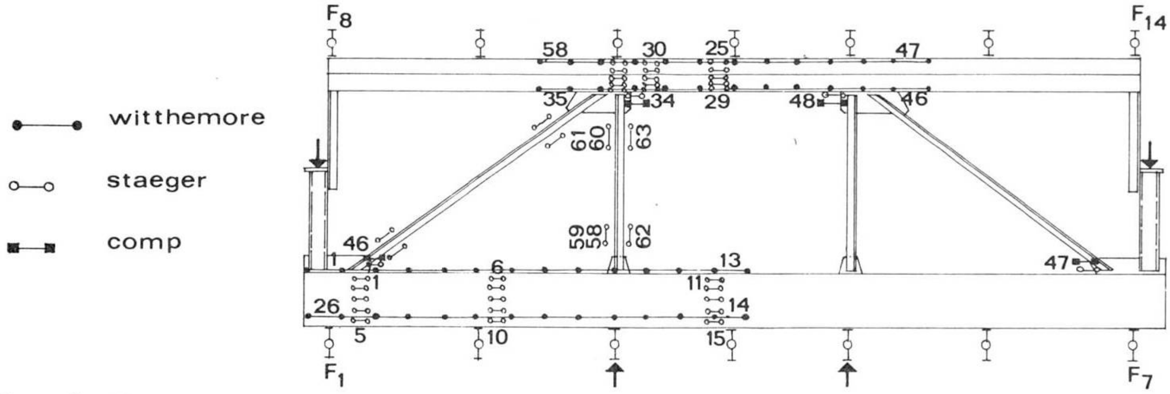

Fig. 3 Measurement location for the 2 beams tested at Turin Polytechnicum

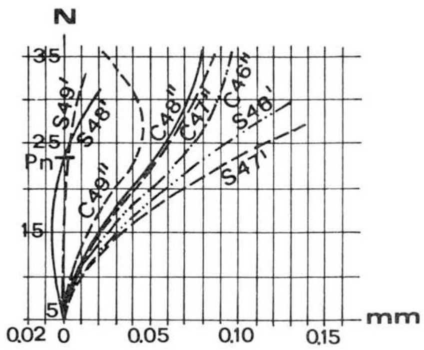

Fig. 4 Relative displ./load for Fig.3 beams

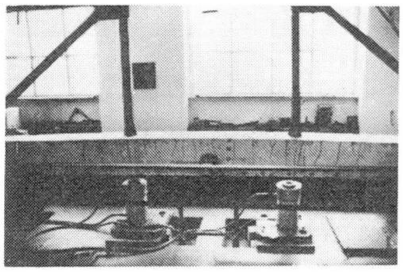

Fig.5 The limit state for one of the tested beams of Fig.3

Both beams were tested by means of two hydraulic jacks of 1000 kN and behaved almost elastically up to working conditions; the second beam (Fig.5) showed a permanent deflection of about 0,20 mt. after a loading range totalling 2.3 times the working load; the whole beam revealed full exploitation of all structural elements (p.c. and r.c. chords, steel diagonals and sruts, shear connectors and gusset plates).

2.2. Tests - (Symmetric Specimens)

Push-out tests on 3 double specimens (A Serie) equipped with (80x45x6)mm. channels and measuring (160x120x200)mm. (dimensions are slightly different from the standard AIPC-CEB-CECM-FIP ones) in 1983 at Milan Polytechnicum, revealed the intrinsec sensitivity of the proof to arch effects related to lateral thrust at the base of specimens. The third test with reduced distance for the two lower hinges, showed however lower rotations of twin specimens up to ultimate load, and an almost tangential working for the shear connectors in the elastic range: this effect caracterizes axial shear connection for truss beams.

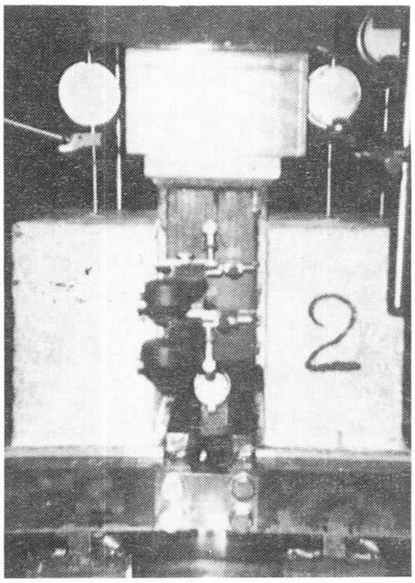

Fig. 6 Statical tests of symmetric specimens

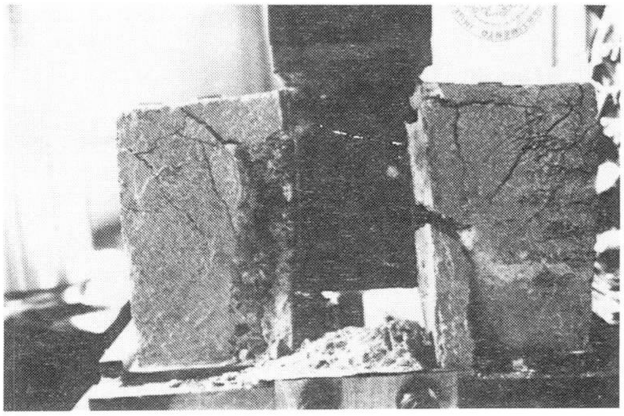

Fig. 7 Failure of specimen B5 under static load

2.3. Dynamic Tests - (Symmetric Specimens)

Dynamic pulsating tests on 7 double specimens (B Serie) same as the static tested ones, were carried out also in 1983 at Milan Polytechnicum. The pulsating force was held between 40 and 150 kN, by a frequence of 9.9 Hz. Each sample underwent 2.E6 loading cicles without visible cracking effects. Seven stops during tests made it possible to perform a statical test under a load of about 40 kN. The measurements showed a longitudinal adaptation less than 0,1 mm. of the tested specimens for the first 10000 cicles, slightly increasing for 2.E6 cycles, and an angular rotation of the head of the specimen held constant. Another result of the tests, which will be illustrated more in detail in a next publication, was the tendance of tested specimens to raise internal states of self-stressing related to cracking, under localized shrinkage effects. Three specimens were also statically tested up to failure as shown in Fig. 8 averaging an ultimate load not lower than that reached by the same specimens not prooved by dinamic tests. Failure was given by crashing of concrete, and almost simultaneously reached excessive elongation of connectors web. Fig.7 shows a characteristic failure crack pattern for specimen B2 which permits to "read" the real path of compressive struts through concrete. The steel connector was a St 430 MPa, weld was magnetically inspected, and a preliminar chemical investigation was made for mill steel. Concrete reached a characteristic R'bk = 50MPa cubic strength at 28 days.

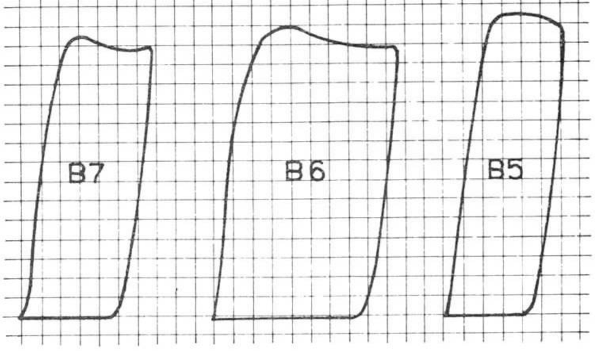

Fig. 8 F-ε diagram for static loading to failure, of 3 dynamic tested specimens.

3. APPROACH WITH F. E. METHOD.

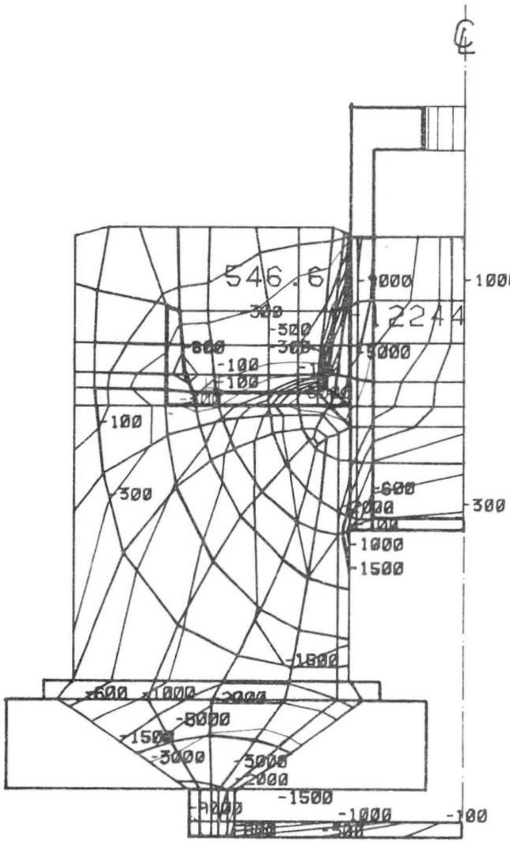

Fig. 9 Stress level plot after elastic analysis with a F.E.M. mesh of 151/2-D elements.

An analysis conducted with 2-D quadrilateral strain F.E. confirmed high stress concentrations at the fore toe of the channel;after inibition of some F.E. at the interface with the steel pipe. Though quite inadequate, the elastic analysis showed that real behaviour of this type of connection is far from being well understood and needs for more intensive and widespread ascertainement in the future. This gap of our knowledge has been recently remembered by other investigators (6).

4. UP TO DAY REALIZATIONS.

The first realizations include 2 bridges over the river Melezzo built during 1982 in the high Vigezzo Valley, about 30 Km. far from Locarno. The Mugnago bridge spans 3x40 and the Meis bridge 30 -45- 30 meters. Both are pedestrian bridges designed for a 270 kN truck. The bridge deck was entirely prefabricated in a plant 300km. away, and assembled within 2,5 months/bridge. Each 1/3 span single beam segment was assembled during a workday by 4 workmen,and raised on provisional piers by a little self-moving crane: maximum load of one segment was 170 kN. Two days after sealing of 60 mm.thick chord joints by means of high strength cement mortar; tensioning of the (2x2) lower chord 1200 kN/each prestressing tendons produced self-raising of both main beams. Another group of realizations still under construction comprises the aseismic floor decks of 4 major buildings of the nuclear plant ENEL (general designer ANSALDO) near Rome. The flat deck of the Turbina building for ex. measures

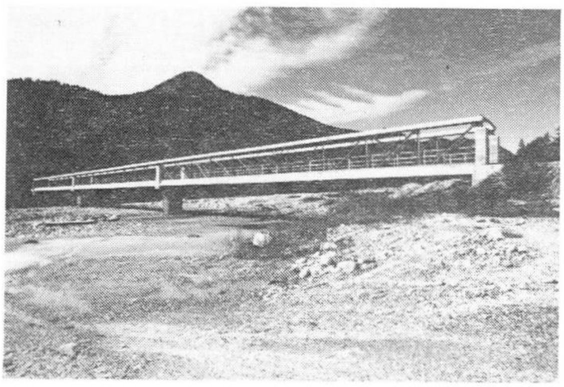

Fig.10 The Mugnago Bridge spanning (3 x 40,0 meters)

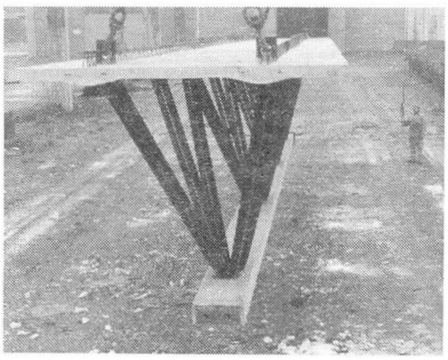

Fig. 11 Assembler beam for nuclear plant near Rome

(115,0x35,0)m. and is a completely isolated plate 0,40 m. thick, supported by 48 adjacent Assembler-type truss beams and post-tensioned by 46 cables; linked to the exterior wall of the 35,0 m. high structure by means of some 38 hydraulic shock-absorbers and 4 neoprene/teflon bearings of 6000 kN (7). The last realization is the Plusc bridge of Verbania on the boarder of the Maggiore Lake in North Italy, spanning 70,0 m., also under construction. The bridge deck is designed for highway loads and is supported by 3 Assembler-type beams weighing 2100 kN spaced at 3,75m.

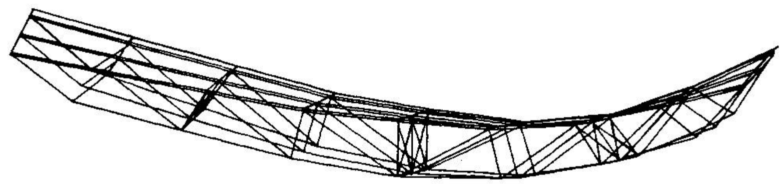

Fig. 12 Elastic deformed structure of the 70,0 m. span of Plusc Bridge of Verbania

5. THE ECONOMICAL SIDE.

Computations of designed works give (Table I) the following values for building materials. Table II gives extrapolated items for a 90 m. span.

Concrete of the plate-deck	0,24 m^3/m^2	0,24 m^3/m^2
Concrete of pref. beams	0,32 m^3/m^2	0,65 m^3/m^2
Structural steel	1,05 kN/m^2	1,33 kN/m^2 *
Prestressing steel	0,17 kN/m^2	0,34 kN/m^2
High-bond steel pref. beams	0,19 kN/m^2	0,40 kN/m^2
Table I Materials for a 70,0m. h.-bridge		Table II Materials for a 90,0 m. h.-bridge
* adopting for diagonals and struts circular seamless pipes.		

Further savings will be obtained by utilization of high strength concrete so to reduce the weight of the prefabricated truss beams.

6. CONCLUDING REMARKS.

This new type of structure permits to improve the range of spans for prefabricated structures, eliminating the heavy webs of traditional prestressed beams. Contemporary use of prestressing steel and structural steel for the beams, and r.c. in the upper zone of the deck, avoids provisional windbracing of steel beams during erection and gives best behaviour under impact and dynamic loading.

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SOUS-THÈME 4.3

Innovation in the Field of Structures

Innovation dans le domaine des structures

Neuerungen auf dem Gebiet von Bauwerken

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Demountable Bridge Spans Made of Prefabricated Box Beams

Ponts démontables réalisés avec des poutres en caisson préfabriquées

Abmontierbare Brücken aus vorgespannten Kastenbalken

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SUMMARY
The results of tests on prefabricated prestressed reinforced concrete box beams are presented. Beams of this type can be used for the construction of demountable spans of road bridges. Some of the results obtained from the tests are compared with results yielded by the finite strips method.

RÉSUMÉ
L'article présente les résultats d'essais de poutres en caisson en béton armé, qui ont été préfabriquées et précontraintes. Des poutres de ce genre peuvent être utilisées dans la construction de ponts-routes démontables. Quelques résultats d'essais sont comparés avec les résultats dérivés de la méthode des bandes finies.

ZUSAMMENFASSUNG
Es werden Resultate von Versuchen mit vorgefertigten, vorgespannten Kastenbalken aus Stahlbeton vorgestellt. Balken dieser Art können in der Herstellung von abmontierbaren Strassenbrücken verwendet werden. Einige Versuchsresultate werden mit Resultaten der begrenzten Streifen-Methode verglichen.

1. INTRODUCTION

Demountable bridges are commonly used as metal constructions in by-pass objects, on forest tracks at felling areas, as well as for military purposes. They are usually made of rolled metal elements joined with screws, high tensile bearing-type bolts or pins. Concrete as a material for the building of demountable bridges is rarely used. However, due to their functional qualities the constructions of this type are more and more frequently employed. Concrete constructions are hardly used for building demountable bridges because of their considerable weight and inconvenience of joining them into a span. They are, however, superior to steel structures due to their longer service life, functional qualities and smaller labor demands.

Adaptation of typical prefabricated prestressed and reinforced concrete beams for demountable bridges building as well as some results of experimental and theoretical investigations corroborating their usability are presented in the paper.

2. DEMOUNTABLE SPANS

Prefabricated box girders are commonly used (also in Poland) to build small and middle-sized bridges. Prefabricated box-section elements are joined into a span by means of a wet poured reinforced plate [1]. Typical beams are 9-18 m long but their height varies. A typical prefabricated span for the theoretical length - 17.50 m is shown in Fig.1. Box girders filled with a light material are of simple shape and small weight which does not exceed 80 kN for a beam 18 m long. These characteristics make the girders especially suitable for demountable bridges of small and average spans. Cross-sections of demountable box bridges of partially changed reinforcement are shown in Fig.2. Since shackles are closed, the top surface of the beam is smooth, only assembly slings protrude slightly. In the cross-section shown in Fig.2a, a curb with rails made of angle or channel bars, fastened by means of a rifle anchor is concreted of the lateral box element (right or left). Other elements of the railing are fastened with screws. Box beams placed side by side form the bridge deck on which, after sealing contacts between the beams with fibreboard, an asphalt carpet 0.03-0.05 m thick is laid, thus covering assembly slings of the beams. Operation life being terminated, the asphalt-concrete bridge flooring is removed and all the other elements are demounted and transported to another place. A demountable bridge of a smaller load capacity with separated box girders is shown in Fig.2b. Prefabricated reinforced slabs of the plate deck (1.40 x 2.00 x 0.10 m) of full section or with vertical holes (to reduce weight and drain off rain-water) form the road. Different cross-sections of prefabricated beams are shown in Fig.3 [1], [4]. The main box-section (or I-section) girder is split vertically into two channel-section elements in order to reduce the weight of the box elements. The channel-section elements are joined into beams on the spot by means of high tensile bearing-type bolts or demountable bar cables. The main channel elements are made as reinforced elements from B-30 concrete and Q_r = 360 MPa steel. Un-

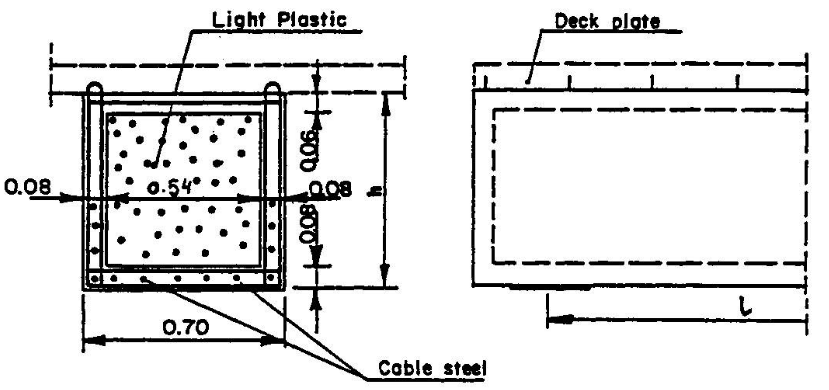

Fig.1 A typical prefabricated box span l = 17.50 m

desirable initial prestressing strains of elements do not occur due to the use of reinforced concrete. Methods of joining the elements into a span shown in Fig.3 are similar to those presented in Fig.2 except the method shown in Fig.3b, where the section is prestressed by means of demountable bar cables.

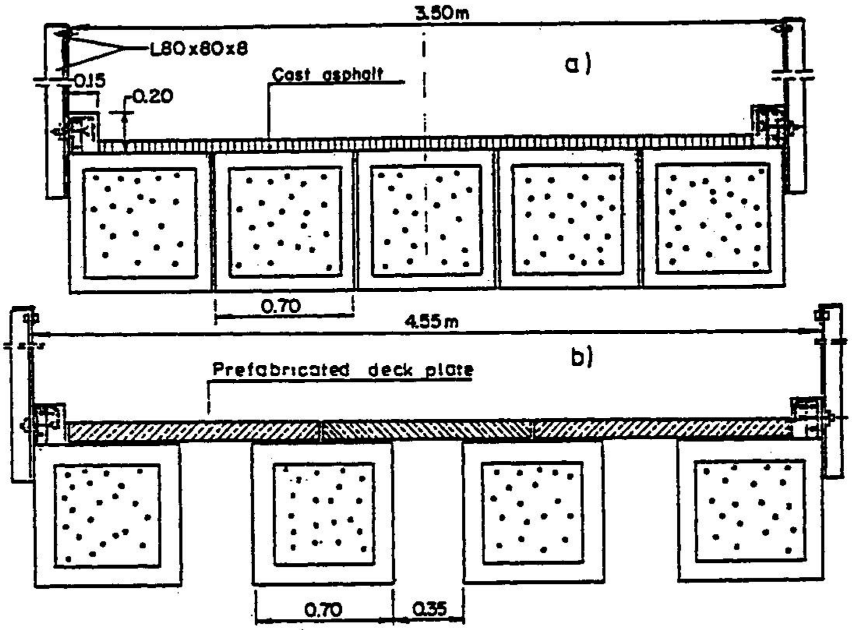

Fig. 2 The cross-sections of demountable reinforced box span

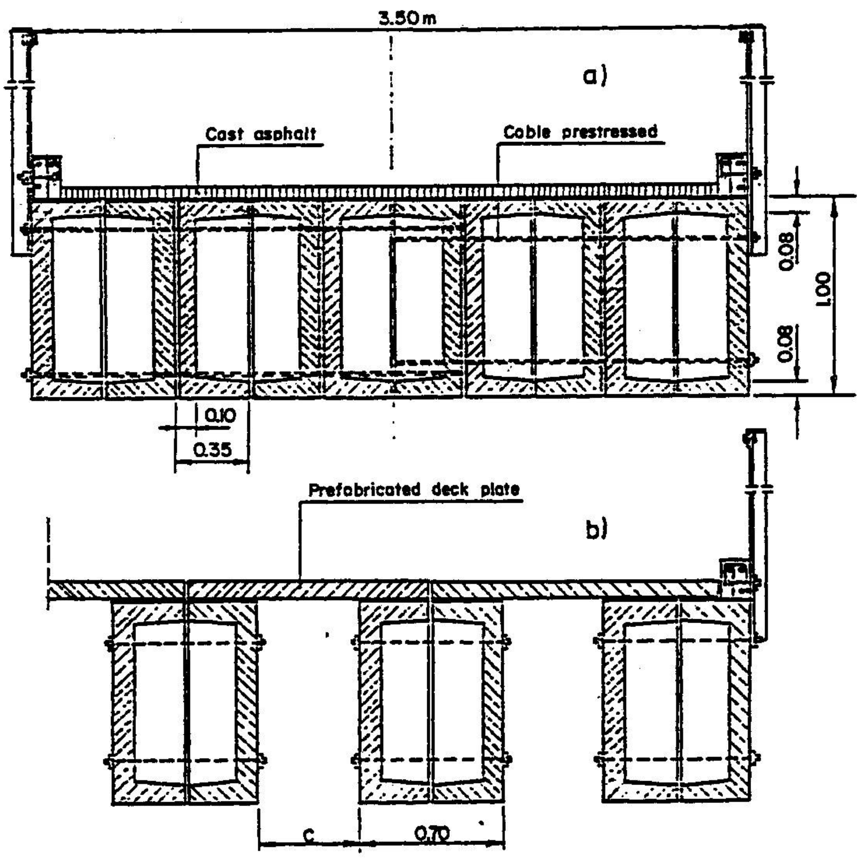

Fig.3 The cross-sections of demountable prefabricated box span

3. EXPERIMENTAL AND THEORETICAL INVESTIGATIONS

The presented designs of prefabricated demountable bridges are characterized by the application of main box-section girders. The box-section has high flexural and torsional rigidity which is especially important in demountable bridges where there is no transverse mating between main girders and the load is often carried along the edges of beams, thus causing bending and considerable torsion of the section. The experimental and theoretical investigations were carried out to determine the work and load capacity of box beams. A calculation program based on the finite strips method and checked during the analysis of steel bridges was applied to the theoretical analysis [2], [3]. Loads were estimated after Polish loading standards according to which wheel loads placed along the longitudinal bridge axis at every 1.60 m, were 60 kN. The schemes of loading realized during the experiments are presented in Fig.4. The results of the analysis for a closed-type box beam with the top deck of different thickness are presented in Fig.5. The theoretical analysis corroborated the high load capacity of the beams. Then, experimental investigations of a prestressed beam, selected from current production, were carried out. The detailed description and results of the experiments are published in [4].

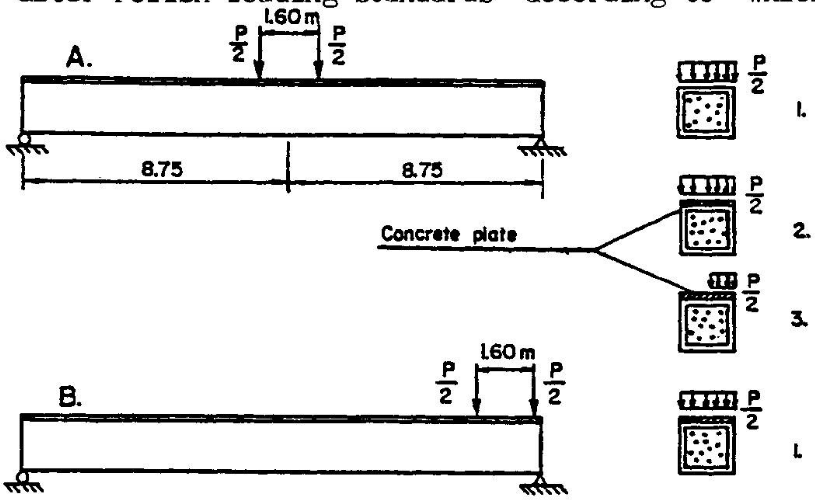

Fig.4 Schemes of loading during the tests

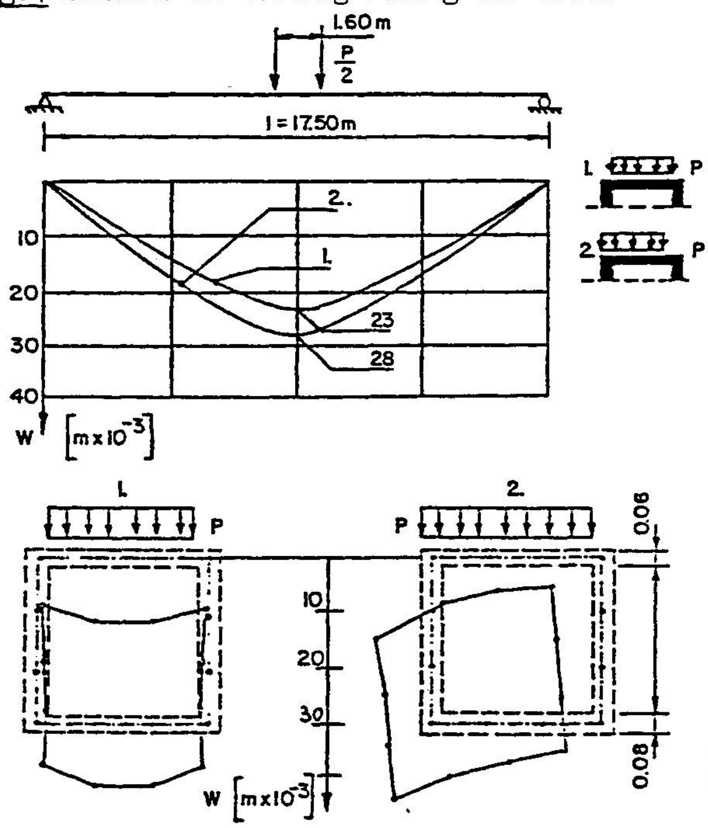

Fig.5 The results of the analysis for a closed-type box beam

The analysis aimed at:

- determining the actual load capacity and effort of the beams under increasing and cyclic loadings,
- estimating the effect of loading on permanent displacements of the beam,
- evaluating the work of the beams under torsional load (load causing torsion).

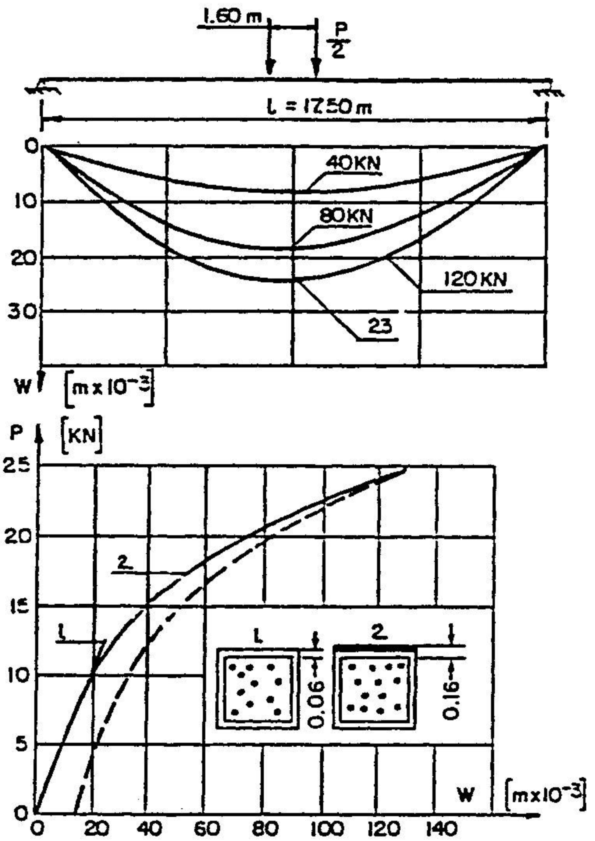

Fig.6 The exemplary distribution of dis placements for A-1 scheme of load

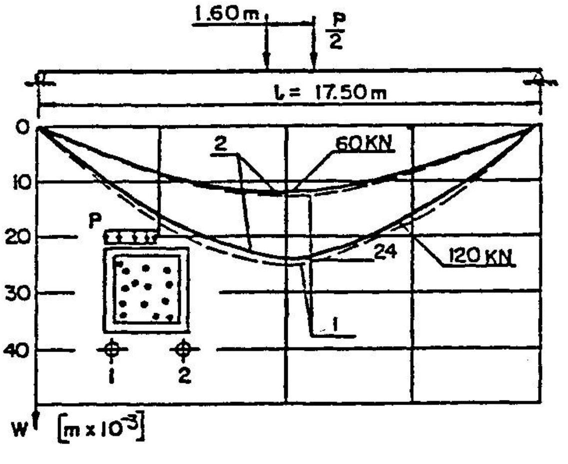

Fig.7 The relation between deflections and loads under maximal loading

The investigations were carried out on a test stand where necessary loads were imposed by dilating the beam and the supporting structure with hydraulic lifts. The force was measured by means of elastic dynamometers.

The investigations were carried out in the following way: after setting up the beam on the stand, the tests were carried out according to A-1 scheme. At this stage the maximal loading was 120 kN. Then, a reinforced concrete slab joined with a beam 0.08 m thick was concreted to the box beam, thus thickening its top plate. A-1 scheme was repeated and then A-2 and B-1 (P_{max} = 180 kN) schemes were applied. The beam was subjected to recurrent loading of the amplitude of 20-120 kN and 2000 cycles and then it was destructed (A-1 scheme being applied). The exemplary distributions of deflections of the beam along its length for A-1 scheme before and after the imposition of recurrent loading as well as the relationship between displacements in the middle of the span and loads of successive schemes are presented in Fig.6. A-3 scheme was applied to examine the work of the beam under off-centre loading (when the edge of the beam is loaded by a vehicle). The relationships between displacements of vertical deflections of the central section of the beam and imposed loads as well as deformations of the cross-section under maximal loading (120 kN) are shown in Fig.7. It is worth noting that there is a linear dependence of strains and loading and that the measured maximal strains are smaller than the theoretical ones. Changes of strains in the concrete reinforcement of the beam are presented in Fig.8 as a function of increasing loading up to the breaking (failure) load. Strains in the strings increase linearly up to the load of 120 kN which corresponds to the scrach ing moment. The scratching load being exceeded, the inclinaction angle of the P/ decreases while the strains still increase linearly.

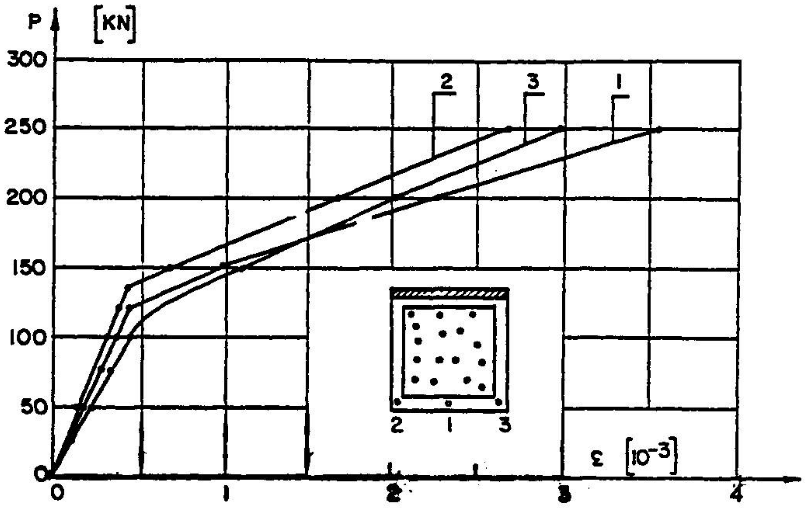

Fig.8 Changes of strains in the concrete reinforcement of the box beam

4. CONCLUSIONS

The investigations of the box beam showed that the prefabricated box-section girders may be applied for demountable bridge buildings. The application of these elements is especially advantageous because of their small dead weight and high torsional and flexural rigidities. These properties facilitates site assembly and transport of the elements and cut down the time of building the objects.

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Les viaducs de Sylans et des Glacières

Sylans und Glacières Brucke

Sylans and Glacières Viaducts

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RÉSUMÉ

Cet article présente les principales innovations des viaducs de Sylans et des Glacières situés sur l'autoroute A40 entre Mâcon et Genève. La structure du tablier est une poutre caisson tridimensionnelle en béton précontraint, constituée de voussoirs préfabriqués. La méthode de construction comporte quelques améliorations permettant de réduire les cycles de construction et les efforts transmis aux appuis.

ZUSAMMENFASSUNG

Dieser Aaufsatz setzt die meisten Neuerungen der Sylans und Glacières : Brücke vor, die auf der A40 Autobahn zwischen Mâcon und Genève stattfinden. Der Brückenbelagstruktur ist ein aus Spannbeton dreidimensional Kastentrager, der aus vorgefertigten Säcke besteht. Die Neuerungen der Bauweise verfähren eine Verkürzung der Baukreisprozesse und der Stützenkrafte.

SUMMARY

This article discusses the novel design features of the Sylans and Glacières viaducts located along the A40 highway between Mâcon and Geneva. The decks of the structures consist of three-dimensional post-tensioned concrete truss girders composed of precast segments. The construction method involves some innovations leading to reduced erection times and decreased foundation loads.

1.0 INTRODUCTION

Les viaducs de Sylans et des Glacières situés sur l'autoroute A 40 Mâcon-Tunnel du Mont Blanc (fig. 1), actuellement en cours de réalisation, font appel à des solutions originales, tant pour la structure que pour le mode de réalisation. Ces ouvrages constituent une étape importante dans l'évolution des ponts en béton de moyenne portée.

Ces viaducs sont constitués de deux dalles, le hourdis supérieur et le hourdis inférieur, réunis par un réseau de treillis tridimensionnel précontraint (fig. 2). La géométrie des tabliers est complexe : les rayons de courbure sont variables avec un minimum de 424 m, ainsi que les dévers qui varient de -2,5 % à 6,5 %. Ces ouvrages ont une longueur développée de 2 x 1266 m pour le viaduc de Sylans et 2 x 214 m pour le viaduc des Glacières. Ces ouvrages portent chacun une chaussée de 9,75 m de large. La largeur de la structure est de 10,75 m.

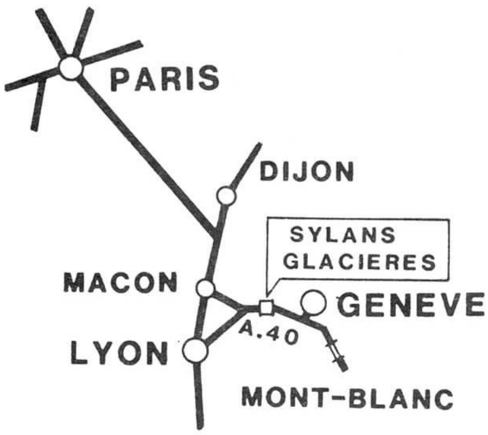


Fig.1.

PERSPECTIVE D'UN VOUSSOIR

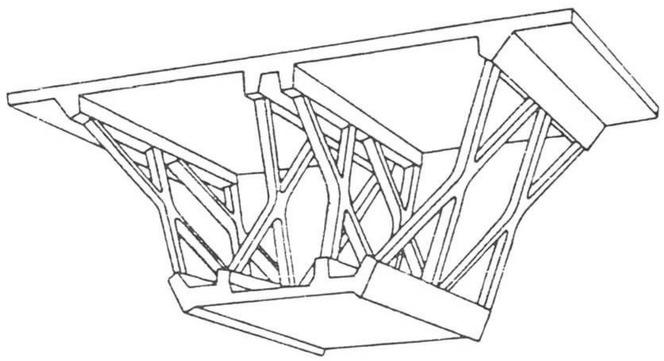
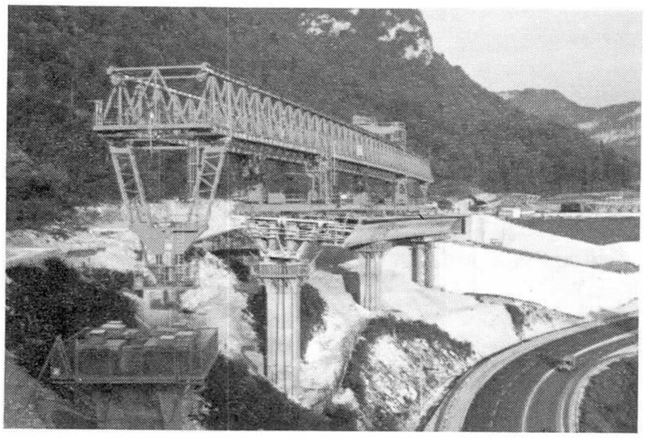
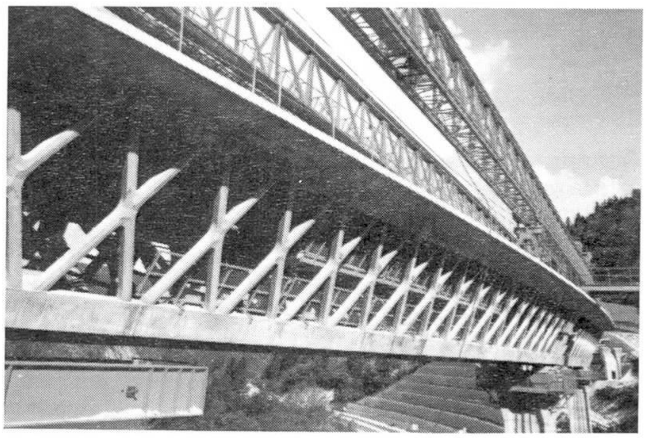


Fig.2





2.0 RAPPEL HISTORIQUE

La conception du tablier est le fruit de l'expérience de l'Entreprise BOUYGUES qui a développé sous l'impulsion de Pierre RICHARD trois schémas de structures triangulées.

2.1 LA COUVERTURE DU STADE DE TEHERAN

Les poutres treillis du stade ARYAMEHR (TEHERAN-1974) de 82,30 m de portée constituent l'ossature secondaire de la toiture d'une piscine et d'une salle omnisport. La triangulation en N en élévation, s'écarte en V transversalement de manière à assurer la sécurité au déversement latéral dans les phases provisoires. Les diagonales de 0.40 m x 0.50 m de section se trouvent tendues sous le poids propre. Elles sont précontraintes par les câbles longitudinaux, qui se dévient au droit des noeuds inférieurs. Les montants naturellement comprimés ne sont pas précontraints. Ces poutres toutes parallèles s'encastrent dans deux poutres primaires caisson de portée 82,30 m également, réalisant un ensemble de 108,4 m par 108,4 m y compris les porte-à-faux de 13,05 m.

La construction a utilisé toutes les possibilités de la préfabrication. Les noeuds, les diagonales, les montants et les membrures sont préfabriqués, un clavage, à chaque extrémité, est coulé sur un banc d'assemblage. La poutre une fois précontrainte est levée puis ripée et solidarisée aux poutres primaires (fig. 3).

2.2 LE PONT DE BUBIYAN

Le pont de BUBIYAN (KOWEIT-1983) est constitué de 59 travées de 40.16m et d'une travée de 53.84m, réparties en 6 viaducs indépendants. Le tablier large de 18.20 m se compose d'une poutre caisson à 8 âmes triangulées transversalement. Chaque plan d'âme se compose d'un treillis en V avec montants dédoublés de part et d'autre des noeuds supérieurs. La mise en oeuvre des diagonales (20x20) et montants (16x20) est ici résolue grâce à la préfabrication de triangles en béton armé. Ceux-ci sont munis de barres en attente noyées dans les membrures lors du coulage.

La précontrainte longitudinale totalement extérieure se dévie au droit des noeuds inférieurs exerçant des efforts verticaux équilibrant ceux des charges gravitaires. Les diagonales n'ont donc qu'à équilibrer une part très faible de l'effort tranchant total. La limitation sévère de la fissuration garantit leur durabilité. Le tablier est ici assemblé travées par travées ; les voussoirs sont suspendus en porte à faux à un lanceur haubané. La conjugaison des voussoirs est réalisée sans résine (fig. 4).

2.3 LES VIADUCS DE SYLANS ET DES GLACIERES

Ces ouvrages sont d'une conception différente. Le schéma de triangulation longitudinale est donnée par des X situés dans quatre âmes inclinées. Toutes les diagonales sont précontraintes. La précontrainte longitudinale est partiellement extérieure. Des câbles intérieurs dits de goujonnage traversent tous les noeuds. La méthode de construction, pose de voussoirs préfabriqués en encorbellements successifs, exige l'utilisation d'unités de précontrainte de faible puissance, et des bossages régulièrement répartis le long des ouvrages (fig. 5).

STADE d' ARYAMEHR

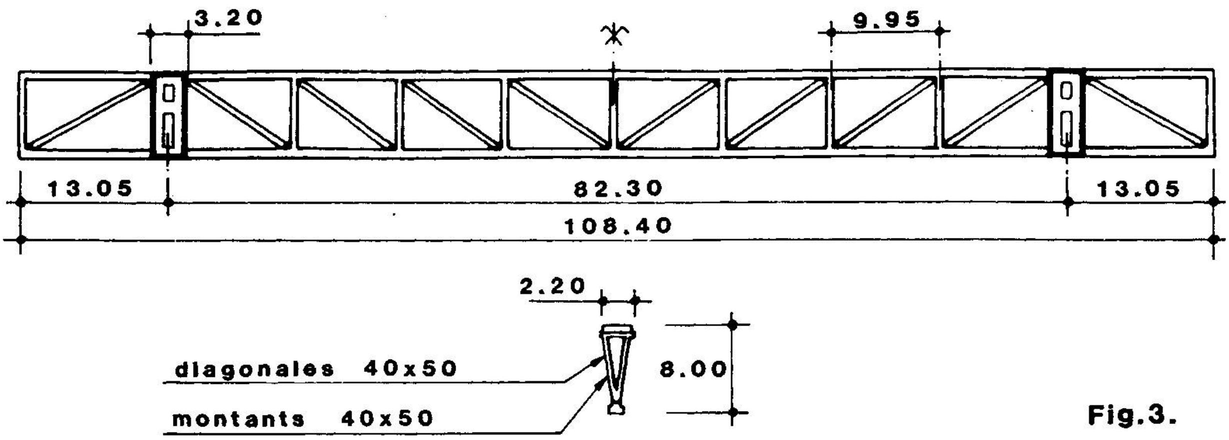


Fig.3.

PONT de BUBIYAN

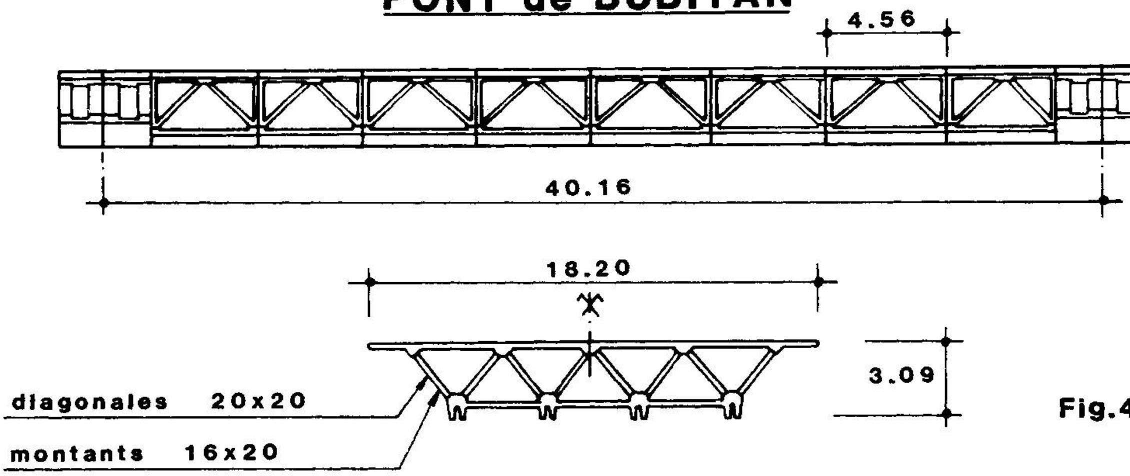


Fig.4.

VIADUCS de SYLANS et des GLACIERES

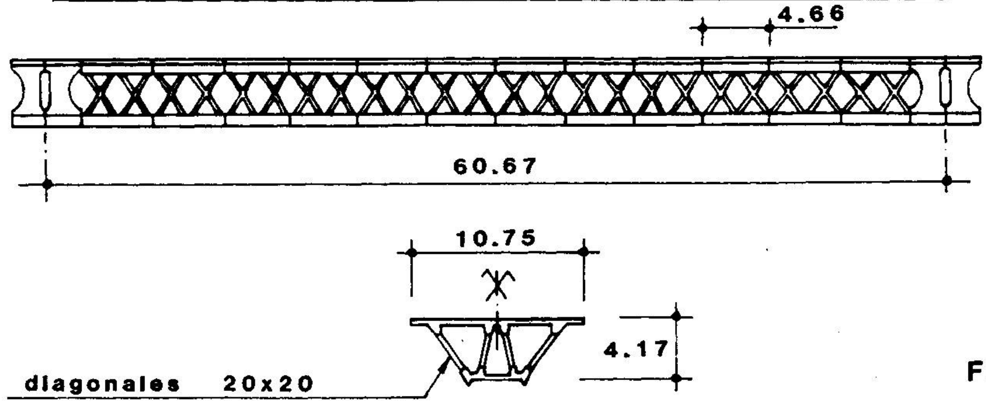


Fig.5.

3.0 BASES DU DIMENSIONNEMENT

Les règles de dimensionnement ont été fixées par l'Administration au moment de la mise au point du projet. Les principes suivants ont été retenus :

- Les fibres extrêmes du tablier restent totalement comprimées sous les sollicitations à l'état limite ultime.
- Tout noeud inférieur ou supérieur doit être traversé par au moins un câble intérieur au béton.
- Les diagonales sont précontraintes de telle manière que l'effort normal soit toujours une compression sous combinaisons de service. Les moments locaux sont repris par des aciers passifs justifiés en fissuration préjudiciable. A l'état limite ultime les aciers passifs et les surtensions des câbles équilibrent les efforts de traction (fig. 6).

COUPE TRANSVERSALE

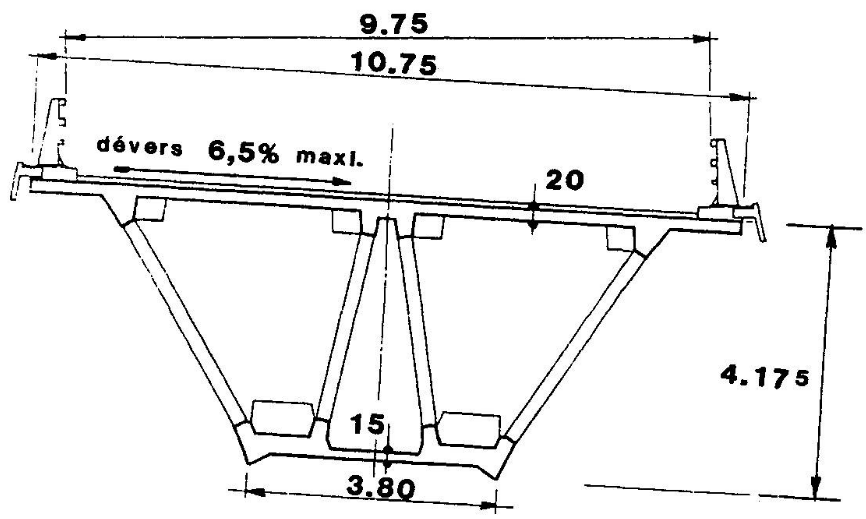


Fig.6.

4.0 PARTICULARITES DES OUVRAGES

La précontrainte extérieure est constituée d'unités 12T15 symétriques par rapport aux appuis, qui s'ancrent dans des bossages inférieurs communs aux blocs déviateurs. La limitation de l'effort du relevage par bossage a conduit à dissocier l'ancrage et le déviateur. Certains de ces câbles se croisent en travée de manière à limiter la précontrainte de clavage à une paire de câbles 12T15. Comme recommandé dans les circulaires ministérielles, cette précontrainte est démontable. La protection des torons est assurée par une gaine continue de polypropylène injectée de coulis de ciment.

Les câbles de fléau supérieurs sont constitués de 4 câbles 4T15 par voussoir ancrés en bossage. Ces câbles sont provisoires et réutilisables, excepté 4 câbles intérieurs qui sont conservés (fig. 7).

Des câbles inférieurs 7T15 sont ancrés dans les voussoirs courants adjacents aux VSP. Ces câbles, intérieurs au béton, sont nécessaires au maintien des sections entièrement comprimées à l'état limite ultime. Il est prévu en plus, comme c'est l'usage, une précontrainte complémentaire pour pallier les aléas liés aux coefficients de frottement et une précontrainte additionnelle pour un renforcement ultérieur éventuel.

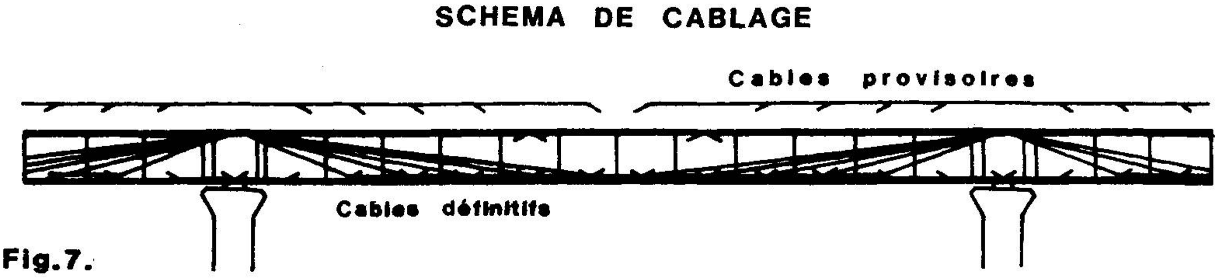


Fig.7.

Les voussoirs sur appui sont constitués de voiles de 20 cm inclinés nervurés qui permettent l'équilibrage des poussées de déviation des câbles extérieurs avec les réactions d'appui. Les nervures en V renversés ont été disposées côté intérieur pour le raidissement sous efforts transversaux. Les voussoirs de culée, demandent un soin particulier afin d'ancrer les 2 000 tonnes de précontrainte extérieure. La poutre transversale, de 1 m d'épaisseur, dans laquelle s'ancrent les câbles, est bloquée en rotation sur deux nervures transversales indépendantes des voiles extérieurs.

Le Viaduc de Sylans (1 266 m de long) a été divisé en trois ouvrages. Ceci a nécessité 2 piles culées. Les voussoirs de pile culée, semblables aux voussoirs sur culée, doivent en plus assurer pendant la pose du fléau le monolithisme de la structure ; ils comportent pour ce faire un jeu de cales supérieures et inférieures permettant un brêlage par les câbles de fléaux provisoires.

La résistance du béton des diagonales fixée par le CCTP à 37 MPa, a été améliorée pour obtenir une résistance élevée au jeune âge. Cette disposition est nécessaire pour raccourcir les cycles de stockage et de manutention de ces pièces minces. La résistance élevée du béton permet de limiter la fissuration des pièces au levage. Le module d'élasticité plus élevé permet de réduire leur déformabilité à la mise en tension et au stockage. En outre, l'amélioration des performances confère à l'ouvrage en service, une sécurité supplémentaire garante de la durabilité des diagonales. La résistance caractéristique du béton atteint 65 MPa à 28 jours.

5.0 MODE DE CONSTRUCTION

Les ouvrages treillis permettent une préfabrication à différents stades. La préfabrication commence d'abord par les X. Il faut noter que ces pièces sont longues de 3.50 m et ont une section de 20 x 20 cm ; elles comportent deux tubes de 40 mm de diamètre, équipés de câbles 5 ou 10 ϕ 7 tendus ultérieurement. Le ferraillage est constitué de 8 barres de 20 mm pour les pièces les plus sollicitées (fig. 8). Ensuite, ces X sont mis en place dans la cellule où l'on coule le voussoir qui se limite aux deux hourdis avec leurs différents bossages.

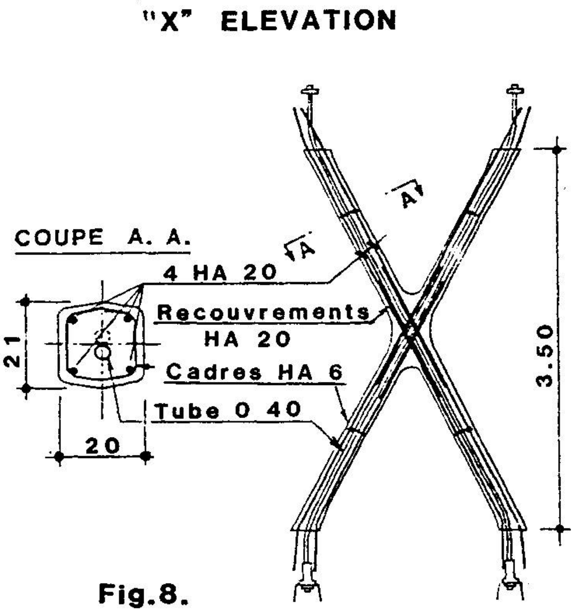


Fig.8.

Le seul point particulier est que l'on exécute ici des tronçons de 423 m, intégralement conjugués, ce qui nécessite une organisation très poussée pour le contrôle géométrique : mesures, traitement sur le site, et suivi permanent par le Bureau d'Etudes. Les voussoirs d'appui sont fabriqués dans la même cellule.

La pose est effectuée avec une bipoutre (celle qui a posé les viaducs de l'échangeur de St Maurice près de Paris) prévue pour ne pas avoir à se déplacer au cours de la pose d'un fléau et pour poser les voussoirs par paire, symétriquement par rapport à la pile (fig. 9). Cette poutre est équipée de suspentes permettant la suspension provisoire du voussoir avant, le voussoir arrière étant toujours suspendu au portique de pose. Cette disposition permet de réduire de moitié les moments de déséquilibre transmis aux appuis. Par sécurité il a été pris en compte le déséquilibre d'un demi-voussoir.

La mise en précontrainte de chaque paire de voussoirs est effectuée par 4 câbles 4T15 extérieurs, provisoires, et 2 barres inférieures de 36 mm de diamètre.

PRINCIPE DE POSE EN SYMETRIE

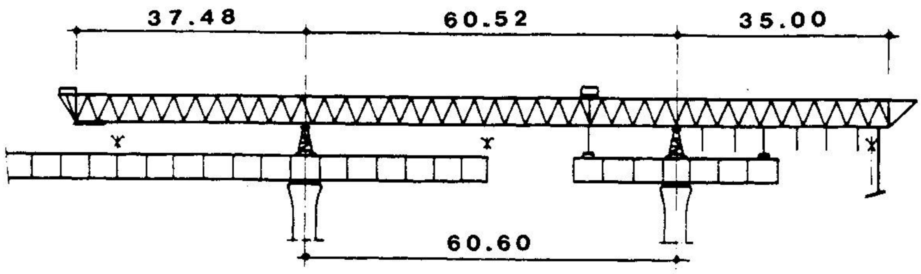


Fig.9.

Le fléau est construit 20 cm en avant sur 4 cales provisoires ; au moment du clavage, des appuis glissants sont interposés et le clavage est effectué par ripage du fléau. Un vérinage de l'ordre de 5 cm (vers le haut) est effectué antérieurement sur la pile précédente de manière à rendre parallèles les plans de joint au moment du clavage (fig. 10).

PRINCIPE D'ASSEMBLAGE DES FLEAUX

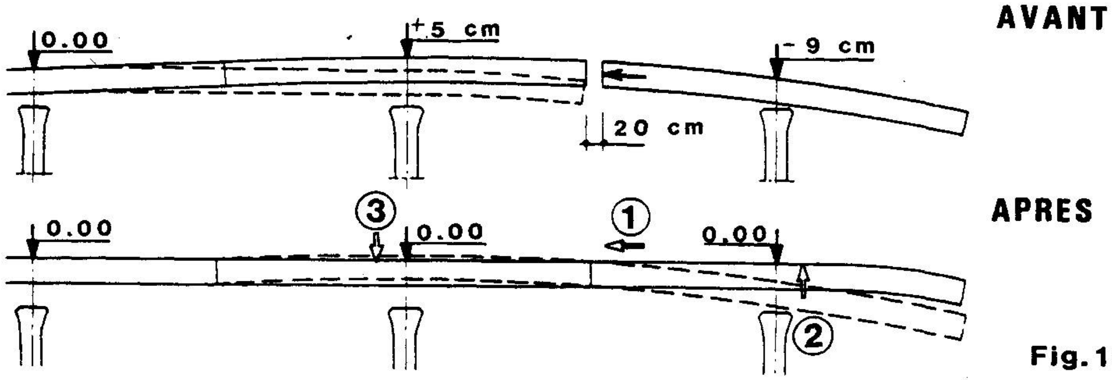


Fig.10.

6.0 CONCLUSION

Les viaducs de Sylans et des Glacières ont permis de progresser tant dans le domaine structurel par l'économie des quantités, que dans le domaine constructif par la réduction importante des cycles de pose.

En outre, la réduction des efforts dus aux déséquilibres en cours de pose, associé à l'allègement de la structure, ont permis un gain notable sur les fondations.

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Protection contre les chocs de navires dans le projet Euroroute

Schutz der Brückenpfeiler gegen Schiffsanprall im Projekt Euroroute

Protection of Piers against Impact of Ships in the Euroroute Project

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Jean-Paul Teyssandier, né en 1944, est ancien élève de l'École Polytechnique et de l'École Nationale des Ponts et Chaussées. Il s'est particulièrement occupé de la réalisation de grands ouvrages, dans l'Administration française, puis dans l'Entreprise.

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RÉSUMÉ
Après une brève présentation du projet Euroroute de lien fixe trans-Manche, l'article décrit le système de protection des piles de pont contre le choc de navires importants. Celui-ci se compose d'anneaux en béton de grand diamètre, qui servent également de flotteurs pour la construction des ouvrages.

ZUSAMMENFASSUNG
Nach der Vorstellung des Projektes Euroroute, welches eine feste Verbindung durch den Ärmelkanal beinhaltet, wird das System beschrieben, welches die Brückenpfeiler vor dem Anprall grosser Schiffe schützen soll. Der Schutz besteht aus Betonringen mit sehr grossem Durchmesser, welche auch als Schwimmkörper beim Bau der Brücke dienen.

SUMMARY
In this paper, the Euroroute project for a permanent passage through the English Channel, is outlined. Then, the system of protection of piers against impact from large ships is described. This system consists of large-diameter concrete rings, also used as floaters during the erection of the structures.

1. PRESENTATION DU PROJET

1.1 - Introduction

Dans le cadre du concours lancé par les Gouvernements français et britannique en Avril 1985 pour la traversée de la Manche, le Groupement EUROROUTE remit une offre qui présentait une originalité technique certaine, étayée par des études approfondies.

Le Pas-de-Calais est un bras de mer d'environ 40 km de largeur et d'une profondeur maximale de 60 m au droit du franchissement. C'est la voie maritime la plus circulée au monde. Toutefois la navigation y est très règlementée et, de ce point de vue, plusieurs zones sont définies:

- au centre, sur 20 km de large, règnent les chenaux principaux de navigation, dans lesquels l'établissement de tout obstacle fixe est interdit.

- de part et d'autre de ces chenaux, se trouvent les zones côtières, dans lesquelles des obstacles fixes peuvent être implantés sous réserve de dégager un gabarit minimal de 400 m de large entre appuis.

1.2 - Caractéristiques principales du projet

La liaison routière du projet EUROROUTE se compose:

- d'un tunnel immergé, d'une longueur de 20 km, sous les chenaux principaux de navigation

- de deux ponts, l'un de 10 km dans la zone côtière britannique et l'autre de 7 km dans la zone française

- de deux îles artificielles assurant la liaison entre chacun de ces ponts et le tunnel immergé.

Parmi ces différents éléments, nous allons plus particulièrement nous intéresser aux ponts et à leur système de protection contre le choc des navires.

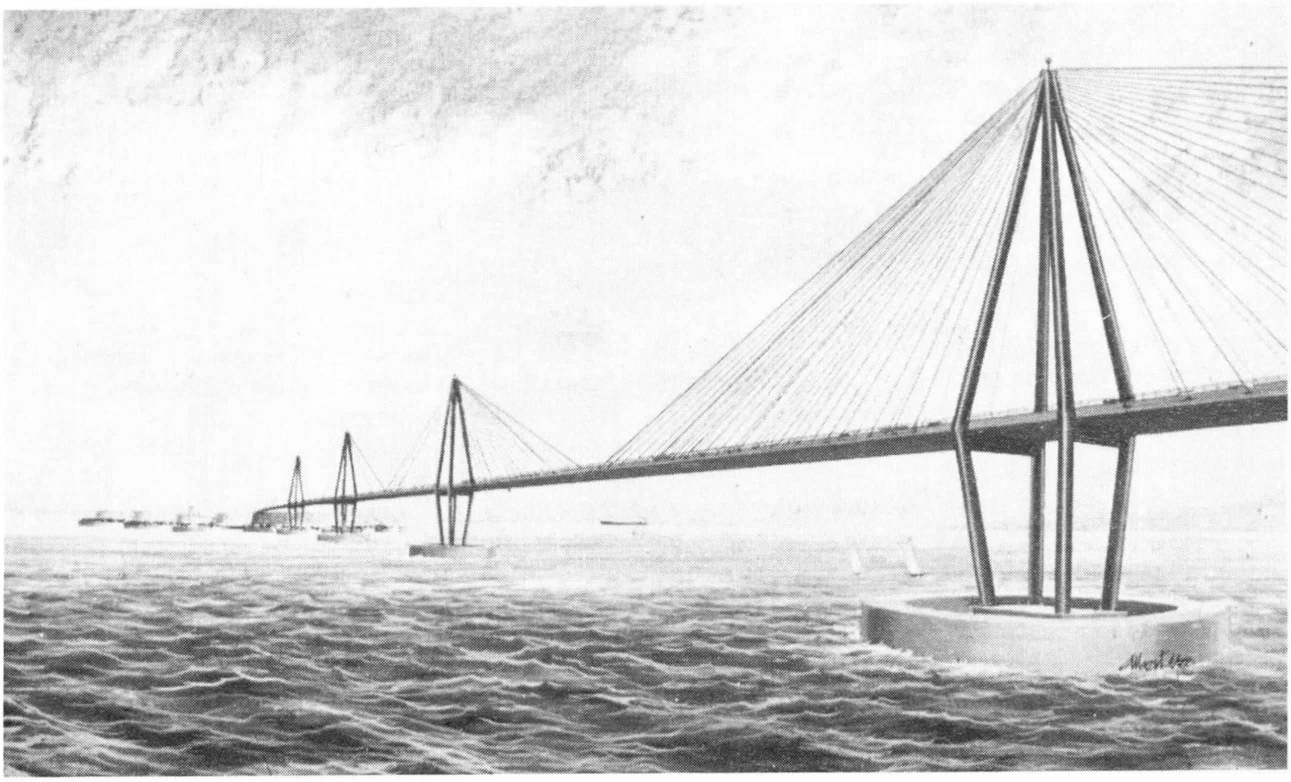

Fig. 1 - Vue générale d'un pont

2. PRESENTATION DES PONTS

2.1 - Caractéristiques principales des ponts

Les ponts se présentent comme une succession d'ouvrages à haubans de 500 m de portée. Chacun d'eux se compose de deux parties en porte-à-faux de 218 m, réunies à l'ouvrage voisin par une travée indépendante de 64 m de longueur.

Le tablier est formé d'un caisson métallique rectangulaire de 2,50 m de hauteur et de 22,10 m de largeur.

Les pylônes sont constitués de 4 tubes métalliques de 3,50 m de diamètre emplis de béton, formant une pyramide sur une hauteur d'environ 90 m au-dessus du tablier. Les haubans, ancrés de part et d'autre du tablier, convergent au sommet des pylônes (voir Fig. 2).

Les appuis sont constitués de caissons rectangulaires en béton, de 35 m x 20 m, cloisonnés.

Ces caissons sont directement fondés sur la craie affleurante au fond de la mer. Autour de ces caissons sont disposés des anneaux de protection en béton de 80 m de diamètre extérieur et de 16 m d'épaisseur. Caissons de fondations et anneaux sont lestés par du sable (voir Fig. 3).

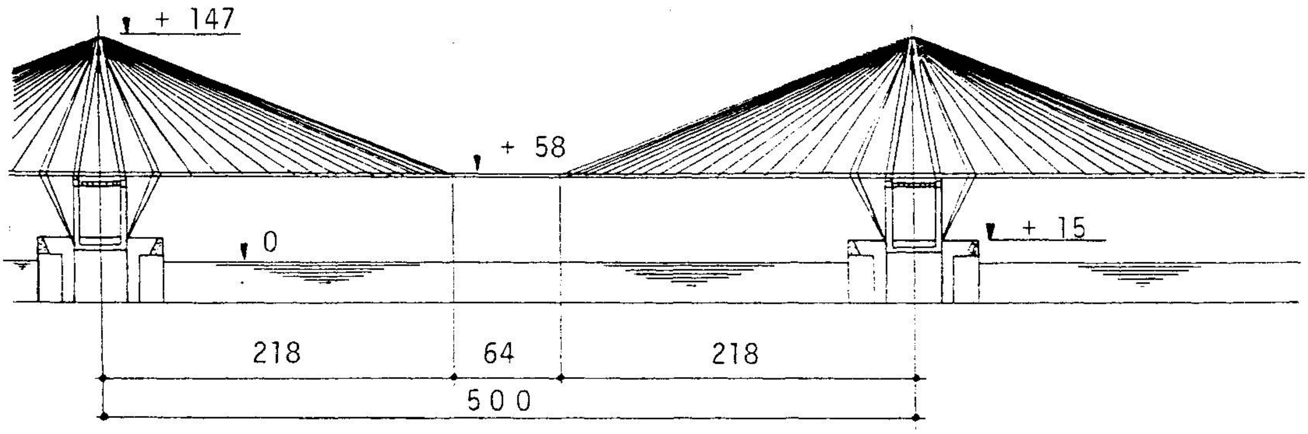

Fig. 2 - Elévation d'une travée de pont

2.2 - Raisons de ces dispositions

Les pylônes assurent à eux seuls la stabilité de l'ouvrage vis-à-vis de la dissymétrie des surcharges et des efforts du vent. C'est la raison pour laquelle ils présentent 4 jambes convergentes au sommet, ce qui leur assure une excellente rigidité de flexion et de torsion.

De même la suspension latérale des haubans, avec convergence au sommet, assure une excellente rigidité de torsion, nécessaire pour la stabilité aéro-élastique.

L'existence d'une travée centrale diminue les cassures de profil en long aux extrémités des porte-à-faux sous l'effet des surcharges et assure une indépendance totale entre travées, pour éviter un effondrement en chaîne de l'ouvrage.

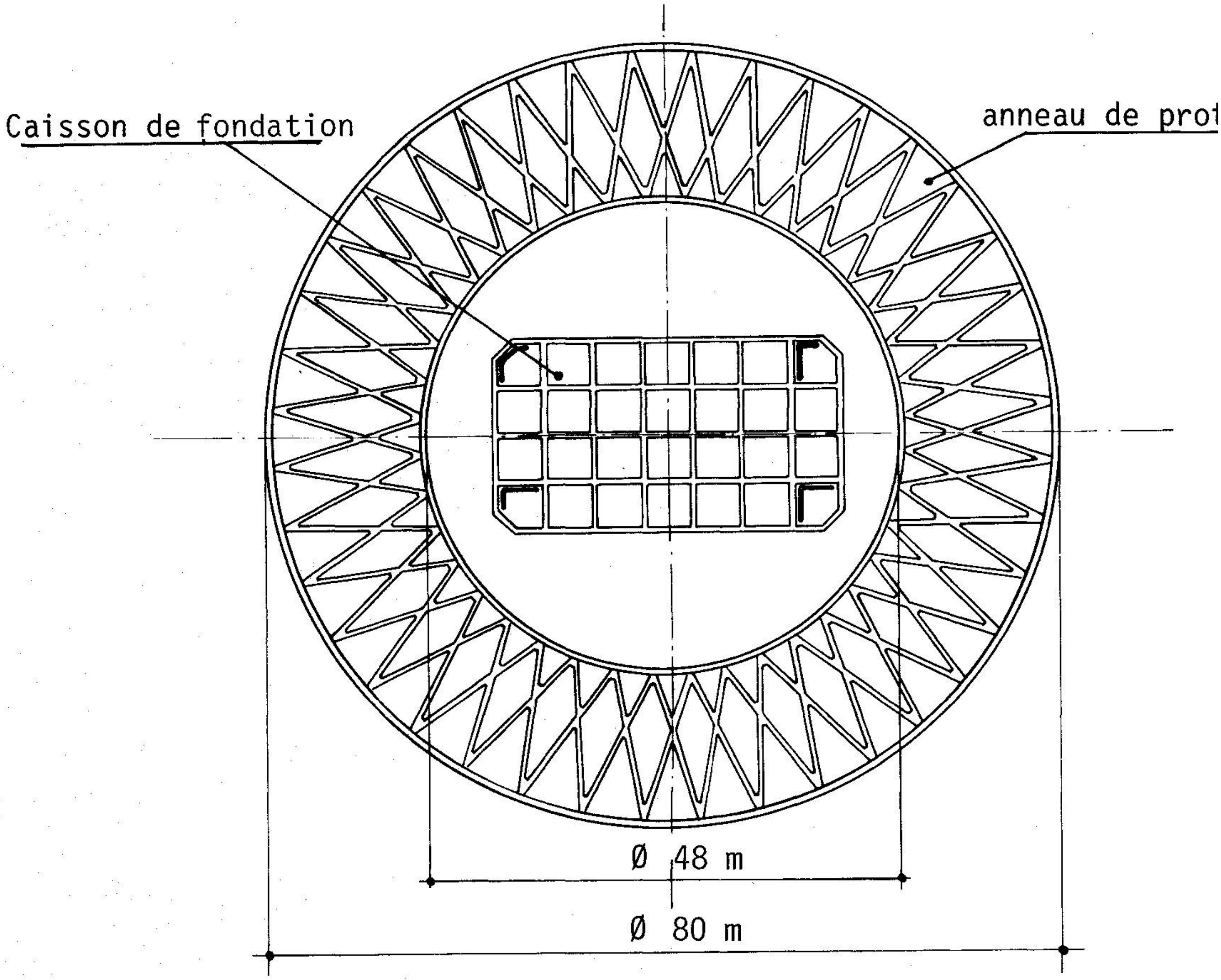

Fig. 3 - Caisson de fondation et anneau de protection

3. PROTECTION CONTRE LES CHOCS DE NAVIRE

3.1 - Présentation du problème

Les règles du concours prévoyaient que les obstacles fixes devaient résister au choc d'un navire de 300 000 tonnes lancé à 17 nœuds. C'est la première fois qu'une telle résistance est demandée à des appuis d'ouvrage.

Pour protéger les piles de pont contre des chocs de navires importants on a traditionnellement recours à des remblais. Or dans le cas considéré, la réalisation de tels remblais soulevait deux objections fondamentales:

- la profondeur d'eau et les houles importantes auraient conduit à mettre en place de gros volumes de matériaux, avec des carapaces de protection très importantes. Une telle réalisation aurait été forcément très coûteuse.

- d'autre part l'ensemble de ces îles aurait provoqué une obstruction importante du détroit, modifiant très certainement l'équilibre général des courants marins dans le secteur.

Ces raisons nous amenaient au contraire à concevoir des protections préfabriquées, qui puissent donc être aisément mises en place, et aussi compactes que possible.

3.2 - Estimation des forces d'impact

Pour concevoir de telles structures, il convenait tout d'abord de définir les forces auxquelles elles devaient résister.

Des études, menées en collaboration avec les chantiers navals membres du groupement EUROROUTE (Alsthom côté français, British Shipbuilders côté britannique), ont permis de déterminer la valeur des forces développées lors d'un impact entre un navire et un obstacle fixe, à partir des plans de navires existants.

Dans le cas d'un choc frontal, on a supposé que la totalité de l'énergie était absorbée par écrasement de l'avant du navire. Les études menées à partir de la structure d'un pétrolier existant de 317 000 tonnes de déplacement maximal, ont montré que la force d'écrasement en fonction de la longueur écrasée avait l'allure présentée en figure 4. On voit que celle-ci croit rapidement pour devenir constante et égale à 550 MN. La longueur écrasée, pour une énergie correspondant à une vitesse initiale de 17 nœuds, est de 26 m. Une telle longueur est insuffisante pour atteindre les premiers réservoirs du pétrolier.

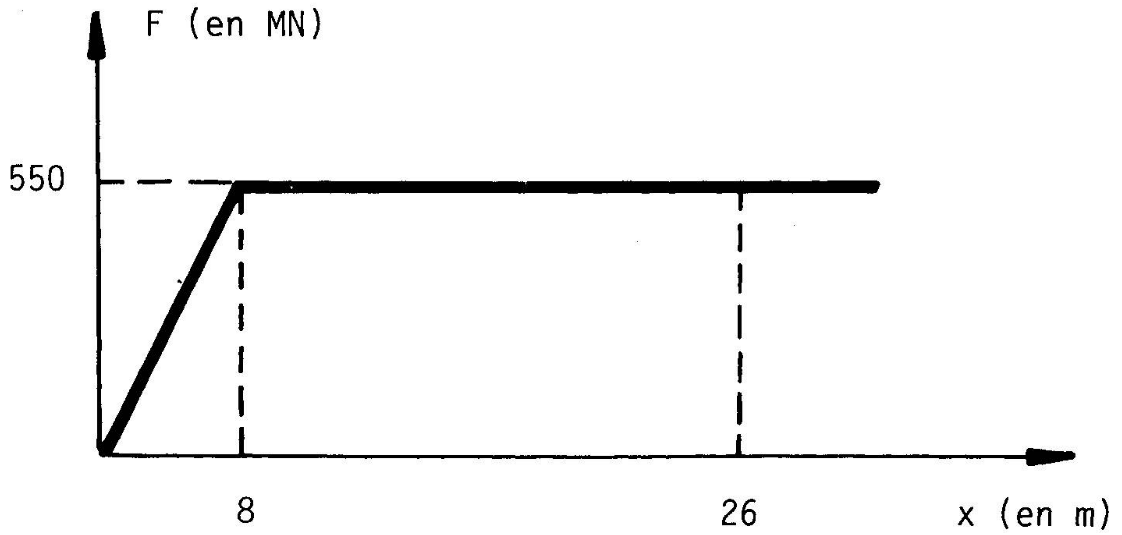

Fig. 4 - Force d'écrasement

Il est à noter que la valeur ainsi trouvée est tout à fait comparable à celle qui est donnée par l'estimation de Woisins.

Il y avait également lieu de considérer le cas d'un navire à la dérive, poussé par le courant et le vent contre une pile et venant heurter celle-ci latéralement à mi-longueur (cas le plus défavorable). Une étude conduite sur le même navire a conclu que, pour une vitesse de dérive de 4,5 nœuds, la force d'impact développée était de l'ordre de 500 MN et que le choc n'entraînait pas la rupture de la structure du navire.

3.3 - Conception de la structure de protection

Il a été décidé dès le départ de séparer les deux fonctions, d'une part appui du pont assuré par le caisson de fondation, d'autre part résistance au choc de navire assurée par l'anneau de protection. L'anneau sert en quelque sorte de structure sacrificielle destinée à être déplacée et même endommagée lors d'un choc important, sans que la fondation de l'ouvrage ait à en souffrir.

Par contre ce choix compliquait sensiblement la structure de l'anneau. Celui-ci est constitué de deux voiles cylindriques, d'un diamètre respectif de 80 m et 48 m, reliés entre eux par des murs verticaux en X. En haut et en bas de l'anneau se trouvent deux couronnes massives en béton.
Les forces d'impact sont reprises par les murs verticaux en X et retransmises aux couronnes supérieures et inférieures, qui assurent pour l'essentiel la résistance de la structure à l'ovalisation.

Une étude théorique, menée à partir du diagramme précisé ci-dessus, a montré que l'amplification dynamique en début d'impact reste très modérée.
Le poids des anneaux est tel qu'il ne se produit aucun glissement sur le sol durant l'impact, pour un coefficient de frottement de 0,50.

L'intervalle laissé entre le caisson de fondation et l'anneau permet un léger déplacement de celui-ci, par exemple s'il se produit un choc d'une intensité plus importante.

3.4 - Autre intérêt de l'anneau de protection

Dans le cadre du projet envisagé, il était capital de trouver des méthodes d'exécution aussi peu tributaires que possible des conditions météorologiques et conduisant à un délai global aussi réduit que possible. De ce fait, il apparaissait exclu de construire sur place les 34 travées de 500 m.

Les anneaux de protection ont permis en fait de concevoir une méthode originale résolvant ce problème: il suffisait de préfabriquer dans un site protégé l'ensemble d'une travée haubannée de part et d'autre du pylône, puis de l'amener par flottaison sur l'ensemble caisson-anneau, et enfin de l'échouer à son emplacement définitif.

Des calculs théoriques, ainsi que des essais au Laboratoire National d'Hydraulique de Chatou ont montré la parfaite validité de cette méthode.

Design of a Floating Berth

Conception d'un poste de mouillage flottant

Projekt eines schwimmenden Anlegeplatzes

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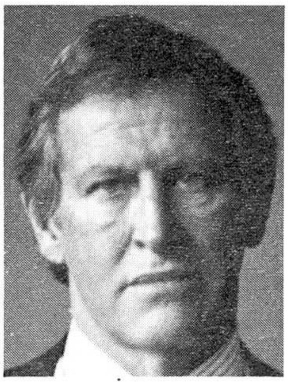
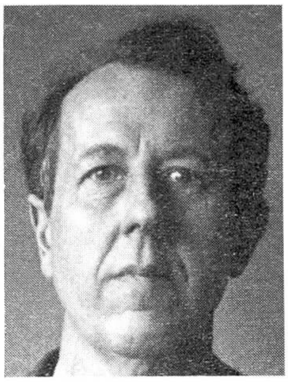


David Wainwright took his engineering degree at Imperial College, London in 1956. He has since worked in several design offices on projects involving water engineering, concrete structures and earth dams including a number of years on the Thames Barrier in London.



Peter Clark graduated in Civil Engineering in 1976 and has specialised in maritime works including the design, maintenance and inspection of coastal defences and port structures.

SUMMARY
The paper describes a covered berth for naval vessels, and the reasons underlying a novel and unique solution, which is a floating pre-stressed concrete twinhulled structure. The chosen geometry of the pontoons is discussed, model tank testing is described in some detail, together with the design procedures and an outline of the anticipated construction methods which contractors will adopt.

RÉSUMÉ
Cet article décrit un poste de mouillage couvert pour bateaux, et les raisons qui ont conduit à mettre au point cette solution unique entièrement nouvelle : celle-ci consiste en une structure flottante à double ponton en béton précontraint. La géométrie choisie pour ces pontons est expliquée et les essais de modèle en bassin sont décrits en partie. Cet article donne également un aperçu des études réalisées et un résumé des méthodes de construction prévues, qui seront utilisées par les entrepreneurs.

ZUSAMMENFASSUNG
In dieser Abhandlung wird ein überdachter Anlegeplatz beschrieben. Es werden die Gründe für eine neue und einzigartige Lösung, eine schwimmende Doppelrumpf-Struktur aus Spannbeton, angeführt. Es werden die gewählte Anordnung der Pontons diskutiert und die Modellversuche angesprochen zusammen mit den Konstruktionsvorgängen und einer Skizzierung der Vorgesehen und von den Unternehmern anzuwendenden Baumethoden.

1. INTRODUCTION

1.1 This paper describes the design of the main elements of a floating covered jetty for special naval vessels, which will provide craneage as well as support services for the vessels while moored in the berth. A floating structure was chosen as the most cost effective solution because the depth of water, 70 metres, and the sea bed-rock at a slope of 40^{o} would have rendered conventional piling extremely expensive. However, the floating solution poses problems since it will permit dynamic motions caused by wind, waves and tides.

1.2 The jetty will comprise twin pontoons, each 200m long x 25m wide, separated by 30 m of water, having a draft of 7m and a freeboard of 5m, and displacing 70,000 tonnes. At one end, the pontoons will be connected by a rigid box link and at the opposite end by an underwater tubular steel brace allowing vessels with a draft of up to 13m to enter the berth. The enclosure to the berth will be about 60m wide, a maximum of 45m high at the ridge and composed of aluminium clad structural steelwork. Two overhead electric travelling cranes and a two leaf vertical lift vessel access door will be provided. The jetty will be permanently moored to the shore by articulating tubular steel booms and provided with steel box girder bridges catering for a maximum tidal range of about 4m.

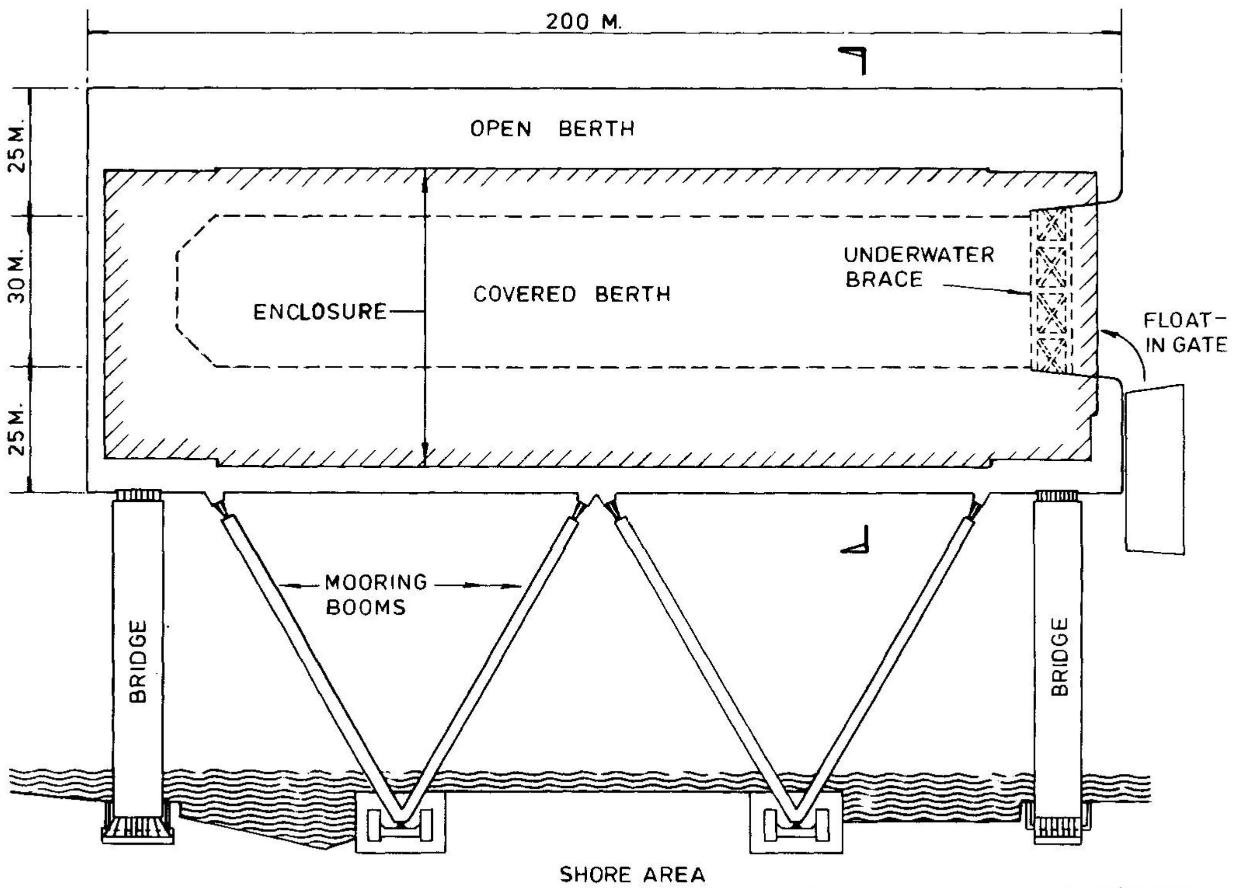


FIGURE 1 - PLAN

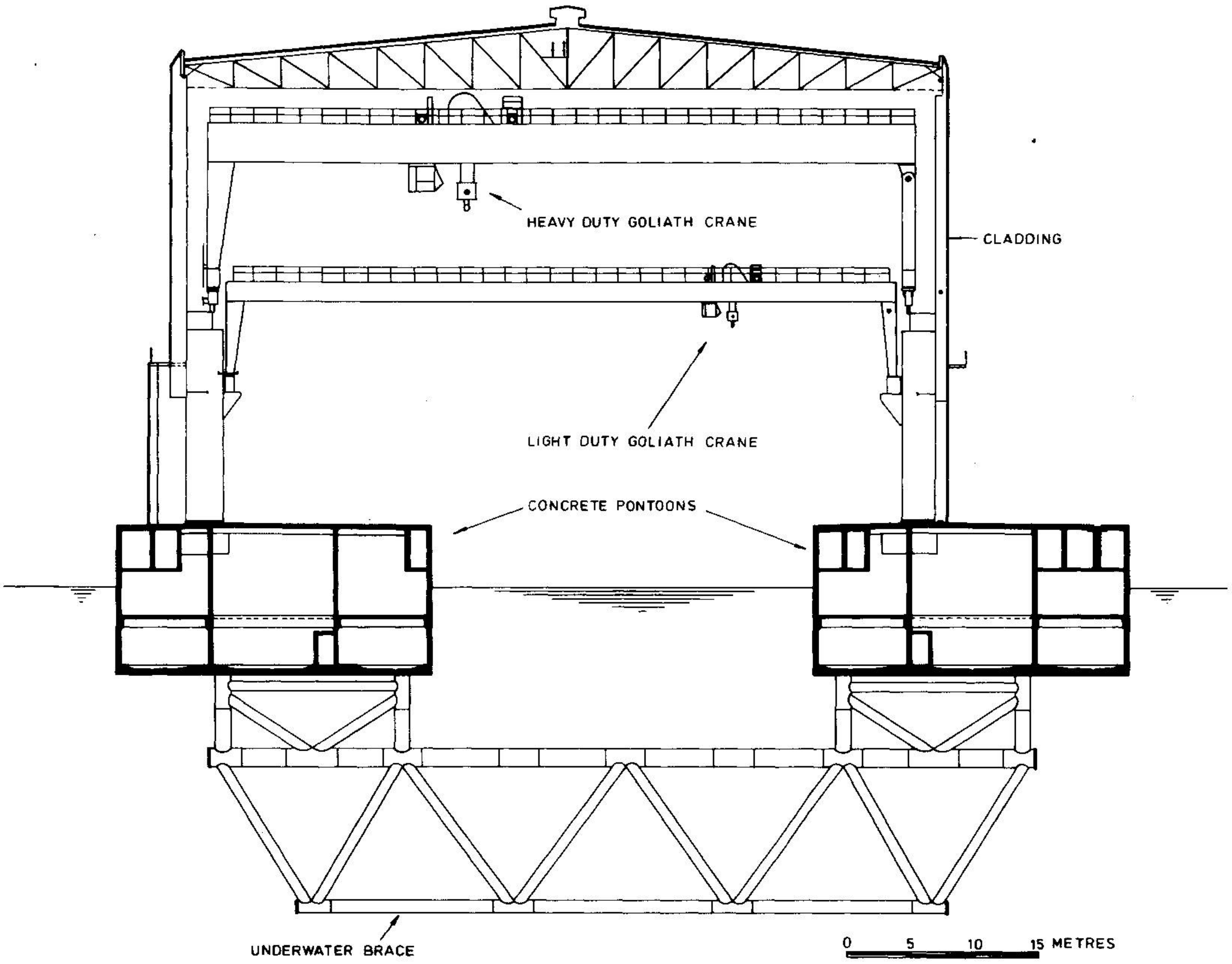


FIGURE 2 - CROSS - SECTION

2. STRUCTURAL FORM

2.1 The general layout of the jetty is shown in Fig. 1 and a typical cross-section in Fig. 2. The two longitudinal bulkheads in each pontoon are placed symmetrically to coincide with the eccentric enclosure column line on each pontoon. Spacing of the outer transverse bulkheads is limited by collision damage considerations, and is generally 30m. An intermediate deck is provided to support the outer walls against hydrostatic pressure, with internal transverse bulkheads at 15m centres, which both transfer the hydrostatic loads and add support to the bottom slab.

2.2 The hull girder bending moment, shear and torsion strength requirements strongly indicated that the pontoons should be longitudinally prestressed. A number of options were also considered for transverse and vertical prestressing, but for a number of reasons it was decided to use only unstressed reinforcement transversely. Local shear stresses were the major parameter in determining wall and slab thickness and haunching for the basic rectangular hull section.

3. MODEL TESTING

3.1 To prove the operational acceptability of the jetty, model tests were commissioned at the laboratories of Hydraulics Research Ltd. Initial tests identified the motion characteristics of the structure, in operational, and extreme wind, wave and current environments. Later tests were undertaken to assess accurately the absolute and relative motions of the prototype jetty and vessel moored within the enclosed berth. The test tank and model jetty are shown in Plate 1.

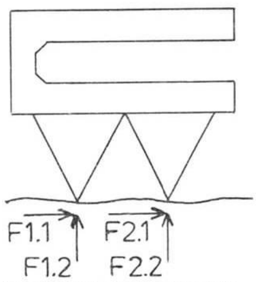
3.2 Surge, sway and yaw of the structure were measured using metal probes, fixed to the floating jetty, positioned between electrodes in fluid filled pots supported on a rigid platform within the enclosure. Heave, pitch and roll motions were obtained by $\pm$ summation of the outputs from twin wire resistance probes fixed to the jetty. Axial boom forces were measured by strain gauged proof rings. Typical maximum motion and force figures for an extreme 1 : 100 year return wind case are shown in Table 1.

Table 1

100 year return conditions - typical maximum force and motion figures

	Tonnes Max	Tonnes Min
F 1.1	621	507
F 1.2	565	445
F 2.1	759	573
F 2.2	562	432

		Max	Min
Surge,	m	0.28	0.19
Sway,	m	0.08	0.16
Heave,	m,	0.05	0.04
Yaw,	degrees	0.13	0.11
Roll,	degrees	0.11	0.05
Pitch,	degrees	0.14	0.11



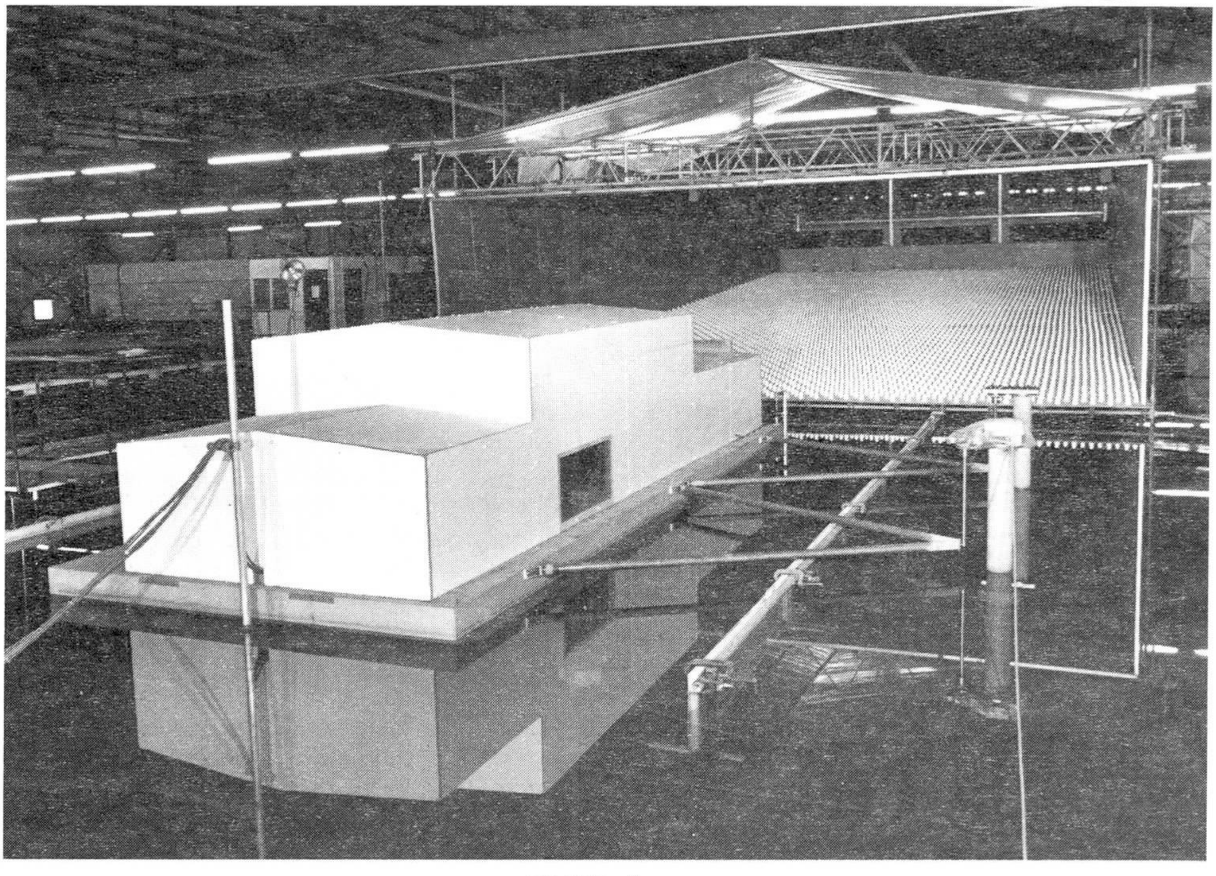


PLATE 1

4. DESIGN PROCEDURES

4.1 Preliminary design of the jetty was carried out using a simplified global model consisting of a detailed tubular brace structure with the two pontoons and the south link represented by line elements. From this it was possible to obtain stresses and deflections for the main pontoons and the brace. Although grillage analysis has been shown to be capable of giving accurate results for this type of structure, the finite element membrane method was chosen for the detailed analysis because of its flexibility. In particular, a detailed model of the closed end could be used with a relatively coarse model of the straight pontoon sections. Support conditions were modelled by using springs and restrained freedoms to give the correct mooring boom articulation and by distributed vertical springs representing buoyancy stiffness. Static loading was applied to simulate wave loads, wind loads, dead load, ballast tanks, prestressing, damage cases and collision forces as well as 8 different combinations of live loads giving maximum hogging, sagging, torsion and racking at key sections.

4.2 The structure is aseismically designed and can also be subjected to accidental collision forces. It is also necessary to ensure that certain sensitive plant items such as the crane load pendulum, the vessel access door and cantilevered service booms are not dynamically coupled to the pontoon motions.

4.3 After the finite element membrane analysis of the global loading had been first run, an approximate hand analysis was made to check on the magnitude and disposition of the prestress. Adjustments were made to achieve the most economic and effective use of the prestress and then the final part of the analysis was rerun. Post processor plotting of the envelopes of the membrane tensions, compressions and shears due to global loading then followed.

4.4 The cross-section including the service passageway walls and floors and

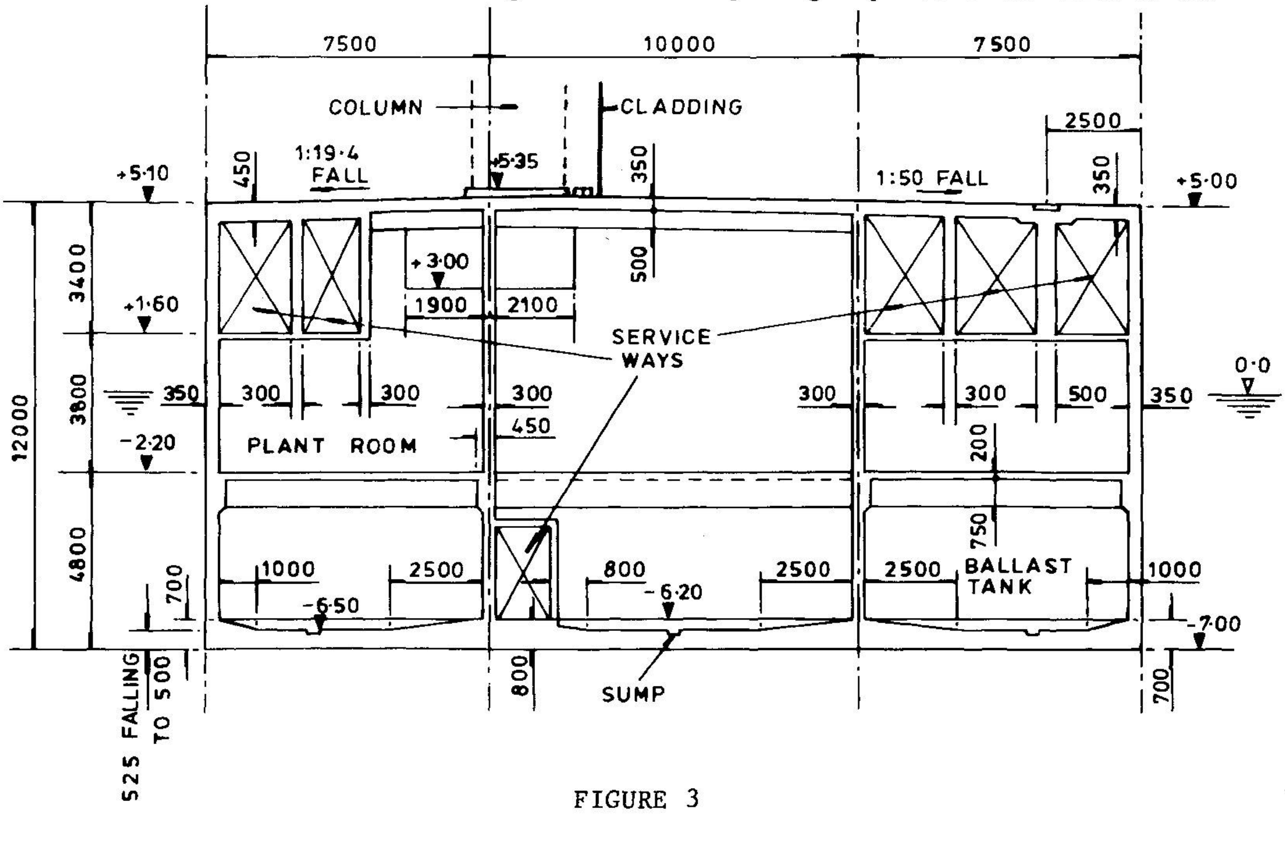


FIGURE 3

intermediate deck is illustrated in Fig 3. About 70 A1 size drawings have been produced to show the concrete outline and prestressing arrangements by computer aided draughting

4.5 The individual panels and loaded areas of the structure were analysed by conventional hand methods for the various local load conditions such as water pressure and plant loads. These were then combined with the stresses obtained from the global analysis, already adjusted to combine the membrane shears into direct membrane stresses. The areas of high external load such as mooring boom connection points, brace connections, bridge supports, and column bases were analysed by hand. Concrete member thickness has been minimised but is sufficient to ensure that the use of shear reinforcement in slabs and walls is not required. The deck is designed to carry heavy vehicles, cranes and stacked loads.

5. SERVICEABILITY

5.1 It is anticipated that the draft of the structure will be slightly lower than that required for operational requirements. Some of the cells in the lower part of the structure have been designated for water ballast and provided with a ballast water pumping system. It will be possible to trim the jetty fore and aft and athwartships.

5.2 Great attention has been paid to durability of the structure for minimum maintenance. The concrete specification is aimed at producing high strength impermeable concrete of 50 N/sq.mm characteristic strength. The minimum cement content will be 400 Kg/cu.m and at least 30% pulverised fuel ash or 50% blast furnace slag will be incorporated. Cover to reinforcement in the splash zone and exposed deck is 75mm, 50mm for submerged surfaces, and 30mm for internal surfaces except the ballast tanks which are 50mm.

5.3 Although a bilge pumping system will be installed it is expected that the structure will be virtually watertight. Construction joints will be carefully detailed with waterstops, and the longitudinal prestressing will assist in maintaining watertightness.

6. CONSTRUCTION

6.1 At the mooring site there is no land available or suitable for a construction yard, and therefore the jetty will be built elsewhere and towed to the Site.

6.2 The jetty is in effect a vessel without propulsion and construction is foreseen as being very similar to ship construction. Fitting out the jetty with plant, services, bilge and ballast systems, accommodation units and steel superstructure should be completed within the construction yard.

6.3 After the tow, mooring of the jetty in the permanent position will be a routine operation. The Contractor will have positioned the mooring booms on shore connections, with the outer ends supported on pontoons. Once the jetty is connected to the booms, the bridges can be placed and commissioning of the on-board plant will commence.

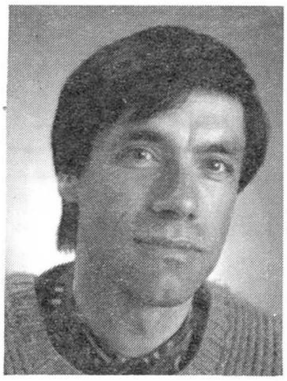
ACKNOWLEDGEMENTS

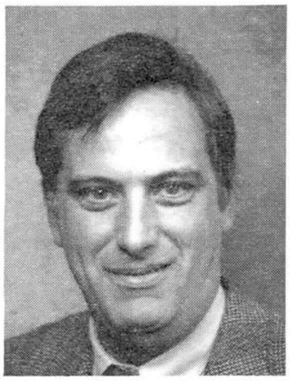
The thanks of the Authors are due to Mr. R.F.Hughes, Director of Civil Engineering Services and Mr. S.G.D.Duguid, Director of Works (Special Services) at the Property Services Agency - Department of the Environment, and to the Ministry of Defence (Navy) for permission to present this Paper.

Aéroréfrigérants de la centrale nucléaire de Golfech

Luftkühltürme des Kernkraftwerkes Golfech

Natural Draught Cooling Towers of Golfech Nuclear Power Plant





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RÉSUMÉ
L'article traite des réfrigérants atmosphériques à tirage naturel équipant la centrale nucléaire de Golfech. Outre leur hauteur qui constitue le record du monde actuel, ces ouvrages présentent la particularité d'être fondés sur des piles situées dans des plans verticaux radiaux. L'article donne un aperçu des problèmes d'études rencontrés ainsi que des innovations de mise en œuvre.

ZUSAMMENFASSUNG
Die Luftkühltürme mit Naturzug des Kernkraftwerkes Golfech werden vorgestellt. Neben der Höhe (zur Zeit die höchsten Kühltürme der Welt) haben sie die Besonderheit, auf Stützen zu ruhen, welche in radialen Vertikalebenen angeordnet sind. Der Beitrag gibt einen Überblick über die angetroffenen Probleme und Innovationen bei der Bauausführung.

SUMMARY
Natural draught cooling towers of Golfech power plant. These structures are outstanding because of their height, to date a world record, and also because they are supported by piers located in radial vertical planes. Some problems of the project are described, along with some constructional innovations.

1. INTRODUCTION

Les réfrigérants atmosphériques qui équipent les Centrales d'EDF comportent tous, à l'exception de ceux de CHINON qui sont à tirage induit, une coque en béton de grande hauteur dont la fonction est d'assurer le tirage par effet de cheminée. Pour s'adapter à l'accroissement des puissances unitaires des groupes turbo-alternateurs des centrales thermiques classiques puis nucléaires, les surfaces d'échange et le tirage, donc le diamètre et la hauteur des aéroréfrigérants, ont dû fortement augmenter :

Tableau n° 1

Evolution depuis une trentaine d'années :

SITE	Puissance Centrale	Diamètre à la base	Hauteur
Pont/Sambre 1	125 MW	67 m	93 m
Pont/Sambre 2 et 3	250 MW	82 m	110 m
Dampierre	900 MW	131 m	163 m
Golfech	130 MW	146 m	178,5 m

Un sujet de réflexïon qui a récemment passionné les concepteurs d'aéroréfrigérants est le supportage de la coque. Pour l'homme de structure, le réfrigérant idéal est constitué d'une coque continue sans ouverture encastrée dans le sol.

Pour le thermicien, la meilleure coque est, au contraire, suspendue à bonne hauteur, au-dessus du sol. Entre ces exigences antinomiques le supportage par colonnes inclinées dites diagonales (en X ou en W) satisfait au mieux l'ingénieur de génie civil.

Mais les entrées d'air ne sont pas suffisamment dégagées, au détriment du rendement global de l'appareil.

C'est pourquoi EDF a préconisé un nouveau mode de supportage par piles situées dans des plans verticaux radiaux, procédé par ailleurs satisfaisant du point de vue du coût et de la facilité de réalisation.

Ainsi, on trouvera dans le tableau suivant l'évolution des caractéristiques géométriques des aéroréfrigérants du parc nucléaire EDF.

Tableau n° 2

CENTRALE	Tranche	Hauteur Totale (m)	Diamètre			Epaisseur du voile (m)		Supportage
			Sol	Col	Couron[t]	Linteau	Col	
BUGEY	4 - 5	127	102	61	68	0,76	0,18	W
DAMPIERRE	1 à 4	165	131	77	83	1,08	0,21	W
ST.LAURENT B	1 - 2	125	175 (1)	84	88	0,97	0,20	X
CRUAS	1 à 4	155	132	78	81	1,10	0,21	W
CATTENON	1 à 4	165	205 (1)	84	88	1,05	0,21	X
BELLEVILLE	1 - 2	165	147	83	84	1,16	0,23	W
NOGENT	1 - 2	165	147	83	84	1,07	0,25	W
CHOOZ	1	172	153	85	88	1,55	0,25	piles
GOLFECH	1	178,5	146	83	83	1,20	0,20	piles

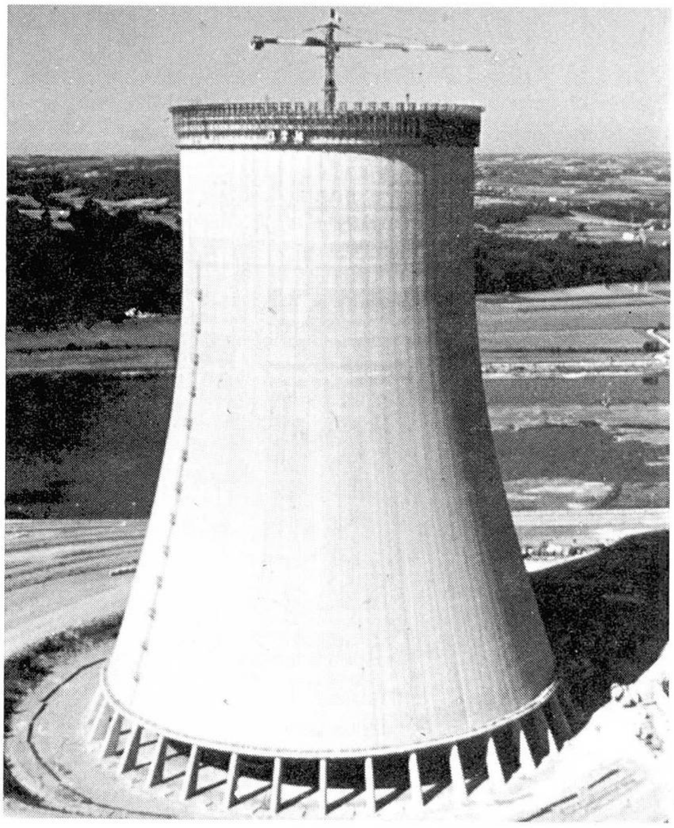

Fig. 1 : Vue générale

Ainsi les réfrigérants de GOLFECH représentent la dernière évolution technique préconisée par EDF :

– Tirage naturel, contre-courant, supportage sur piles ; et de façon anecdotique puisque ne concernant pas les structures, dispositif de recueil de l'eau refroidie. En outre, leur hauteur totale hors sol exceptionnelle de 178,5 m (le record du monde actuel) permet d'augmenter le tirage, donc de diminuer la section à la base et ainsi d'optimiser le coût global de l'ouvrage (coque et équipements).

2. LES AEROREFRIGERANTS DE GOLFECH

2.1. Géométrie de l'ouvrage

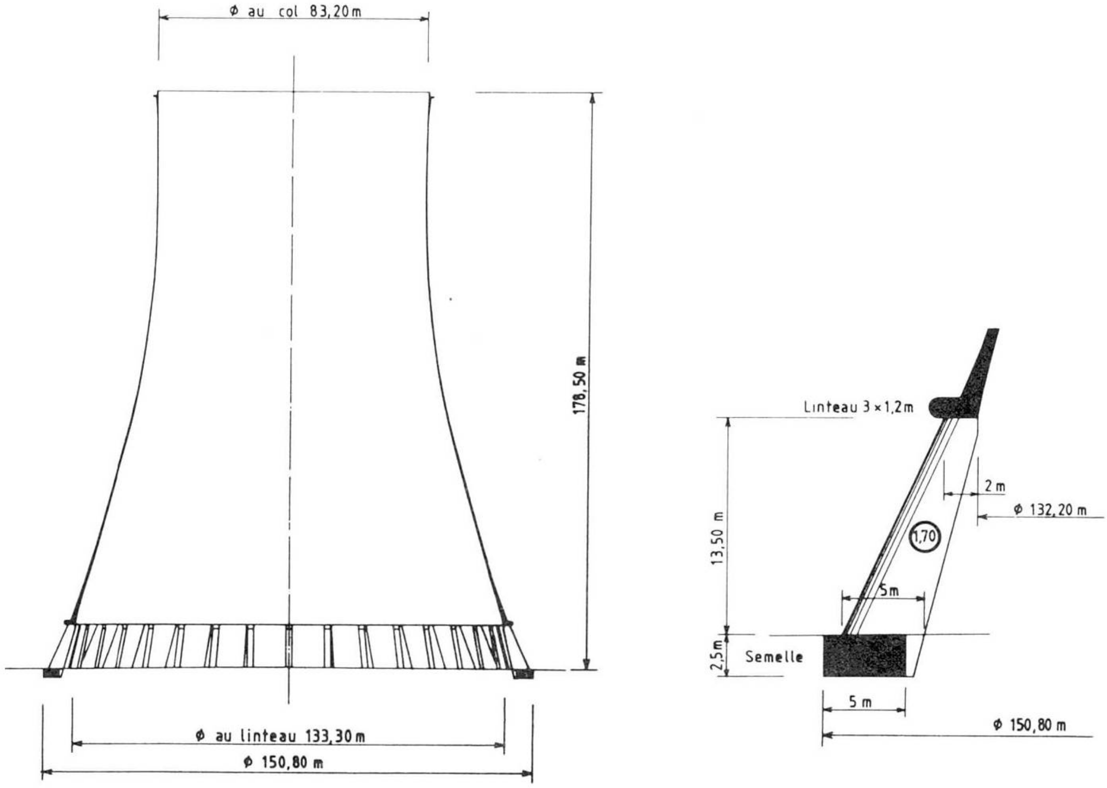

Fig. 2 : Géométrie de l'ouvrage

2.1.1. fondations

Semelle polygonale (5,00 m x 2,50 m hauteur) dont la ligne moyenne en plan est inscrite dans un cercle de 145,8 m de diamètre. En phase définitive, cette semelle est précontrainte par 24 câbles SEEE, FUC 12-600.

2.1.2. les piles

Sur cette semelle s'appuient les piles situées dans des plans verticaux radiaux. Ces piles ont une hauteur libre de 13,50 m.

Dimensions en plan des piles :

à leur base : 5,00 radial x 1,70
à leur sommet : 2,00 x 1,70

2.1.3. linteau

Ces piles sont liées à la coque par l'intermédiaire d'un linteau (section transversale : 3,00 x 1,20 hauteur).

2.1.4. la coque

Diamètre au linteau : 133,30 m
Diamètre au col : 83,20 m
Altitude du col : 146 m
Son épaisseur varie de 1,20 m à la base à 0,20 m au col.
La méridienne est constituée de 2 coniques (hyperbole sous le col, ellipse au dessus du col).

2.2. conception de la coque

Les calculs ont été effectués en appliquant les spécifications du CST d'EDF édition Mars 83.
Les principaux cas de charges élémentaires à prendre en compte sont :

- Le poids propre
- La précontrainte
- Le vent
- Les variations dimensionnelles : température, retrait
- Les gradients thermiques (de fonctionnement, d'ensoleillement)
- Les tassements d'appuis.

Pour tous ces cas de charges, sauf les tassements d'appuis, le sol est pris en compte comme un matériau élastique; 3 valeurs du module de réaction du sol ont été retenues : modules permanent, semi-rapide, dynamique.

Outre les vérifications classiques d'intégrité des matériaux, béton en section homogène et fissurée, et aciers pour de nombreuses combinaisons de cas de charges élémentaires, nous devions vérifier :

- La stabilité de forme de la coque par la méthode de Bochum sous le cas de charges : poids propre + tassements + vent.
- La sécurité au flambement Eulérien sous le cas de charges : poids propre + vent
- La période propre fondamentale de la structure inférieure à 1,5 sec.

Les différentes phases de la conception ont été les suivantes :
- 1ère phase :

  Prédimensionnement de la coque elle-même, c'est-à-dire de toute la partie de la structure située au-dessus du linteau.
  Cette phase a consisté à optimiser la méridienne, dans la fourchette de géométrie autorisée, de façon à mettre en oeuvre un minimum de béton de coque. Cette optimisation a été faite à partir du critère de stabilité de forme de Bochum qui est déterminant dans le choix de

l'épaisseur de la coque à tous les niveaux.

Dans cette phase le calcul des efforts a été fait à l'aide d'un programe SEEE de calcul aux éléments finis d'une coque de révolution.

- 2ème phase

Dimensionnement définitif de la coque et de ses appuis. Dans cette phase le calcul des efforts a été fait avec le programme CASTEM, logiciel de calcul de structures aux éléments finis développée par le CEA et par la Société Informatique Internationale. Les passages du programme ont été faits sur l'ordinateur CRAY de la CISI.

Le modèle comportait 2000 nœuds environ et intégrait le comportement élastique du sol de fondation, sous la forme de ressorts simples verticaux et horizontaux de compression traction en sous-face de la semelle et de ressorts hélicoïdaux de moment d'axe parallèle à la fibre moyenne de la semelle (correspondant à de la torsion dans la semelle).

En outre, nous disposions d'un second modèle permettant de prendre en compte les tassements différentiels prévisibles propres au site, obtenus en faisant varier de façon sinusoïdale les raideurs des ressorts de sol le long de la semelle.

Dans cette phase, le problème déterminant auquel nous nous sommes heurtés fut le soulèvement local de la semelle de fondation sous la combinaison :

0,9 x poids propre + 1,5 + vent + variations dimensionnelles.

En effet, EDF nous imposait le non-soulèvement en tout point de la semelle sous l'action de cette combinaison sans autoriser la moindre atténuation de la raideur d'encastrement de la semelle de fondation dans le terrain, en particulier sous poids propre.

Les puissants moyens informatiques mis en œuvre nous ont permis de paramétrer les appuis (inclinaison des poteaux, excentrement de la semelle) de façon à limiter au mieux cette décompression.

Dans la structure ainsi obtenue (qui est la structure retenue décrite ci-dessus), le reliquat de soulèvement a été supprimé en créant un état d'autocontrainte du sol de fondation par vérinage d'un effort normal dans chaque étai de poteau avant coulage du linteau de façon telle que :

- D'une part le soulèvement s'annule sous la combinaison :

  0,9 (G) + 1,5 (V) + variations dimensionnelles,

- d'autre part les contraintes sur le sol dues au poids propre seul, calculées avec encastrement dans le terrain et en tenant compte des phases de mise en œuvre (y compris le vérinage décrit ci-dessus), soient pratiquement constantes dans le plan radial. Nous nous affranchissions ainsi du même coup, théoriquement au moins, du problème du fluage éventuel de l'encastrement de la fondation dans le terrain sous poids propre.

- 3ème phase :

Exploitation de détail des résultats CASTEM; compte tenu du nombre impressionnant de combinaisons d'efforts à envisager, la conception du ferraillage et les vérifications de contraintes furent particulièrement facilitées par des logiciels que nous avons créés, de post-traitement des résultats des cas de charges élémentaires fournis par CASTEM dans les 2 modèles (qualité de fondation homogène et qualité variant de façon sinusoïdale le long de la semelle).

Ces logiciels réalisaient les combinaisons de cas de charges et leurs enveloppes suivant des critères choisis (tels les maxima de contraintes) tant dans les éléments de poutres que dans les éléments de coque.

2.3. Mise en œuvre

2.3.1. Pompage du béton de la coque

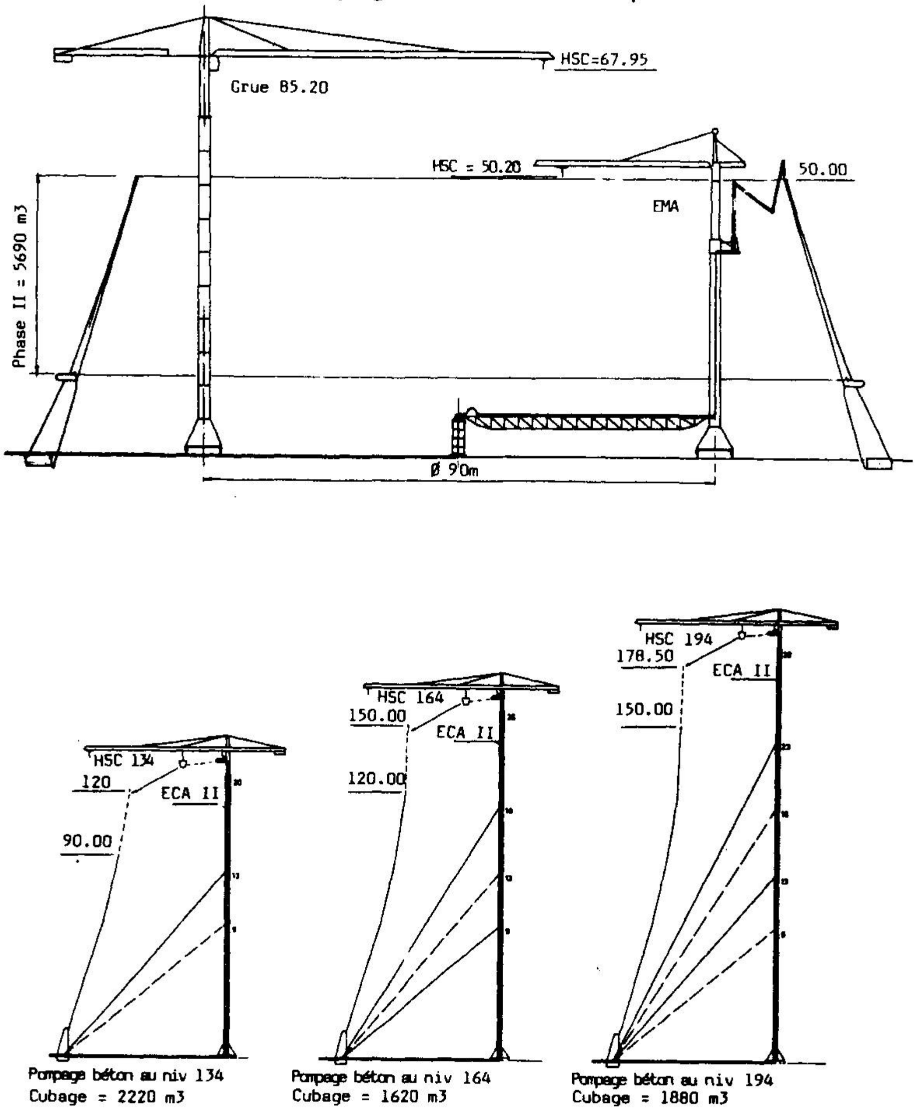

Fig. 3 : Mise en œuvre

Pour l'ensemble de l'ouvrage la solution du pompage du béton a été retenue dans deux configurations principales suivant l'avancement de la coque. Dans les deux cas, la centrale à béton, installée immédiatement à l'extérieur de l'ouvrage, déverse son béton directement dans une pompe haute pression. Le béton est ensuite refoulé au centre du réfrigérant dans une conduite métallique rigide.

- de - 2,50 à + 60 m, le béton est ensuite distribué directement dans les coffrages par un mât de bétonnage monté sur grue à tour associée à une poutre tournante.
- de 60 à 178,5 m, le béton remonte le long du fût de la grue, traverse le pivot de celle-ci et débouche sous la cabine du grutier. Repris à la benne, il est ensuite distribué dans les coffrages par simple translation radiale de la benne.

Un automatisme (breveté) permet la synchronisation de la pompe et de la grue ainsi que l'optimisation du remplissage de la benne en fonction de la portée à atteindre.

2.3.2. Implantation de la coque

Le respect de la géométrie théorique de la coque est essentiel pour son fonctionnement mécanique. Il faut pour cela un coffrage rigide et une technique d'implantation de grande précision. L'équipe de topographes exécute elle-même le réglage du coffrage. Elle doit donner chaque jour 144 points avant bétonnage, puis procéder aux nombreuses vérifications des résultats obtenus sur les levées précédentes. Elle est équipée d'un théodolite électronique et d'un distancemètre couplés à un ordinateur permettant de travailler en temps réel. Alors que la tolérance sur le rayon varie de 5 à 15 cm, 96% des points contrôlés se situent dans la fourchette −2 cm à 3 cm.

Chantier de haute technologie, la construction d'un réfrigérant atmosphérique est l'occasion de développer et de mettre au point les nouvelles techniques de conception et de réalisation qui tendront à se généraliser dans l'avenir. L'importante instrumentation mise en place sur ces ouvrages permettra d'affiner la connaissance du comportement de ces structures. L'utilisation systématique d'outils performants conjuguée à la mise en place d'un plan d'assurance qualité assure au(x) maître(s) d'ouvrage un haut niveau de qualité, et répond en cela à une attente croissante.

Applications of the Concrete Origami Concept in Structures

Application du concept d'origami dans des structures en béton

Anwendung des Origami-Konzeptes auf Betonbauten

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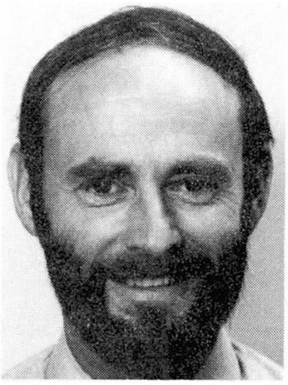

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SUMMARY
The "concrete origami" concept recognises in concrete slabs the characteristics of paper. Paper sheets can be curved and folded from flat surfaces into three-dimensional shapes. This paper demonstrates how these ideas can be turned into concrete realities and suggests practical applications in concrete structures.

RÉSUMÉ
L'origami est un ancien art japonais de plier le papier. La notion d'origami appliquée au béton reconnaît les propriétés du papier dans des dalles en béton. Des feuilles de papier peuvent être courbées et pliées en structures à trois dimensions. L'article montre comment ces idées peuvent se réaliser sur des structures en béton. Des applications pratiques sont également indiquées.

ZUSAMMENFASSUNG
Origami ist die alte japanische Kunst des Papierfaltens. Der Origami-Begriff auf Beton angewendet erkennt in Betonplatten die Eigenschaften von Papier. Papierblätter können zu dreidimensionalen Strukturen gewölbt und gefaltet werden. Dieser Beitrag zeigt wie diese Vorstellungen für Betonstrukturen Wirklichkeit werden können. Auf praktische Anwendungsmöglichkeiten wird hingewiesen.

1. INTRODUCTION

Origami is the ancient Japanese art of paper folding. The "concrete origami" concept recognises in concrete slabs the characteristics of paper. Paper sheets can be curved and folded from flat surfaces into three-dimensional shapes.

On an intellectual level all structural engineers will acknowledge the flexibility of concrete slabs. They are used to thinking of concrete slabs returning to their original shape once the loading is removed. Yet most will have witnessed, as students, the remarkable ductility of an under-reinforced concrete beam or slab in laboratory experiments carried out with loading beyond normal working loads.

Such loadings are normally thought of as "aggressive" tending to destroy the fabric of the material. Alternatively this behaviour can be seen as an opportunity to create curved concrete structures by a new method which offers dramatic cost savings.

This paper takes the folding and bending notions associated with origami and shows how they can be achieved in concrete. Many and varied are the possible applications opened up by this new approach.

2. THE CONCRETE ORIGAMI CONCEPT

2.1 General

The concrete origami concept extends the usual range of concrete fabrication methods very considerably. Designers may now contemplate complex shapes with folded and curved surfaces without the problems of complex and intricate formwork. Simple flat formwork is the key.

The advantages of concrete origami are:

(i) economical casting of horizontal slabs
(ii) ease of finishing
(iii) a great variety of surface textures and finishes are possible at low cost
(iv) a high quality concrete because of the ease of casting, compaction and curing
(v) the speed of construction

2.2 Folded Concrete Slabs

The term "folded-plate" is in common usage to describe what appears to have been done in creating such structures. A genuine concrete folded-plate structure is shown in Fig. 1. It was created from a single flat casting. The flat segments were cast horizontally with open joint lines between them. The mid-plane reinforcing mesh crossed the open joints. About seven days after casting the folding was carried out. The yielded reinforcement forms the hinge along each joint which permits the folding to take place. The joint lines were then filled with mortar to hold the joint rigid.

Using this technique any developable three-dimensional shape is theoretically possible.

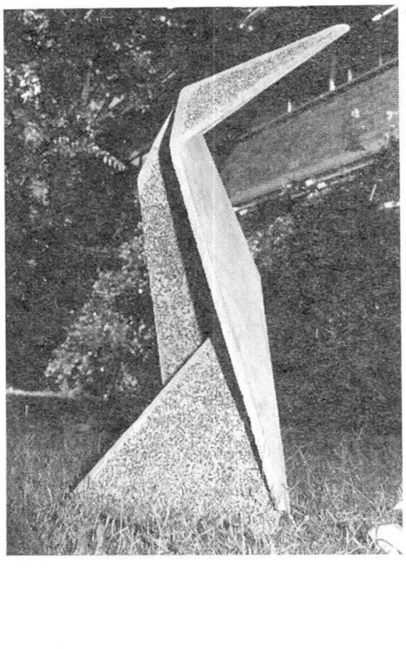

Fig. 1 A folded-plate structure

2.3 Curved Concrete Slabs

A thin concrete slab initially cast flat could easily be lowered over curved templates or lifted at its edges to hang freely in a curved shape.

If, for example, a 30mm thick slab which is under-reinforced, has an effective depth to steel reinforcement of 25 mm it can be bent into an arc of 1m radius. As illustrated in Fig. 2 it will be extensively cracked on the tension face but the compressive surface will not show any distress.

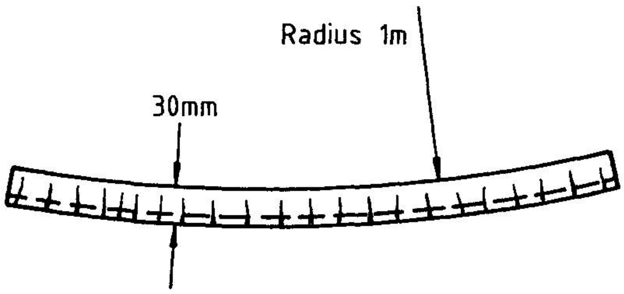

Fig. 2 A curved concrete slab

This cracking would be of no consequence if the curved slab were then used as permanent formwork for further concreting. Alternatively, treatment of the cracked surface to render it waterproof may be all that is required, using materials such as epoxy resins.

By inducing prestress in the slab it is possible to bend the slab without causing cracking at all but the radius of curvature is much larger. A 30mm thick slab, for example, with a uniform compressive stress of 15 MPa induced by prestress can be curved to a radius of about 30mm (assuming an elastic modulus of 30 MPa for the concrete).

Any radius of curvature between these two limits is possible depending on the extent of cracking regarded as acceptable. Concrete creep will, of course, tend to dissipate stresses induced by bending.

3. PRESTRESSED CONCRETE TENSION MEMBERS

The notion of using concrete in tension members or membranes seems to be a paradox. Everyone knows that concrete is weak in tension. When that notion is combined with the ideas embodied in the "concrete origami concept" of folding or bending hardened concrete some interesting and valuable structural possibilities emerge.

Surprisingly little work has been published on the behaviour of prestressed concrete tension members and yet there are many examples of their use in practice. [6] [7]

Without doubt tension members are the simplest of structural elements which demonstrate the principles of prestressing. By prestressing it is possible to make dissimilar materials work together in many ways which have advantages over the behaviour of either.

High tensile steel is by far the most economical and efficient material for carrying tension forces. It is not, however, available in sheets or plates or rolled sections like mild steel and it cannot be joined by welding. Its very high strength is achieved in the form of wires by drawing, yet the elastic modulus remains virtually that of mild steel.

While the cross-sectional area of high tensile steel required to carry a given tension will be about one fifth that of an equivalent mild steel the elongation under load will be five times greater.

Concrete, on the other hand, has an elastic modulus around one eighth that of steel. It has a reasonable compressive strength but low tensile strength. It is a cheap, widely available material. It is dense, it can protect steel from corrosion, and it may be cast into many shapes. Even though it is cheap its cost is very much dependent on the quality and shape required since labour and formwork costs make up a very large proportion of its total cost. In combination high tensile steel and concrete can be used to create tension members with desired ultimate strength and load-elongation behaviour characteristics.

Provided a residual compressive stress remains in a prestressed concrete tension member then it is possible to bend that member as indicated in the previous section.

4. FOLDED STRUCTURES

Many applications come to mind of structures which may be cast flat and then "folded into final shape. One possibility is described.

Fig. 3(b) illustrates the finished cross-section of a box culvert. Fig.3(a) indicates the casting of the base slab and the side walls on the prepared base. The middle wall is then cast on top of the base slab. The next step involves lifting and rotating the walls into their final position. The areas shown shaded in Fig. 3(b) are concreted as the final step in making a culvert.

Many commonly used shapes are developable in this way. Among them are barges and pontoons and the structures used in sewage and water treatment plants. Formwork costs are minimal while the additonal lifting operation need not be onerous.

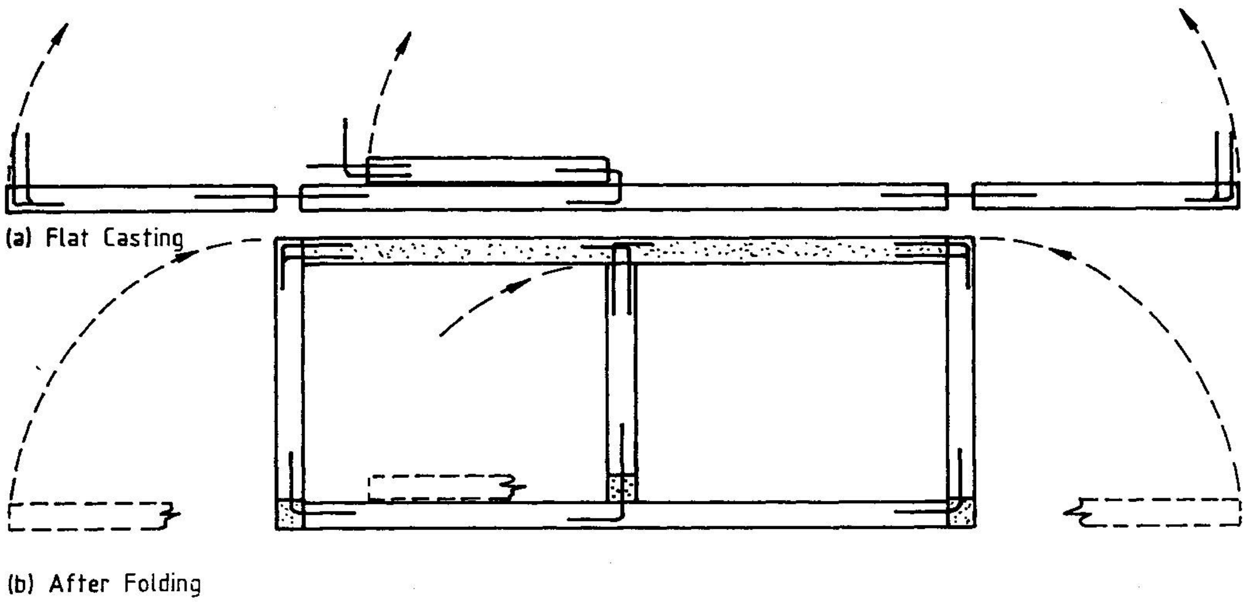

Fig. 3 Box culvert walls "folded" into place

5. CURVED STRUCTURES

Among the many possible curved structures two examples are briefly described. Fig. 4 shows a dome made up of reinforced concrete segments which were cast horizontally on flat formwork. Adjoining segments are joined by concreting in-situ around overlapping reinforcement which projects from both segments. The segments may be lowered onto curved templates and held in the curved shape until effectively joined.

The same principle could be applied to create a barrel vault as suggested in Fig. 5. A semi-circular barrel 10m in diameter could be constructed from 15m long by 2m wide by 30mm thick flat concrete slabs. Each such slab element only weighs about 2 tonnes and is therefore easily lifted into position.

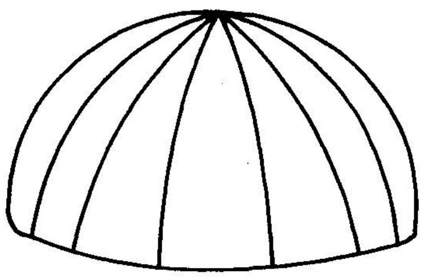

Fig. 4 Segmented Dome

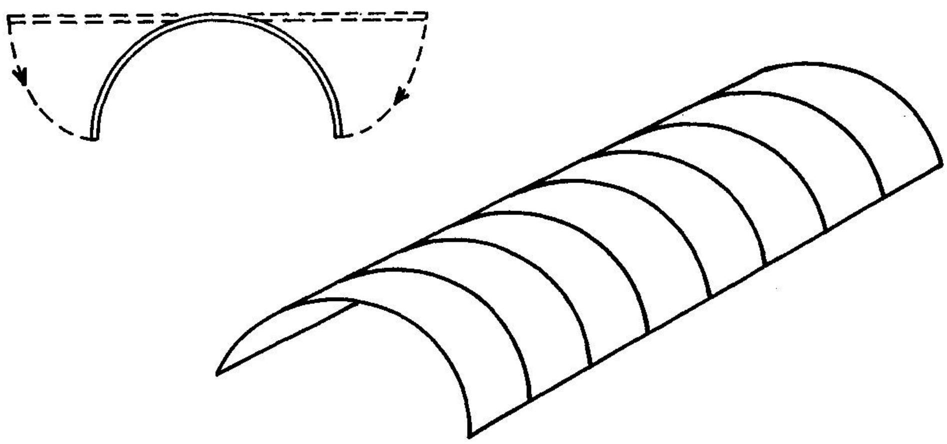

Fig. 5 Barrel vault roof from flat segments

6. TENT-LIKE STRUCTURES

The possibilities for long-span prestressed concrete tension membrane roofs which follow on are based on an extremely economical construction method. The method envisages the lifting of a flat thin prestressed slab under tension allowing it to sag under its own weight yet retaining its structural integrity. Long span tent-like structures can be created using this method which combines already well-proven technologies.

The original description of the concrete origami concept [1] envisaged the casting and stressing long relatively thin prestressed concrete slabs on flat formwork. Provided that a substantial anchoring force is maintained along opposite edges if the slab it was considered possible to either lower the formwork from the slab or alternatively to lift the slab from the formwork allowing it to hang freely in a catenary shape.

Experiments carried out have clearly demonstrated the feasibility of such lifts. This was done by casting a 12m long slab with a 150mm x 50mm cross-section. The slab was pretensioned with 5mm diameter high tensile steel wires. A lifting yoke was located at the midpoint which bore directly on the prestressing wires in a short region which was not concreted. After the concrete hardens sufficient prestress is transferred into the concrete to prevent subsequent cracking.

In a succession of prestress release and lift operations the slab was lifted clear of the formwork to give two symmetrically balanced catenaries.

The experiments were designed to test features of the tension membrane roof idea which were deemed critical to the whole concept. Once demonstrated further development of the idea was undertaken.

The lifting procedure is illustrated diagrammatically in Fig. 6. The experimental work referred to above confirmed that there is no untoward behaviour as the slab is progressively lifted from the formwork along its length.

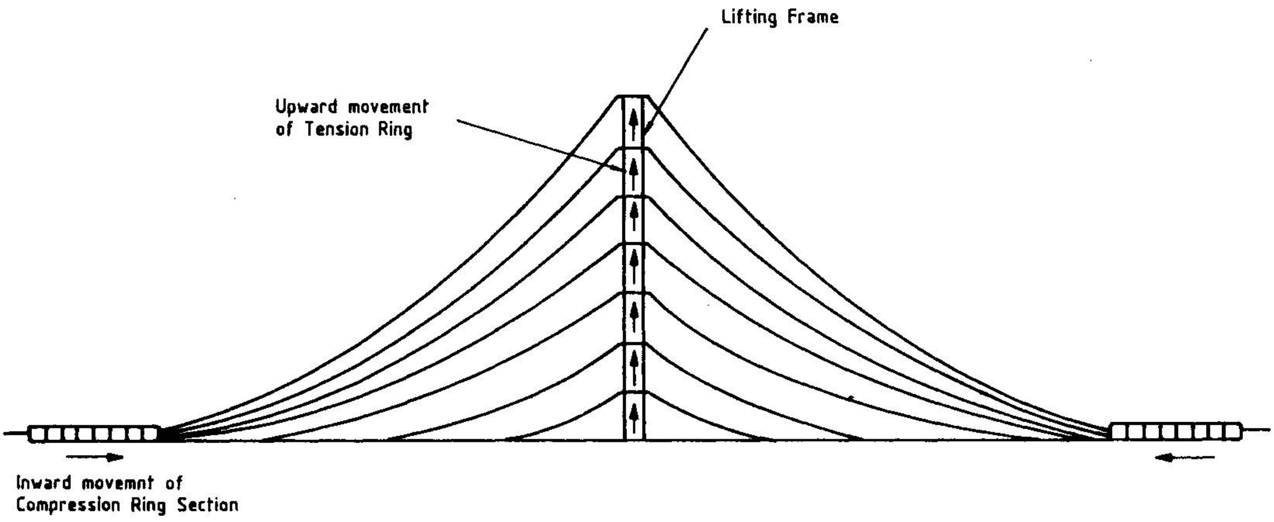

Fig. 6 Progressive lifting of prestressed concrete "tent"

Two forms of roof structure have been considered. The first, illustrated in Fig. 7 is made from pretensioned sector elements cast on the ground. Once cured and after transfer of some of the prestress the elements can be lifted on a central column.

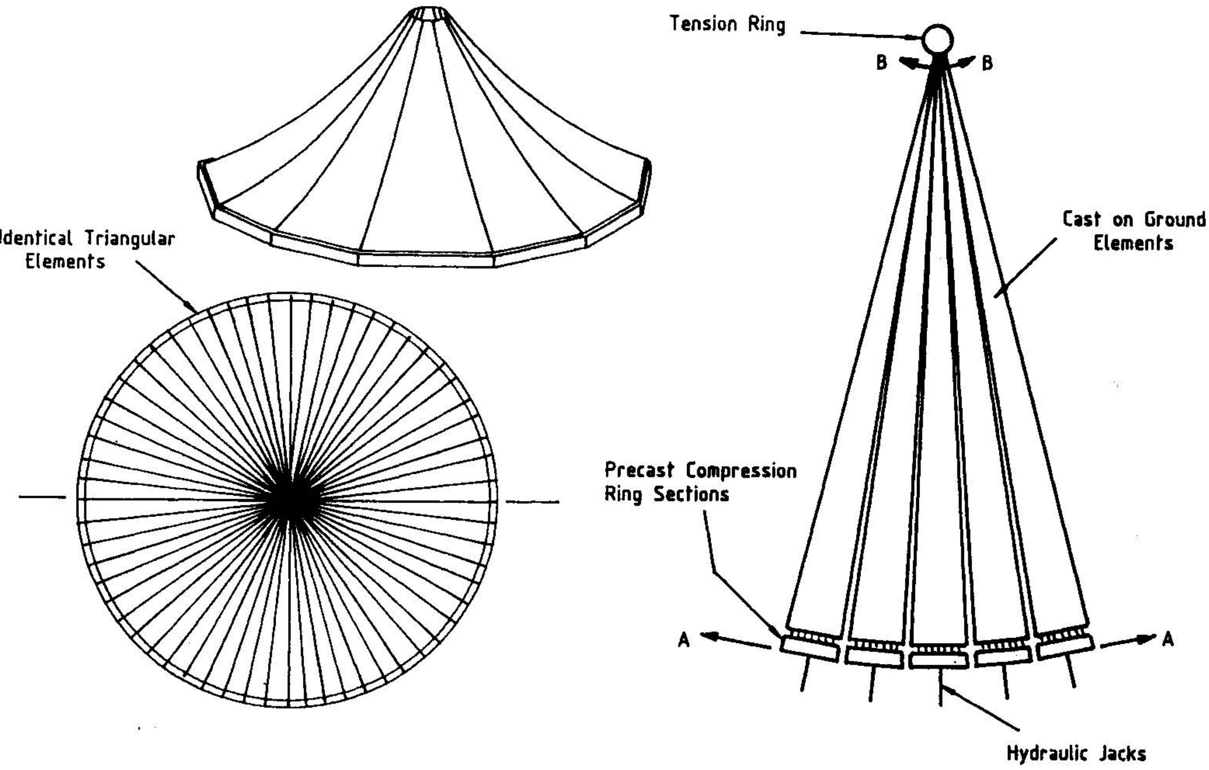

Fig. 7 Conical roof

It is not possible to cast and lift all elements simultaneously because the sector geometry causes edge overlaps. A simple solution is to cast, stress and lift two sets of elements each set consisting of every alternate element.

Joining the edges of the elements is done once all of them are in place. Care must be taken with these joints since they are potentially vulnerable to leakage in heavy rain. One simple solution is the casting in-situ of a capping strip which would bridge the joint between the elements. This capping strip is easily and economically cast since the formwork required is very simple. Advantage could be taken of these strips to make them serve the function of stiffening ribs if required.

The second form of roof structure is illustrated in Fig. 8. Again progressive lifting from the flat formwork along the ridge line of the "tent" is intended.

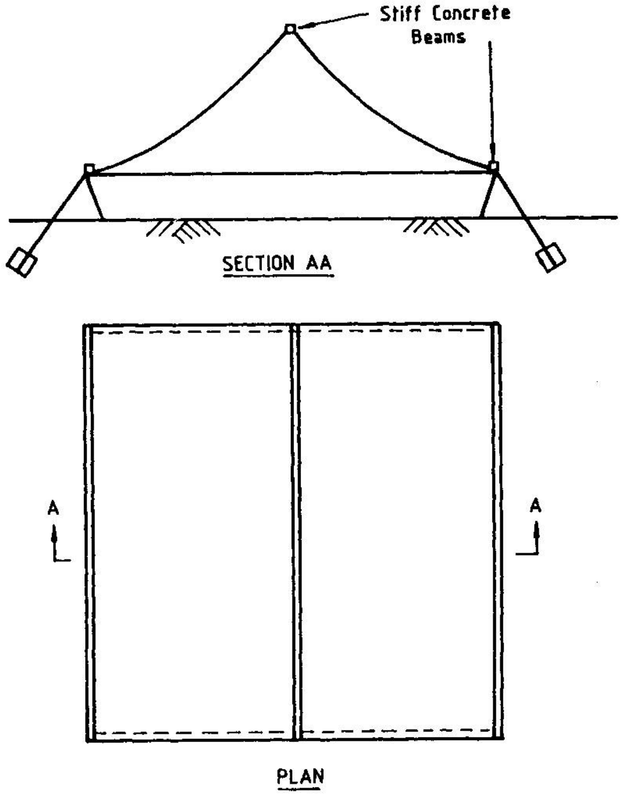

Fig. 8 The prismatic tent structure

A substantial anchoring capacity at the "tent pegs" must, of course, be provided.

Unconventionally thin concrete slabs are proposed. A minimum thickness of around 35mm is possible provided great care is taken to prevent the onset of corrosion. Since concrete cover is already well below code minimum values reliance on the protection that the concrete alone affords is insufficient.

In the first instance a cement-rich mix is called for (economies) in the concrete mix itself are undesirable). In addition partial transfer of prestress to the concrete at very early age is needed to obviate the formation of cracks due to restrained shrinkage.

Despite the fact that very good quality concrete will be achieved (ease of placement, ease of compaction, good thickness control etc.) a further line of defence against the ingress of water is thought prudent. Shortly after casting the concrete an epoxy resin coating reinforced with glass fibre can be applied to the top surface. This coating will serve a dual role. It will limit the loss of moisture from the conrete at early age promoting curing and minimising early age shrinkage. Subsequently it will prevent the penetration of rain water into the concrete.

7. CONCLUSIONS

The concrete origami concept opens the way to a whole new range of concrete products and structures whose scope is limited only by imagination. Major savings are possible since the simplest of formwork can be used.

Where structures are folded from flat elements care is required in detailing the joints which are to be rotated and subsequently frozen by concreting.

Recognition of the fact that concrete slabs can be permanently deformed without damaging their fabric leads to dome and barrel shaped roofs and many other applications.

The final outcome has been the tent-like formwork to hang freely in the catenary shape. Critical features of these ideas have been explored experimentally and have been found to work as expected.

Roof spans as large as 50m and beyond are possible using these techniques. Applications such as for the roofs or sporting arenas, grain storage buildings and aircraft hangars seem immediately feasible.

8. ACKNOWLEDGEMENTS

My colleague, Associate Professor R.Q. Bridge, has played a major part in the development of the ideas which have flowed from the original enunciation of the concrete origami concept. His contribution is gratefully acknowledged. In addition undergraduate students I. Archer. G. Arena, D. McDonald and R. Anderson have added significantly to the work through their undergraduate thesis projects.

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Tête de Défense – Caractéristiques essentielles

Die wesentlichen Merkmale des Bauwerks "Tête de Défense"

Essential Characteristics of the Unusual Building "Tête de Défense"

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RÉSUMÉ
L'article présente le "Cube", cet ouvrage exceptionnel construit à Paris dans le quartier de la Défense, donne une description de la structure, énumère les principaux problèmes rencontrés et les solutions choisies pour les résoudre.

SUMMARY
This article on the "Cube", an unusual building constructed in the La Défense area of Paris, describes the structure, lists the numerous problems encountered in designing and erecting it and how they were solved.

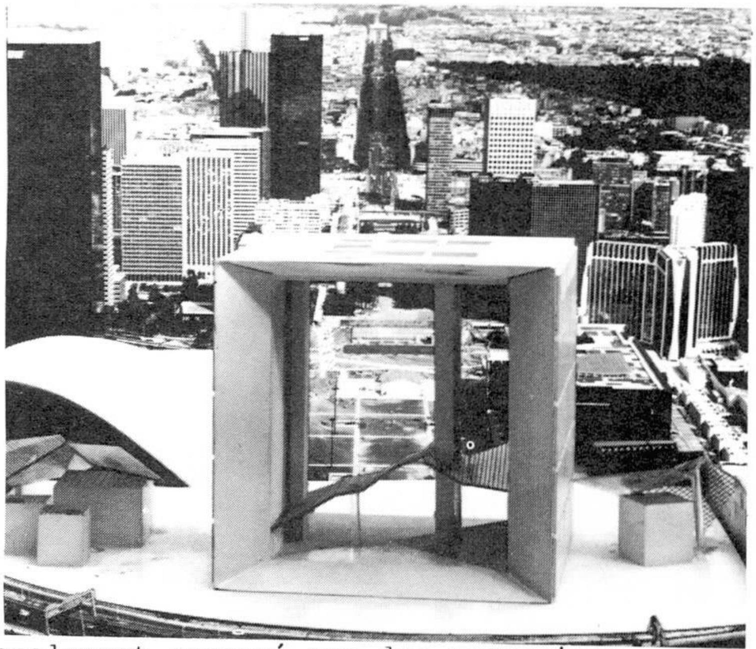
ZUSAMMENFASSUNG
Der Beitrag beschreibt den "Cube", ein ausserordentliches Bauwerk im Quartier "La Défense" in Paris. Es wird eine Beschreibung der strukturellen Ausbildung gegeben. Die wichtigsten angetroffenen Probleme und deren Lösung werden besprochen.

1 - PRESENTATION DU PROJET

La Grande Arche de la Défense s'inscrit dans l'axe historique de l'Arc de Triomphe de l'Etoile à celui du Carrousel en passant par les Champs Elysées et l'obélisque de la Concorde.
L'ouvrage en superstructure est un cube, ouvert sensiblement selon l'axe historique, composé de 2 parois verticales de bureaux appelées 'pattes' reliées entre elles dans la hauteur des niveaux inférieurs (0,1,2) d'une part, et dans la hauteur des niveaux supérieurs (34,35,36) d'autre part, par 2 constructions horizontales de grande portée appelées 'plateaux'.

1.2 DIMENSIONS PRINCIPALES

Les dimensions hors tout de l'ouvrage sont sensiblement 112 X 107 X 111 m d'où l'appellation courante de 'Cube' (dimensions au-dessus du niveau général de la dalle de la Défense, hors infrastructure).
La distance horizontale entre les 2 parois verticales 'pattes' est de 70 m et la distance verticale entre les structures horizontales des 'plateaux' est de 90 m. Pour l'anecdote, le vide intérieur correspondant permettrait de loger facilement Notre Dâme de Paris!
L'épaisseur des 'pattes' est de 18,50 m . La hauteur du plateau inférieur est de 9 m, celle du plateau supérieur est de 10 m.



Chaque 'patte' comprend 37 niveaux (RC à 36) de 2,80 m de hauteur de dessus de plancher à dessus de plancher plus 4 étages techniques de 1,40 m de hauteur.
Chaque 'plateau' comprend un vide technique d'environ 2 m de hauteur, un niveau 'noble' de grande hauteur (5 à 6 m) localement recoupé par des mezzanines.
L'ouvrage ne comporte aucun joint de dilatation en superstructure.
Quatre ascenseurs extérieurs, circulant le long d'une charpente métallique, relient les 'plateaux' inférieur et supérieur.
Enfin, une structure légère, les 'nuages', vient couvrir partiellement le 'plateau' inférieur.Cette structure initialement prévue portée, sera finalement suspendue aux 'pattes' et au 'plateau' supérieur.

1.3 INFRASTRUCTURE ET OUVRAGES EXISTANTS

L'infrastructure composée de 5 sous-sols est totalement indépendante de la superstructure au point de vue structurel.
L'infrastructure sous 'pattes' du projet initial ne comportait que 3 sous-sols mais fut approfondie à la demande du Maitre d'Ouvrage.
Le niveau le plus haut de cette infrastructure est constitué par un plancher de grande portée supportant un jardin d'acclimatation.
Les sous-sols sont traversés, d'une part, par une voie SNCF existante (voie de la Folie) sensiblement dans le sens Nord-Sud, d'autre part, par les 2 voies à construire, de l'autoroute A 14 dans le sens Est-Ouest.
Enfin, 3 voies du RER existantes passent d'Est en Ouest à quelques métres sous l'ouvrage.

1.4 FONDATIONS

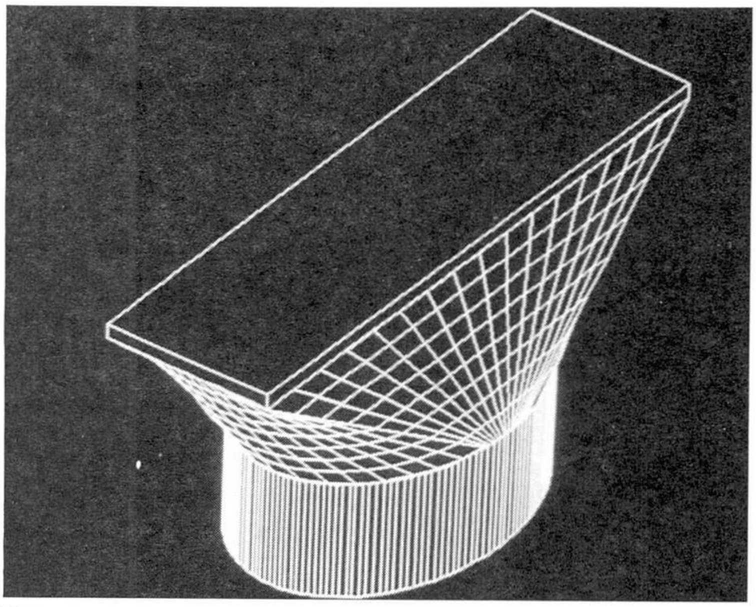
Comme nous l'avons dit plus haut, la superstructure est indépendante de l'infrastructure. Elle est appuyée sur 12 piles elliptiques (8 centrales de 28 m2 environ et 4 d'angle de 19 m2 environ de section) en béton armé reposant directement sur le banc calcaire par un empattement ramenant le taux de contrainte au sol à 30 b maximum.
La hauteur des piles entre leur niveau d'assise et le niveau d'appui du 'Cube' sur les chapiteaux qui constituent leurs extrémités supérieures, est de 30 m.
Le reste de l'infrastructure est fondé sur des semelles isolées ou filantes ou sur puits sur les Marnes et Caillasses.

1.4 DELAIS

Les délais d'éxécution étaient extrêmement courts, compte-tenu d'une infrastructure compliquée qui fut soumise à d'importantes modifications de programme et d'une superstructure, certes répétitives, mais pénalisée par des contraintes techniques inhabituelles que ce soient en ce qui concerne les densités d'armatures aussi bien que les procédés de construction et qui fut, elle aussi, soumise à d'importantes modifications.
Le délai prévu et réalisé pour l'exécution d'un étage courant des 'pattes' soient 3200 m2 est de 4 jours ouvrables.

2 - DESCRIPTION DE LA STRUCTURE

2.1 PILES ET CHAPITEAUX



Chacun des 12 appuis du'Cube' comprend :
* un empattement d'appui sur le calcaire (côte 34 NGF)
* un fût de section rectangulaire dans sa partie enterrée et elliptique dans sa partie en élévation
* un chapiteau, destiné à recevoir les appuis, limité par une surface réglée s'appuyant en partie basse, sur l'ellipse du fût et en haut, sur un rectangle.

Il est prévu un dispositif pour changer les appuis.
Ces piles, qui ont un effet architectural certain,ont été été visibles avant la construction de l'infrastructure et ne le seront · évidemment pas dans l'ouvrage terminé.

Dimensions pour la pile courante:
- Empattement d'assise: rectangle 12 m x 9 m x 5 m de haut
- Fût enterré: rectangle 7,7 m x 5,7 m x 10 m de haut
- Fût en élévation: ellipse: 7 m x 5 m x 10 m de haut
- Chapiteau: rectangle supérieur 12 m x 3,5 m hauteur 4 m
- Appareils d'appui: 4 x 13 = 52 unités 700x700 6(12+3)

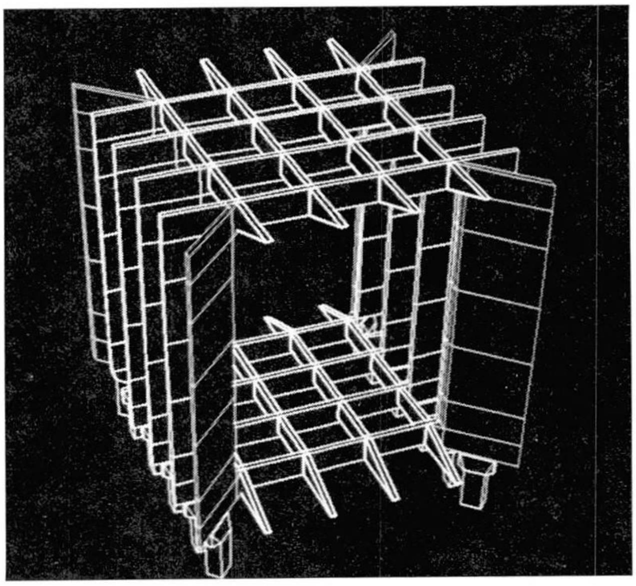
2.2 PATTES

Ce sont les 2 bâtiments de bureaux qui portent le 'plateau' supérieur.
Chaque 'patte' comprend les ouvrages principaux suivants :
* Mégastructures verticales qui sont les porteurs principaux
* Mégastructures techniques)
* Façades) qui constituent le contreventement
* Voiles intérieurs) longitudinal
* Planchers et gaines intérieures

2.21 Mégastructures verticales et pignons

Les mégastructures verticales sont les 8 éléments (4 par 'pattes') verticaux qui liaisonnent les poutres des plateaux inférieurs et supérieurs et constituent les éléments verticaux des 4 cadres qui sont l'ossature du 'Cube'.Ce sont des voiles de 1,50 à 2,20m d'épaisseur dans lesquels se trouvent intégrées les gaines techniques verticales (de 90 cm de largeur).Ces mégastructures sont en fait divisées en 3 piédroits liés par des linteaux à chaque étage de façon à permettre la circulation horizontale dans le bâtiment.Les parties délicates, car extrèmement sollicitées et soumises aux impératifs conjugués de cablage et de réservations, sont les noeuds de raccordement des poutres horizontales des plateaux et de ces méga- verticales. La taille de ces noeuds est de 18 m de largeur, 9 m de hauteur et 2 m d'épaisseur moyenne.
Les pignons sont inclinés à 45° sur l'axe longitudinal du bâtiment. Ils reposent sur les 4 piles d'angle. Ce sont des voiles pleins de 0,50 m d'épaisseur avec un épaississement à 2 m sur les 3 premiers niveaux au-dessus des piles. Ces pignons ne portent pas directement les poutres du plateau supérieur mais du fait de leur liaison avec les mégastructures verticales par les éléments longitudinaux,peuvent voir leurs charges fortement majorées par des transferts de plusieurs milliers de tonnes.

2.22 Mégastructures techniques



Ce sont des poutres horizontales longitudinales aux niveaux 7 - 14 - 21 - 28 qui assurent une partie des liaisons longitudinales. Situées au droit de chaque façade et des 2 voiles intérieurs longitudinaux,ces poutres hautes de 1,58 m, larges de 0,35 à 1,00 m,ont aussi pour rôle de reporter sur les mégastructures verticales les charges de 7 étages de façades ou de voiles intérieurs. Un joint diapason horizontal de mégastructure à mégastructure existe sous chacun des étages techniques au droit des façades et voiles intérieurs.
Cette disposition permet une mise en charge progressive des mégas verticales et évite que des charges considérables ne se retrouvent au pied des résilles de façades ou des voiles intérieurs.
Ces poutres sont précontraintes longitudinalement.

2.23 Façades

Elles sont constituées par une résille en béton armé de 0,35 m d'épaisseur (exeptionnellement 0,25 m d'épaisseur dans la hauteur des convecteurs), qui sera habillée par des panneaux carrés de menuiseries de 2,80 m de côté. Ces panneaux sont fixés aux 4 angles par des pièces spéciales préréglées dans le béton de la résille.Cette résille constitue des ensembles de 21 m par 21 m entre 2 mégas verticales et 2 mégas techniques, très rigides et qui assurent une partie du contreventement longitudinal. Aux niveaux bas et haut, les façades sont constituées d'un voile plein sur 1 niveau (3,00m) côté extérieur et sur 3 niveaux (10 m) côté intérieur. Ces voiles qui sont précontraints renforcent la liaison entre les pignons et mégatructures verticales, et participent à la répartition des charges entre ces éléments.

2.24 Voiles intérieurs

Ces sont les murs séparatifs longitudinaux entre la travée centrale de chaque 'patte' comprenant les circulations verticales, les sanitaires et locaux techniques, et les 2 zones latérales de circulations horizontales et bureaux.Leur épaisseur est de 0,18 m. Ils sont percés de nombreuses baies(circulations et réservations) qui en atténuent fortement la rigidité (les linteaux n'ont que 0,60 m de hauteur). Ils participent également au contreventement longitudinal.

2.25 Planchers et gaines intérieures

Pour améliorer les délais,les planchers sont constitués de dalle béton de 12 cm d'épaisseur, précontraintes (sauf les petites portées) sur lesquels est coulée une chape ciment collaborante de 6 cm d'épaisseur. Les gaines intérieures,que ce soient les gaines d'escaliers ou les gaines techniques situées hors des mégastructures, sont également préfabriquées, en béton armé ainsi que certains petits voiles transversaux.

2.3 PLATEAUX

La structure principale des plateaux est constituée de 4 poutres principales (mégastructures horizontales principales) espacées de 21 m et de 4 poutres secondaires (mégastructure horizontales secondaires) espacées également de 21 m se terminant à leurs 2 extrémités par des consoles de 21 m de portée. L'ensemble de cette structure est précontraint. Les unités utilisées sont des unités de 19T15 et 7T15.

2.31 Mégastructures horizontales principales

Ce sont des poutres en I de 70 m de portée entre murs de façades des pattes. Leur hauteur est de 8,4 m en 'plateau' inférieur et de 9,5 m en 'plateau' supérieur. Leur épaisseur est de 1 m. Ces poutres sont percées sur leur longueur de 2 baies de 6,00 de large par 4 m de haut et 4 de 2,80 x 4,00. Les forces de précontraite sont de 12.000 t pour les poutres de rive et de 9.000 t pour les poutres centrales.

2.32 Mégastructures horizontales secondaires

Ce sont des poutres rectangulaires qui entretoisent les poutres principales et supportent les parties en porte à faux des plateaux (21 m de porte à faux).Ces poutres sont constituées entre 2 mégas principales d'une membrure inférieure et d'une membrure supérieure reliées par un montant vertical à mi-portée. Elles comportent 2 âmes de 0,30 m d'épaisseur séparées par un vide technique de 1,10 m en plateau inférieur et une âme pleine de 1,10 en plateau supérieur.Les forces de précontrainte dans ces poutres secondaires et leurs consoles, sont de 3.000 t environ.

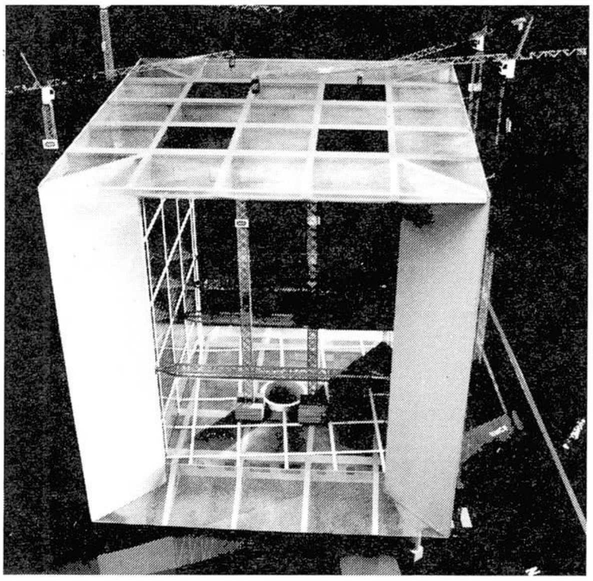
2.33 Planchers entre mégastructures

Les niveaux bas des plateaux sont constitués d'un plancher double dont le vide forme étage technique. Ce plancher double comporte 2 dalles en béton armé de 10 et 12 cm d'épaisseur portées par des poutres 'tertiaires' en béton armé, espacées de 2,80 m. Ces poutres ont 2,10 m de hauteur de la dalle inférieure à la dalle supérieure, 20 cm d'épaisseur et portent sur environ 20 m d'une poutre principale à l'autre.
Les niveaux hauts sont constitués de planchers à nervures croisées en béton armé d'une hauteur variant de 80 à 95 cm suivant les trames. Ces planchers portent dans un sens sur les poutres principales et dans l'autre sur les poutres secondaires.

3 - PROBLEMES RENCONTRES ET SOLUTIONS CHOISIES

3.1 STABILITE EN PHASES PROVISOIRES DE CONSTRUCTION

Ce problème essentiel résulte du fait que l'ouvrage n'est stable que lorsque le cadre est fermé c'est-à-dire lorsque le plateau supérieur est réalisé.En effet, les poutres du plateau inférieur ont besoin d'un encastrement important pour supporter les charges gravitaires, encastrement qui ne peut être acquis du seul fait des piles.De plus à ces efforts gravitaires, s'ajoutent les effets du vent transversal sur un IGH dont la façade peut être encore partiellement ouverte.



A partir de là, 2 solutions ont été envisagées :

* La première était de maintenir un étaiement sous les mégapoutres principales du plateau inférieur jusqu'à l'exécution des mégapoutres principales du plateau supérieur,de façon à augmenter la raideur d'ensemble et à les soulager d'une grande partie des charges gravitaires.
* La deuxième était de maintenir l'écartement des 'pattes' par un butonnage qui reconstitue un cadre provisoire en attendant le cadre définitif.

C'est finalement la deuxième solution (butons) qui fut choisie pour les raisons principales suivantes :
- moindre sensibilité aux effets possibles de tassements différentiels
- suppression de l'inconvénient très important pour le planning général TCE, de la présence dans l'infrastructure d'étaiements capables de supporter plusieurs milliers de tonnes par appui.
- suppression de l'obligation de définir très tôt les passages de ces étaiements dont certains devaient rester incorporer à l'ouvrage.

Par contre cette solution avait quelques inconvénients importants :

- nécessité de 4 butons d'une force unitaire de 2 000 t, d'une longueur de 70 m, pesant chacune environ 90 t.
- gène importante dans l'espace aérien de travail des grues et ensuite décalage de la pose de certaines trames de façade intérieure.
- création de contraintes supplémentaires entre l'avancement des pattes et du plateau inférieur.

3.2 MODES DE CONSTRUCTION DES POUTRES DES PLATEAUX

Le problème était d'abord la réalisation des Mégapoutres principales.
Après avoir envisagé de construire les voussoirs en encorbellement, nous nous avons opté pour le coulage sur un cintre utilisable pour les méga-poutres inférieures et pour les mégapoutres supérieures, charge à porter 35 t/ml. Ce cintre d'un poids de 285 t, s'appuie en 4 points, 2 au droit des façades et 2 appuis intermédiaires.
Dans l'utilisation en plateau supérieur, les 2 appuis intermédiaires sont constitués par 2 fûts de grue capables de supporter 1.000 t par fût.
Ces fûts s'appuient sur les mégapoutres inférieures qui, en l'absence de charges d'équipements et d'exploitation, sont capables de supporter les réactions correspondantes.Ces fûts n'ont pas de résistance sensible en flexion compte-tenu de leur rigidité relative très faible et n'assurent qu'une réaction verticale. La stabilité transversale de l'ensemble cintre + mégapoutre supérieure en construction a nécessité de couler les poutres par éléments, des pattes vers le centre,et de précontraindre ces éléments au fur et à mesure, comme dans un exécution par voussoirs préfabriqués.
Le cintre sera, de plus, épinglé sur les voussoirs déjà exécutés pour le le rigidifier et éviter les désaffleurements entre 2 phases successives.

3.3 MOUVEMENTS DE LA STRUCTURE

Ces mouvements dûs à la mise en charge progressive du plateau inférieur (y compris efforts de précontrainte) et des butons, aux effets thermiques différentiels, à l'exécution du plateau supérieur ont pu être limités dans une fourchette d'aplomb des pattes de + 10 m/m par une étude détaillée du programme de mise en charge des butons.

3.4 COMPLEXITE DE L'ETUDE DE STRUCTURE

3.41 Calcul d'ensemble

Il a été conduit en utilisant un programme de calcul de structure tridimensionnelle à barres (STRUDL). Le modèle utilisé pour la détermination des efforts dans les 'pattes' comportaient 6.632 noeuds et 11.156 barres compte-tenu de la complexité de la structure. Des modèles plus petits dans lesquels les 'pattes' étaient remplacées par des structures simplifiées, de raideur équivalente, ont permis l'étude des plateaux. Les phases provisoires étudiées ont été limitées à 7 pour les 'pattes', par contre pour les 'plateaux', l'étude a suivi de très près l'exécution des différents ouvrages (76 phases provisoires étudiées)

3.42 Mise au point des plans de cablages et d'armatures

Le raccordement des mégastructures horizontales et verticales,les'noeuds' a nécessité une mise au point extrêmement laborieuse pour conjuguer les impératifs de résistance,de passage des cables et armatures passives avec ceux des baies de circulation, des percements des lots techniques et des modifications résultant des mises au point de conception. Le respect des équarissages imposés par les nécessités fonctionnelles du projet a conduit à des densités d'armatures passives exceptionnelles (300 kg/m3).

3.5 MATERIEL DE COFFRAGE TRES PERFORMANT

Les impératifs du planning nous ont conduit à utiliser des coffrages autoréglables selon une technique déjà employée par notre entreprise sur d'autres grands immeubles de la Défense.
Ces matériels exceptionnels permettent l'éxécution sur chaque patte de :
- 1 mégastructure sur 2 niveaux chaque jour (18 m x 5,6 m)
- 2 x 21 ml de façade chaque jour.

3.6 MISE EN OEUVRE DES BETONS

Compte-tenu des énormes quantités de béton à mettre en oeuvre et des problèmes de saturation de grues, tous les ouvrages épais sont bétonnés à la pompe, à des distances de plus de 50 m et des hauteurs de plus de 100 m.

Structural Concepts for Ultra-Tall Concrete Buildings

Concepts de structure pour de très hauts gratte-ciel en béton

Strukturelle Konzepte für extrem hohe Betonbauten

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SUMMARY
In the last twenty-five years, there has been dramatic improvement in concrete technology namely high-strength concrete, pumping of concrete and advances in formwork. Concrete has structural design advantages for tall buildings due to higher mass and higher damping. This article describes concepts for ultra-tall concrete buildings up to one mile (1 600 m) high.

RÉSUMÉ
Pendant les derniers vingt cinq ans, il y a eu un progrès énorme dans la technologie du béton, à savoir : le béton a haute résistance, le béton pompé, et le progrès dans les coffrages. L'avantage du béton armé dans les gratte-ciel est dû premièrement à sa masse, secondement à l'effet d'amortissement. Cet article décrit des idées pour l'étude de très hauts gratte-ciel, jusqu'à mille six cents mètres de hauteur.

ZUSAMMENFASSUNG
In der letzten fünfundzwanzig Jahren sind in der Betontechnologie grosse Fortschritte erzielt worden, insbesondere bei den hochfesten Betonen, beim Pumpbeton und in der Schalungstechnik. Infolge grösserer Masse und grösserer Dämpfung weist die Betonbauweise für grosse Bauhöhen Vorteile auf. Der Beitrag beschreibt Konzepte für extrem hohe Betonbauten mit einer Höhe bis 1600 Metern.

1. INTRODUCTION

In the last twenty-five years, there have been dramatic advancements in the technology of construction of tall concrete buildings with the advent of new forming systems such as slip-forming, flying forms, gang-forms, etc. Also, the development of ultimate strength design, the development of structural light-weight concrete, the development of high strength concrete, the use of admixtures (such as superplasticizers), and concrete pumping techniques have given concrete a great boost for tall structures. The evolution of structural systems particular to concrete construction, notably by the late Dr. Fazlur Khan, gave rise to the potential for taller concrete structures. In 1968, One Shell Plaza in Houston, a 50-story all light-weight concrete building was designed and constructed and became the tallest concrete building in the world. In the 1970's, Water Tower Place in Chicago was built and to this day, holds the record as the world's tallest concrete building at 864 feet (263m) in height. The tallest concrete structure, however, is the CN Tower in Toronto which is 1,500 feet (457m) tall. In 1977, the 75-story, 1,000 foot (304m) tall Texas Commerce Plaza was constructed in Houston. This building is the tallest exterior composite building in the world and has two unique features: First, all the concrete in the project was pumped and second, self-jacking exterior gang-forms were used for the construction of the exterior composite frame. The pumping of the concrete to 1,000 feet (304m) stands today as a record for the tallest height of pumping of concrete with a single-stage pump. The self-jacking exterior forms enabled the construction to proceed at a very rapid pace, and 72 floors of the building were constructed in eleven months due to this combination of techniques.

2. DESIGN CONSIDERATIONS

Tall buildings in non-seismic areas are governed not so much by strength considerations but by performance characteristics under wind loads. The most important considerations here are the sway of the building under wind loads and the motion perception that affects occupant comfort. Due to the inherently higher moments of inertia in concrete members and the higher modulus of elasticity for higher concrete strengths, concrete building design is generally not governed by the sway limits under wind loads. It is known that in tall structures, the two most effective methods of obtaining better motion perception performance under wind loads is to increase the mass and to increase the damping. Both of these factors favor concrete buildings. A concrete building will have a mass in the range of 10 to 25 pounds per cubic foot (160 to 400 Kg/m3) and a damping value ranging from one to two percent of critical damping. Both these values are higher than those for other materials and hence, concrete buildings perform better from motion perception considerations.

The evolution of economical structural systems for tall buildings in general has given rise to two guiding principles:

- Utilize as much of the gravity load as possible to resist the resultant axial forces due to wind load.
- Concentrate the gravity loads on the periphery of the building and preferably, at the exterior corners.

In structures made of very light-weight materials, it is essential that the two principles be followed to get economical design. The transfer of the gravity load to the exterior of the building results in the need for horizontal transfer elements (beams or trusses) at discrete levels in the building. These levels are the so-called "interstitial floors" and their costs offset some of the savings. In conventional building design (less than 100-stories tall), the gravity load transfer elements have to span in the range of 200 feet (60m). The spans will increase for taller structures.

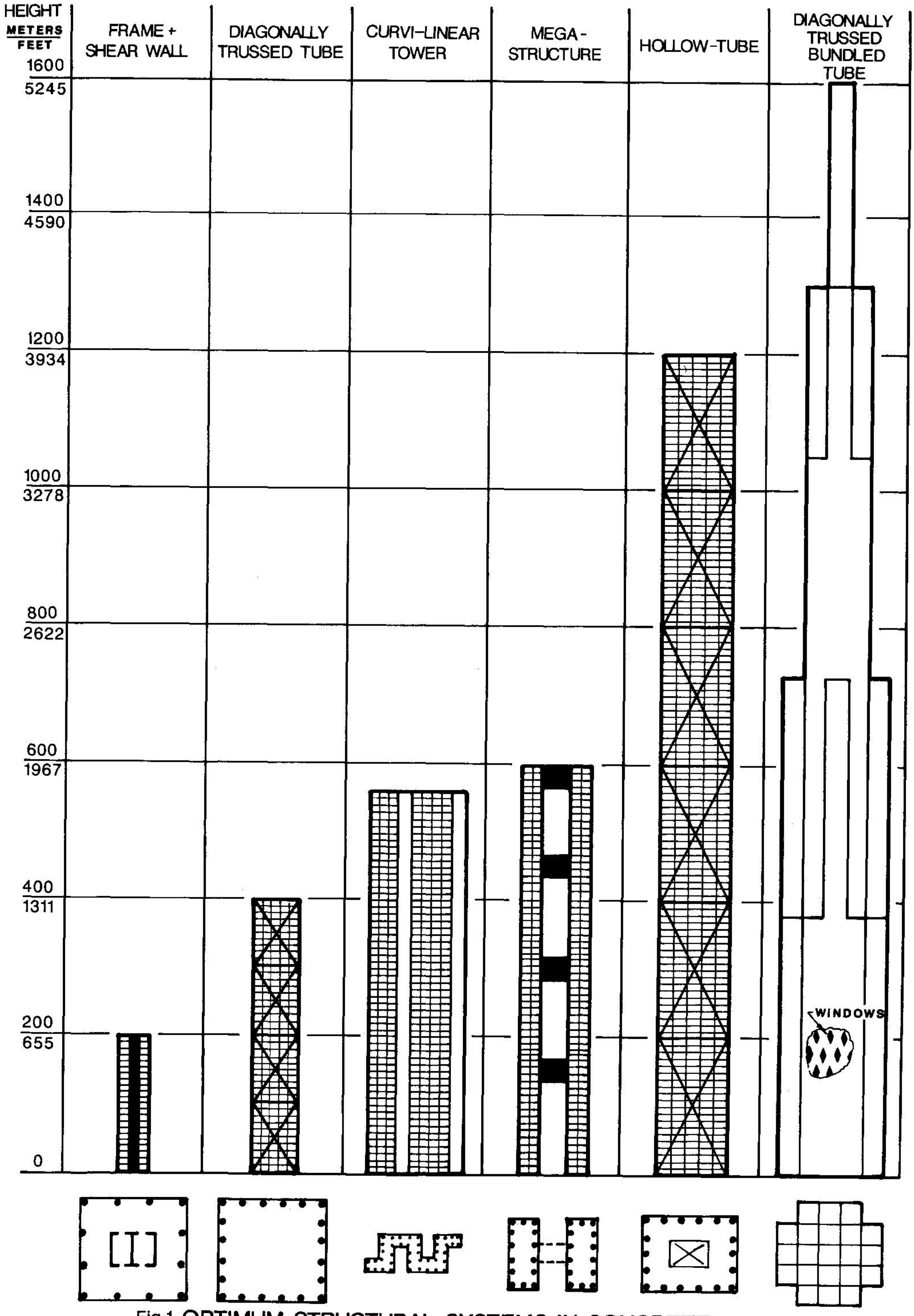

Fig.1 OPTIMUM STRUCTURAL SYSTEMS IN CONCRETE

In concrete structures, because of the high gravity load, it is seldom necessary to have these gravity load transfer elements. For ultra-tall structures (greater than 100-stories tall), the principles enunciated earlier can be achieved by several techniques. Fig. 1 shows a range of structural systems:

(a) Shape of the building in plan: Utilizing curvi-linear shapes or bent "hat" shapes to achieve maximum "depth" with a relatively narrow floor plan, large resistances can be built-up. This is analogous to the development of corrugated decking used for floors and roofs.

(b) "Megastructures": These consist of individual rigid building blocks that are linked together at discrete levels.

(c) "Hollow-Tubes": These are buildings where the inside of the floor plan is hollowed-out into an atrium (or a series of atria). The shape naturally tends to satisfy the two principles of design. If the building loads are further concentrated in the corners, additional advantages are gained.

(d) "Bundled-Tubes": In this concept, load bearing walls or columns are placed to subdivide a floor plan into cells. The columns will be "diagonally" braced. In the case of walls, the openings needed for architectural function are so arranged as to preserve the integrity of the wall.

3. DESIGN EXAMPLE

It was decided to investigate the feasibility of constructing a mile-high (1,600m) building in concrete as shown in Fig. 2. The selected building is 500 feet (150m) square at the base in order to obtain a good aspect (height/width) ratio, arranged in modules 100 feet (30m) square. Diamond shaped windows are the result of designing a "trussed bundled tube." This results in an extremely rigid exterior that resists a major portion of the wind loads and other forces.

Interior columns are spaced 20 feet (6m) on centers along the modular lines and are run continuously from top to bottom without any transfers. This forms 100 ft. (30m) square, column-free open spaces that meet most occupancy needs. As the elevators drop off, the structural modules are dropped off as shown in Fig. 2. The modules top out at 1,250 feet (381m), 2,400 feet (732m), 3,450 feet (1,051m) 4,250 feet (1,296m) and then on to the top of the building at 5,280 feet (1,600m). This gives the building a tapered appearance on the skyline.

A precast floor slab system was considered for the floor framing but a "super-waffle" with ribs at 20 feet (6m) on centers in each direction was finally selected. The waffle ribs are 2 ft. 3 in. (68cm) deep at the midspan and 3 ft. 6 in. (1.06m) deep at the columns. The 5-1/2 in. (14cm) floor slabs are light-weight concrete to minimize some of the dead load coming down the structure. An advantage of the waffle floor slab is that it distributes gravity loads very well. A drawback is the large amount of formwork required.

Wind pressures increase gradually from the bottom to the top. Using the Canadian Building Code, the gust response factor G is 1.02. Wind shear at the base is about 95,000 kips (43,100T) and the base overturning movement is 230 million kip-feet (32 x 106T-m). The sway is approximately 8 ft. 6 in. (2.6m) which is height divided by 621. The maximum wind stress in the exterior wall at the base is approximately 825 psi (58 kg/sq. cm), whereas the gravity stress under working loads is 6,100 psi (430 kg/sq. cm).

The fundamental period is 25 seconds and the building weighs 25 lbs./cu. ft. (400 Kg/m3) which is substantially higher than any other type of construction. For a damping value equal to 2% of critical, the Canadian Code analysis indicates acceleration at the top of the building to be higher than desirable. Experience has shown that compared to wind tunnel results, this analysis

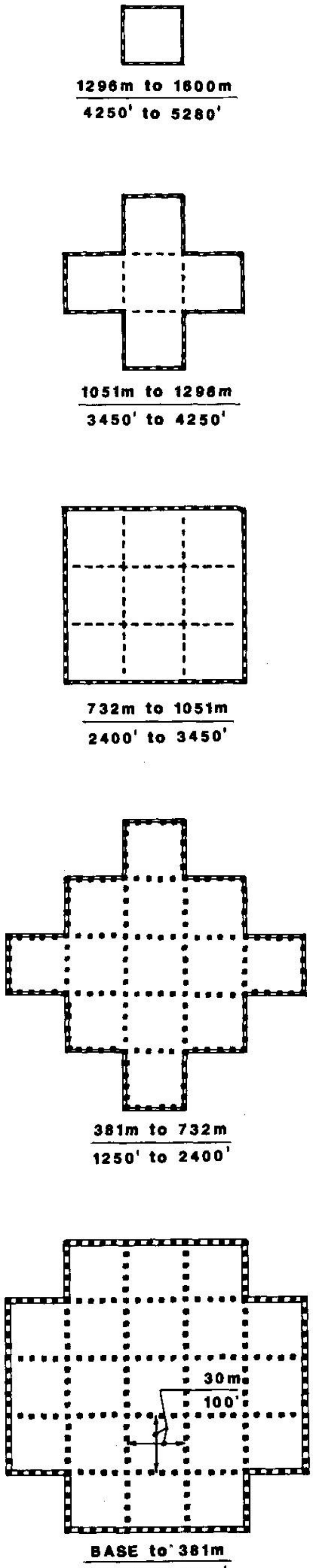

Fig.2a PLANS

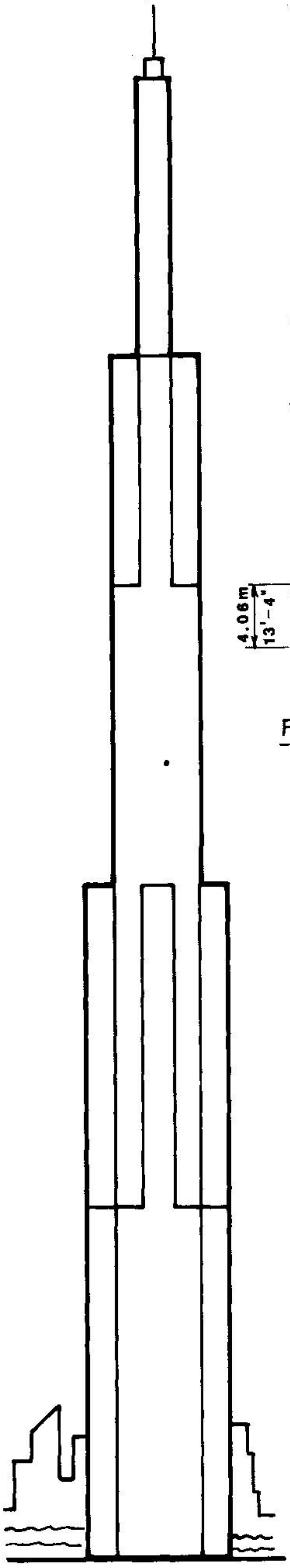

Fig.2b ELEVATION

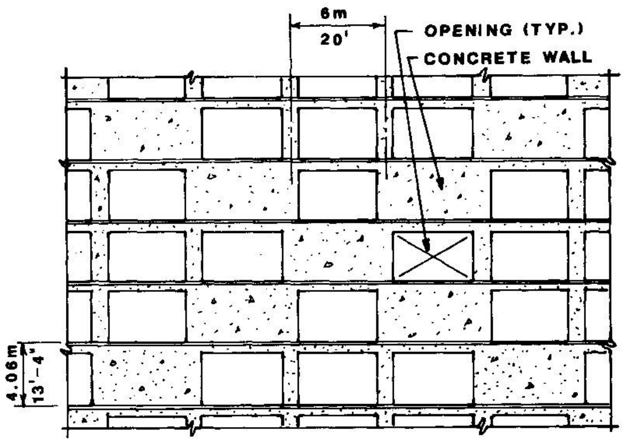

Fig.2d INTERIOR WALL - DIAGONALLY BRACED

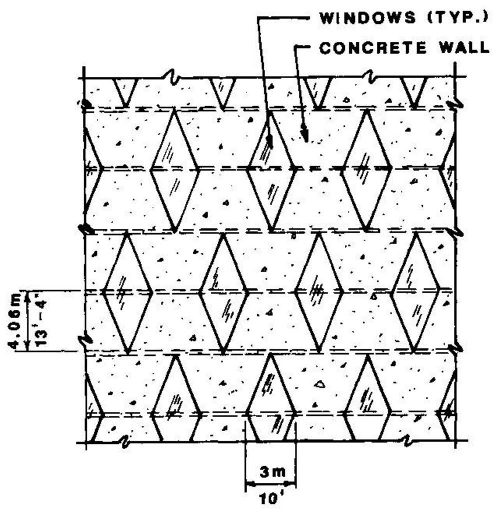

Fig.2c EXTERIOR WALL - TRUSSED TUBE

Fig.2 MILE-HIGH BUILDING

overestimates acceleration by 25% - 30%. Since the building is so massive, it is unlikely that tuned mass dampers or an active damping system will be viable. Hence, other means, such as openings through the building, will be needed to minimize the accelerations.

Because of the dia-grid arrangement of the main building structure, the foundation is a mat 550 feet x 550 feet x 18 feet thick (168m x 168m x 5.5m thick). It could be thinned out at the center of each module, however, to reduce the concrete volume. The foundation loads are about 46 kips/sq. ft. (224T/sq. m.) so a minimum allowable soil bearing of 50 kips/sq. ft. (243T/sq. m.) is necessary - a bearing capacity available in several major metropolitan areas.

One of the main problems with using architectural exposed concrete, as we propose for this skyscraper, is that the exterior structure is subject to temperature variations. In the southern part of the United States, with mean low winter temperatures of 20°F (17°C), the average temperature of the exterior columns at the lower levels would be 49°F(9°C). Since the temperature on the inside is 70°F (21°C), there is a 21°F(12°C) differential between the exterior and the interior. In northern climates, this differential jumps to 35°F(21°C). The interior arrangement of the concrete columns with diagonals has the ability to resist these thermal movements although more detailed analyses are needed for the forces in the cross walls and resulting exterior wall movements.

There are several reasons that concrete was chosen for this "mile-high" structure. Combining architecture and structure saves a great deal of cost in the building skin. Concrete is a naturally fireproof material that does not, in general, require additional fireproofing. Monolithic concrete is able to absorb thermal movements, shrinkage and creep, and foundation movements.

Because of the continuity of concrete members, the structure has a great deal of redundancy. Deflections are low and the structure is inherently stiffer than any other kind of construction.

4. CONSTRUCTION TECHNIQUES

Recent analysis has shown that a cost effective way to design concrete columns is with 1% to 2% reinforcing and as high a concrete strength as possible. This is the philosophy that was used for column and wall design throughout the building. The maximum concrete strength at the base is a readily available 14,000 psi (1,000 kg/sq. cm.). Reinforcing is kept to a minimum for simplified detailing, especially at the splices.

A job-site batch plant, located in one of the basement levels, is essential. Since concrete can now be pumped to 1,000 feet (300m), hoists will lift the concrete to a height where the pumps will take over for the last 1,000 feet (300m).

Insulated, self-jacking forms will be used for the columns and walls. All materials and personnel hoists will be on the inside of the building to provide protection against the weather. Since the walls are very thick, insulation and other techniques will have to be devised to gradually dispose of the heat of hydration.

5. CONCLUSION

The conclusion is that concrete buildings even a mile-high are technically feasible. Concrete offers many advantages for tall buildings and, with careful planning, most of the disadvantages associated with height can be overcome.