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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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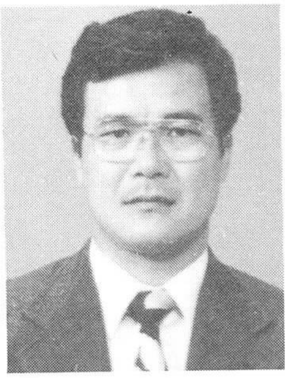

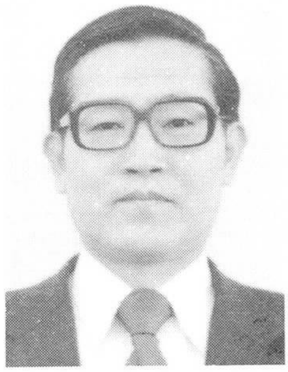

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SUMMARY
A two-stage mixing method is proposed for manufacturing concrete in which the surface condition of aggregate is first improved by a process of coating with cement paste of a suitable quality, following which the remaining water is added and 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 Qualitätssteigerung, 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.

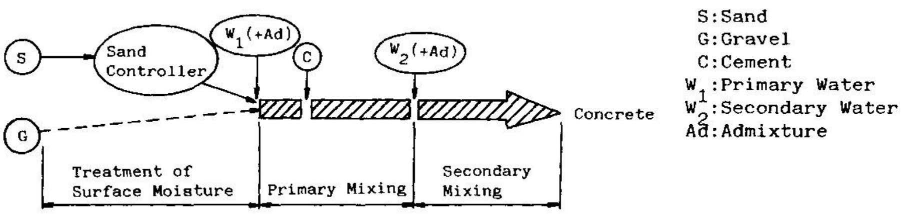

Fig. 1—Example of two-stage mixing method.

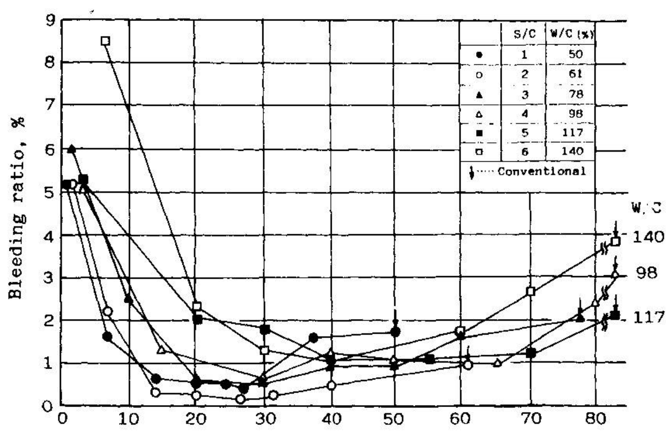

Fig. 2—Primary water-cement ratio and bleeding ratio of mortar.

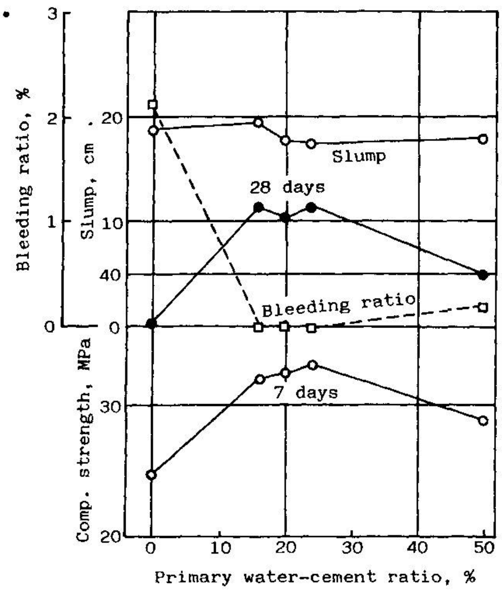

Fig. 3—Primary water-cement ratio and qualities of concrete.

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

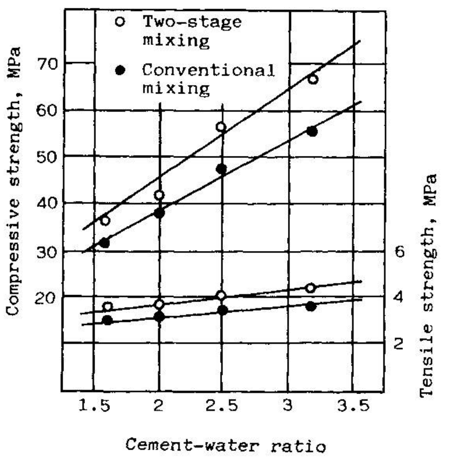

Fig. 4—Compressive and tensile strengths of concrete.

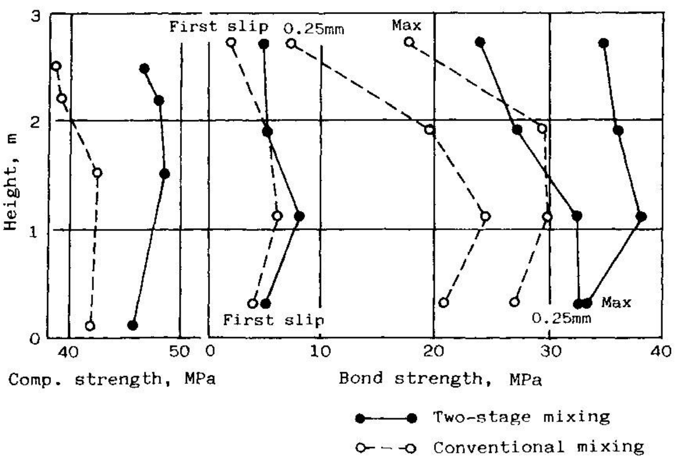

Fig. 5—Distribution of compressive strengths of cocrete and bond strengths of reinforcing steel in wall panels.

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-

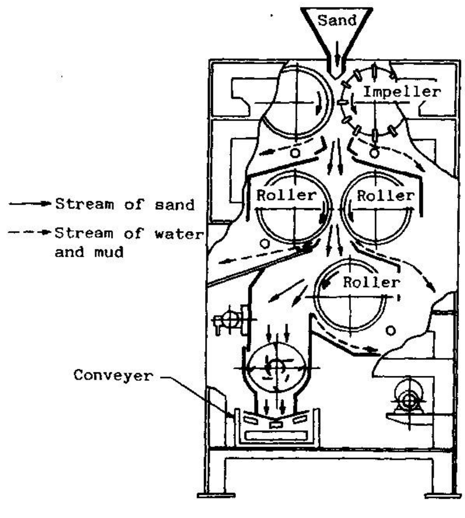

Fig. 6—Sand Controller.

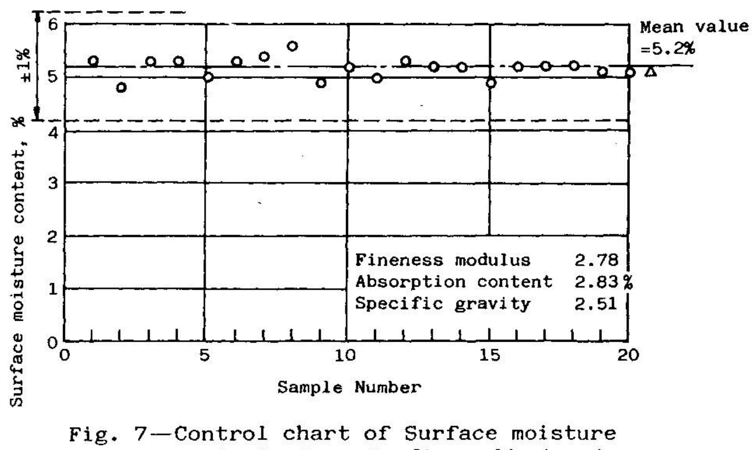

Fig. 7—Control chart of Surface moisture content of sand after adjustment by Sand Controller.

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

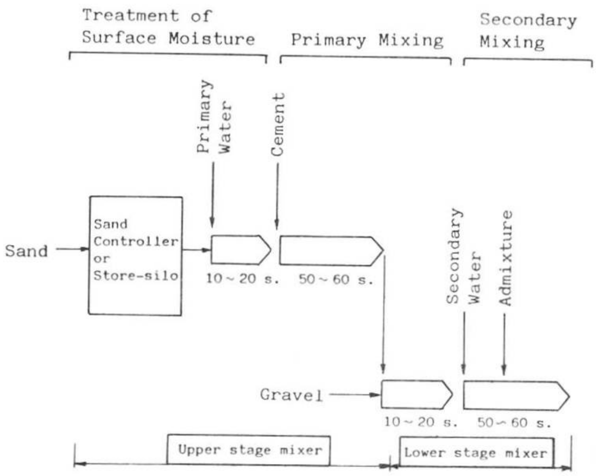

Fig. 8—Flow chart of two-stage mixing method.

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.

REFERENCES

1. HIGUCHI, Y., Coated-Sand Technique Produces High-Strength Concrete. Concrete International, May 1980.

2. MOCHIDA, Y., Progress in Concrete Technology at Seikan Tunnel. Cement & Concrete, No. 396, Feb. 1980 (in Japanese).

3. KISHITANI, K., ITO, Y., KAGA, H., and YAMAMOTO, Y., SEC Concreting Method--Fundamental Theory, Characteristics, Practicalization--Building Engineering, No. 380, April 1983 (in Japanese).

4. SHINOHARA, Y., et al., Pumping SEC Concrete for Dam Construction, Concrete Journal, Vol. 23, May 1985 (in Japanese).

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

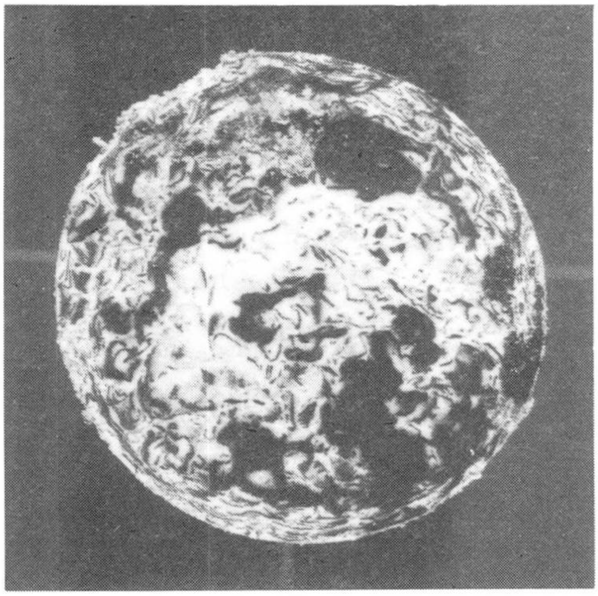


Photo.1 Section of wheat flour and red ink (liquid phase) mixture

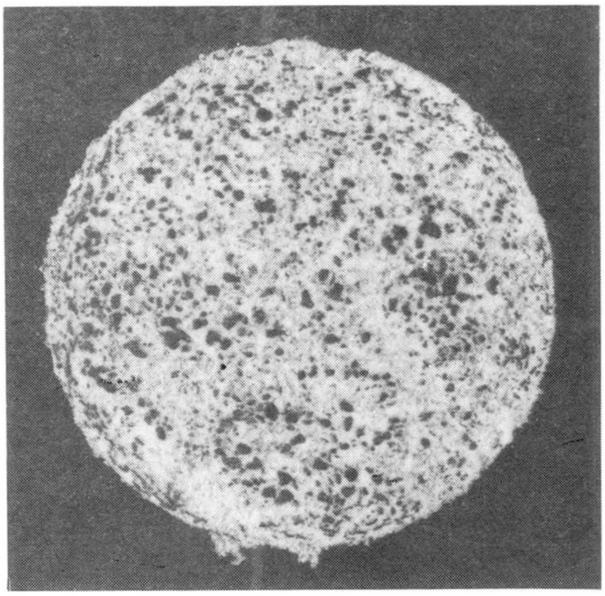


Photo.2 Section of wheat flour and red ink (frozen and sliced) mixture

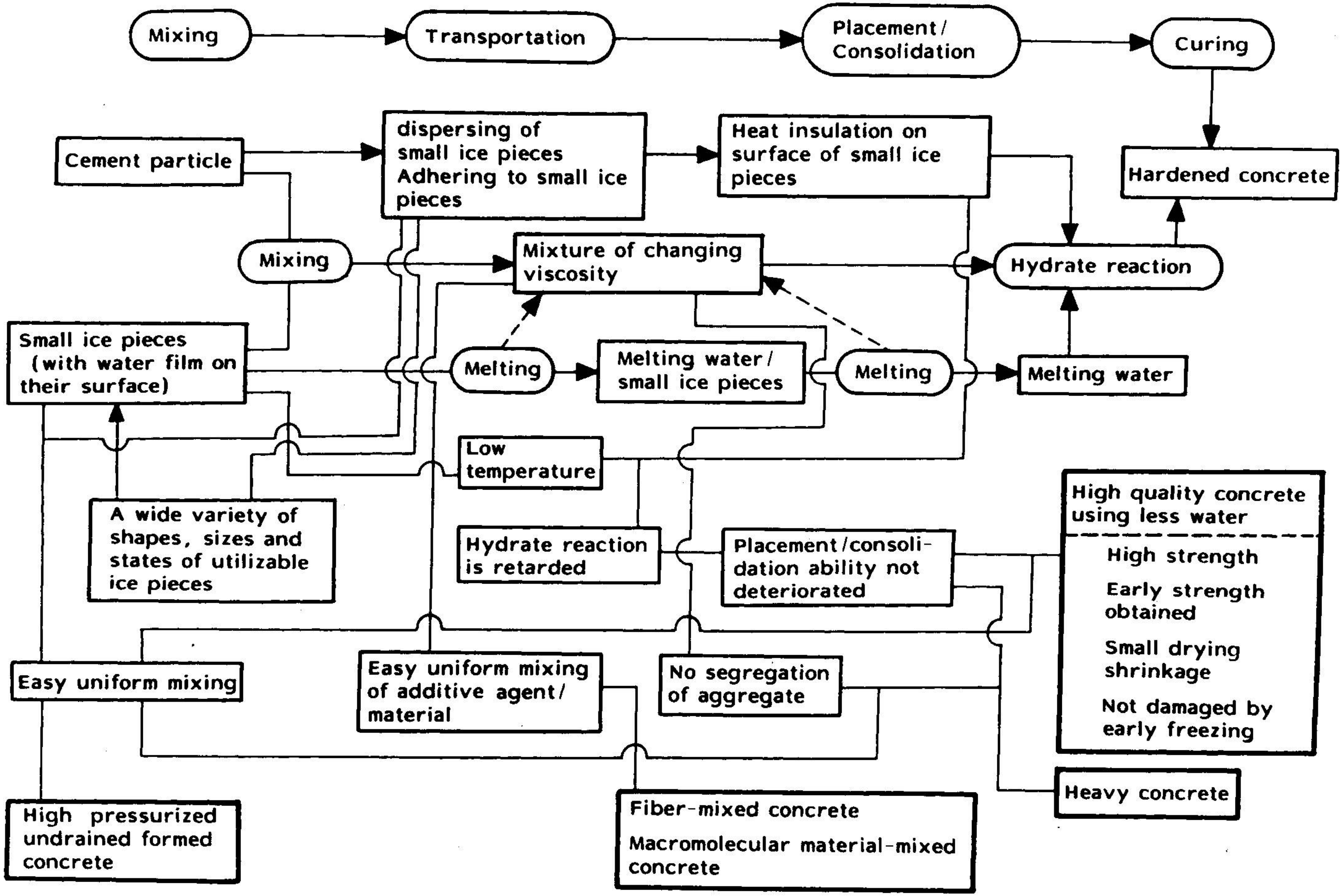


Chart 1 Characteristics of concrete production method utilizing small ice pieces and then melting process

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

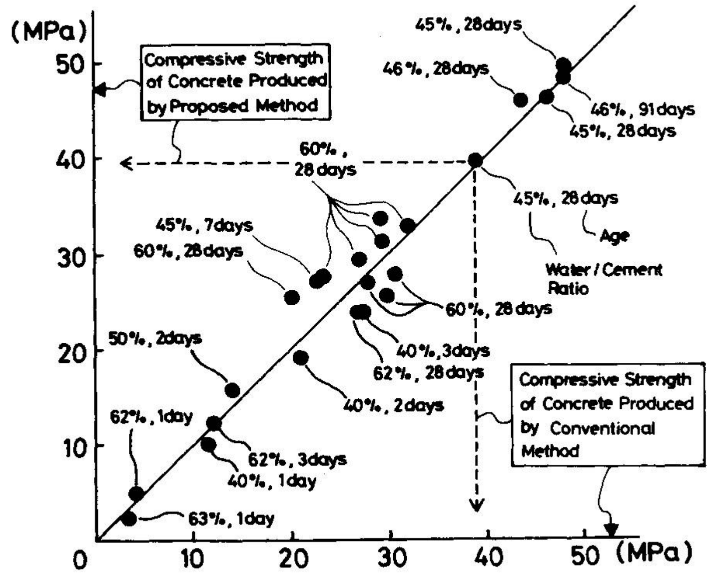


Fig.1
Compressive strength of the concrete produced by proposed method and of conventional concrete

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.

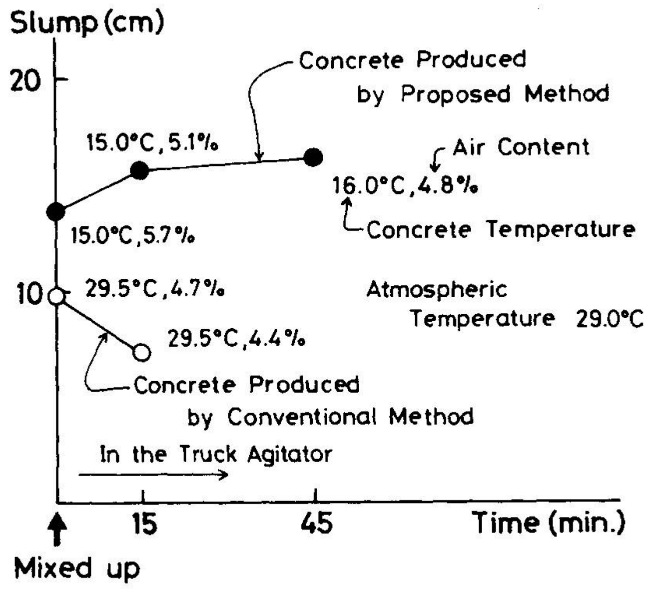


Fig.2 Slump of the concrete produced by proposed method and of conventional concrete

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.

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.

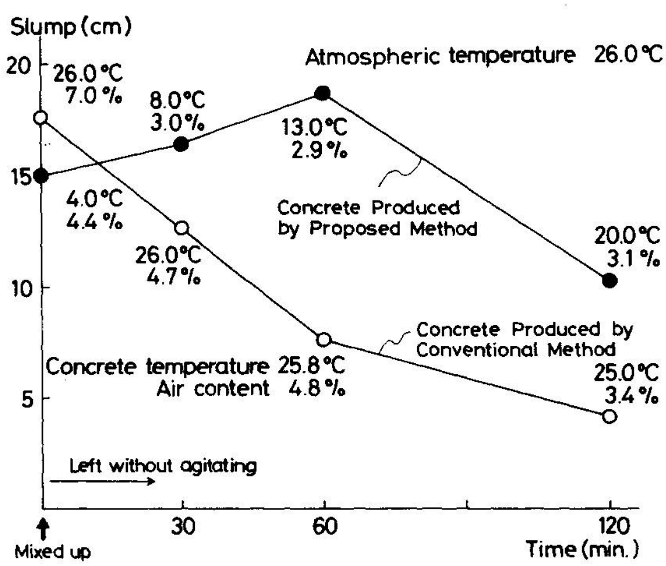
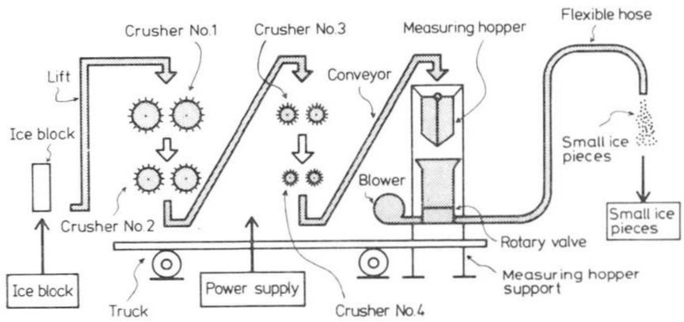


Fig.3 Slump of the concrete produced by proposed method and of conventional concrete



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

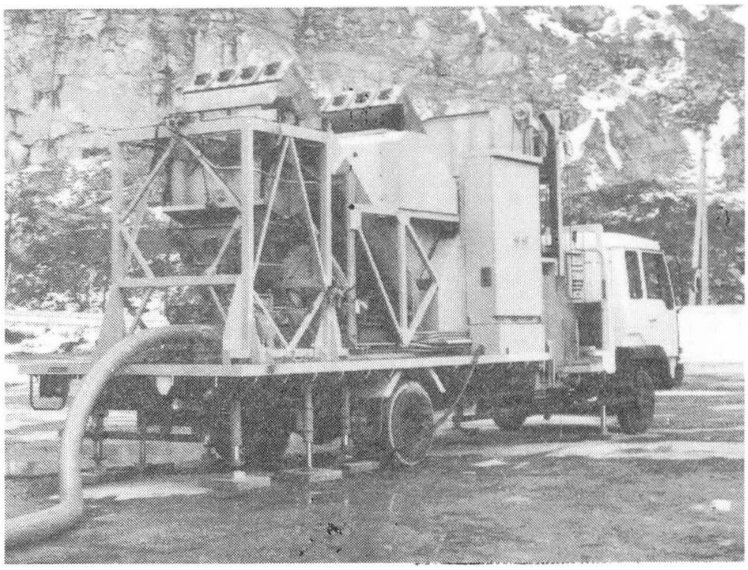


Photo.3
The first experimental plant truck

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.

REFERENCES

1. The Method of Manufacturing Panels for Building Construction. The Official Patent Bulletin of Japan, 1978-5694
2. Control of Cracking in Concrete Structures. Report by ACI Committee 224, ACI Journal, December 1972, pp.717-753.
3. SUZUKI, T., and TAKIGUCHI, K., A New Concrete Production Method Using Small Ice Pieces. Transactions of The Japan Concrete Institute Vol.8, 1986, pp.81-88.
4. SUZUKI, T., TAKIGUCHI, K., and HOTTA, H., Characteristics of Concrete Produced by New Method Using Small Ice Pieces. Transactions of The Japan Concrete Institute Vol.8, 1986, pp.89-96.

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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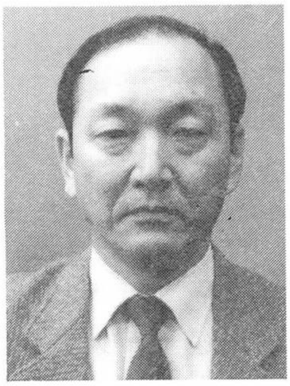

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SUMMARY
This paper outlines a unique method of permeable formwork developed for the purpose of improving surface quality of concrete structures. The major contents of the paper relate to the materials used for the permeable form, the composition of 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

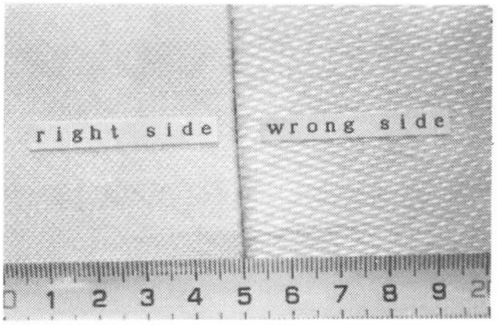

Fig. 1 Texture of double woven cloth

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,

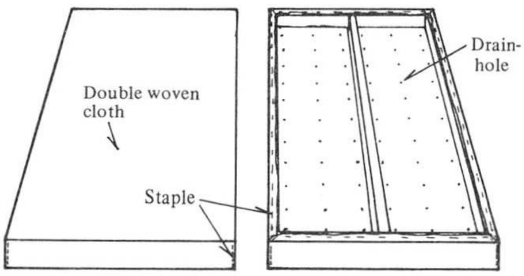

Fig. 2 Permeable form

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.

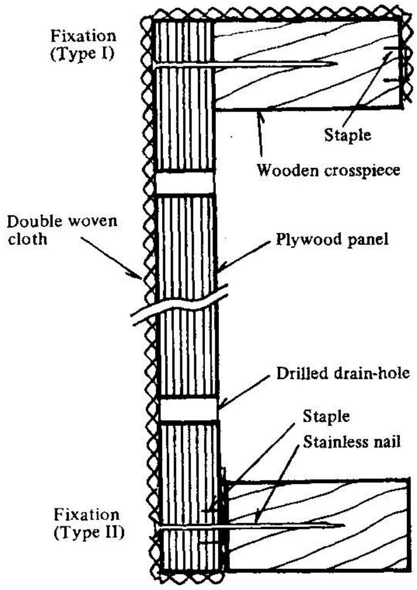

Fig. 3 Cross section of permeable form

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].

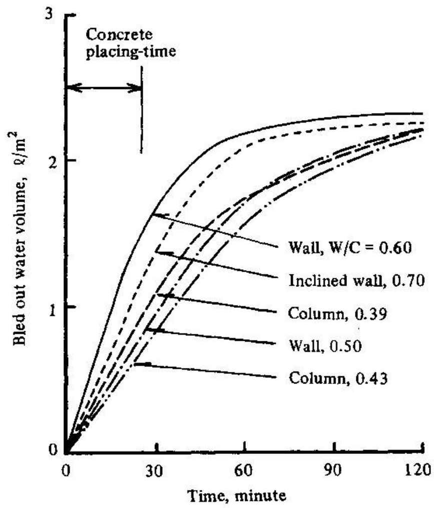

Fig. 4 Water volume bled out from permeable form

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).

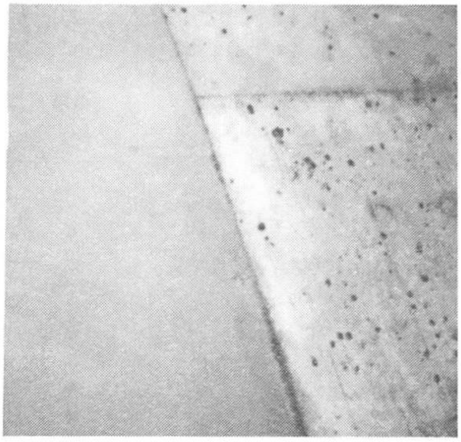

Fig. 5 Slope concrete surface of a dome constructed by permeable form (left) and plywood form (right)

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.

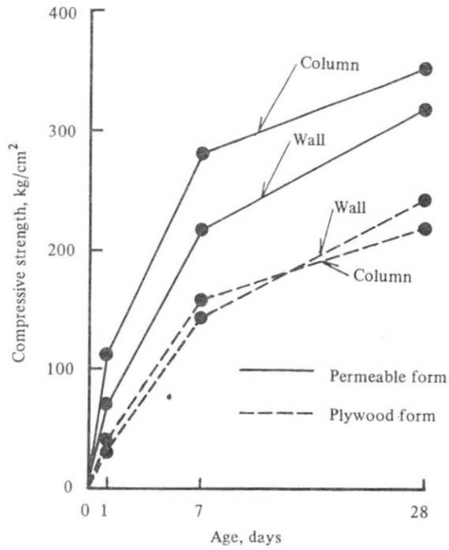

Fig. 6 Comparison of is Schmidt hammer test results

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.

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.

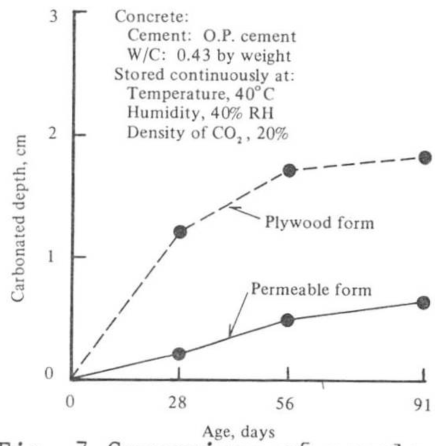

Fig. 7 Comparison of accelerated carbonation test results

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

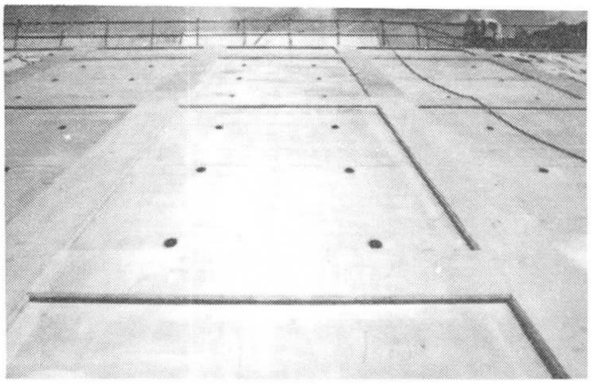

Fig. 8 Inclined retaining wall of Aseishigawa dam (Aomori pref.) [2]

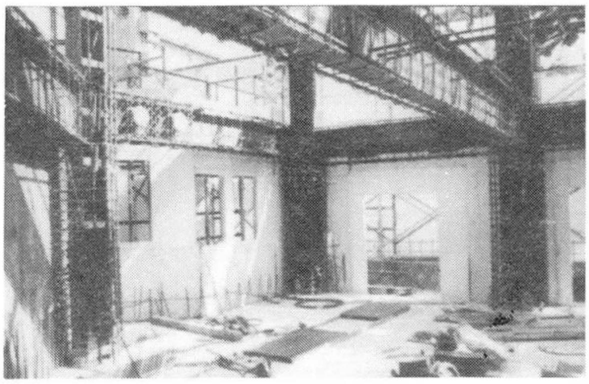

Fig. 9 External wall formwork of Ichikawa Heights bldg. (Chiba pref.)

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.

REFERENCES

1. TANAKA K. et al, A study on improvement of concrete quality using textile-form (Part 1 - 4), Summaries of Technical Papers of Annual meeting of Architectural Institute of Japan, July 1986, pp. 281 - 288. (in Japanese)

2. HORIYA S. et al, Development of "Textile-form method" in Aseishigawa dam, Proceedings of Japan Society of Civil Engineers, vol.373/VI-5, Aug. 1986, pp. 121 - 129. (in Japanese)

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.

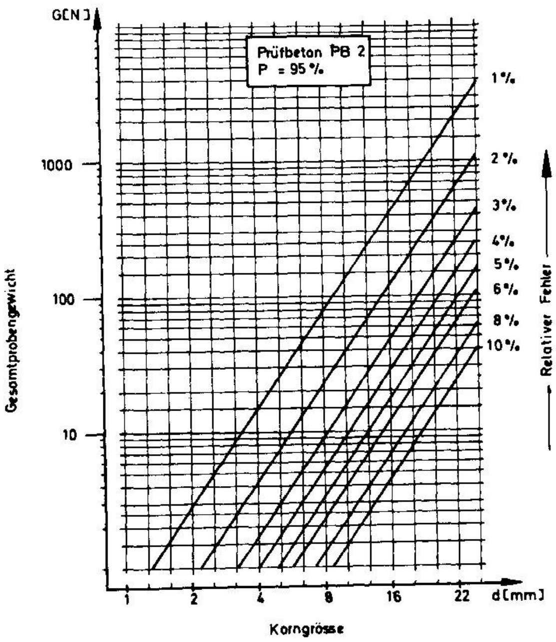

Bild 1 Nomogramm zur Bestimmung des Gesamtprobengewichtes

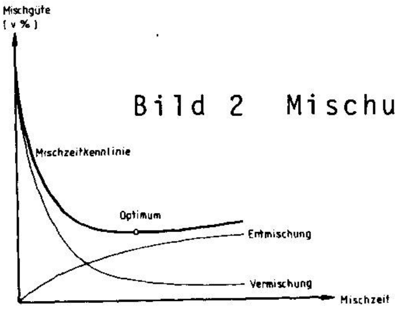

Bild 2 Mischungsverlauf

	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/Z - 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.

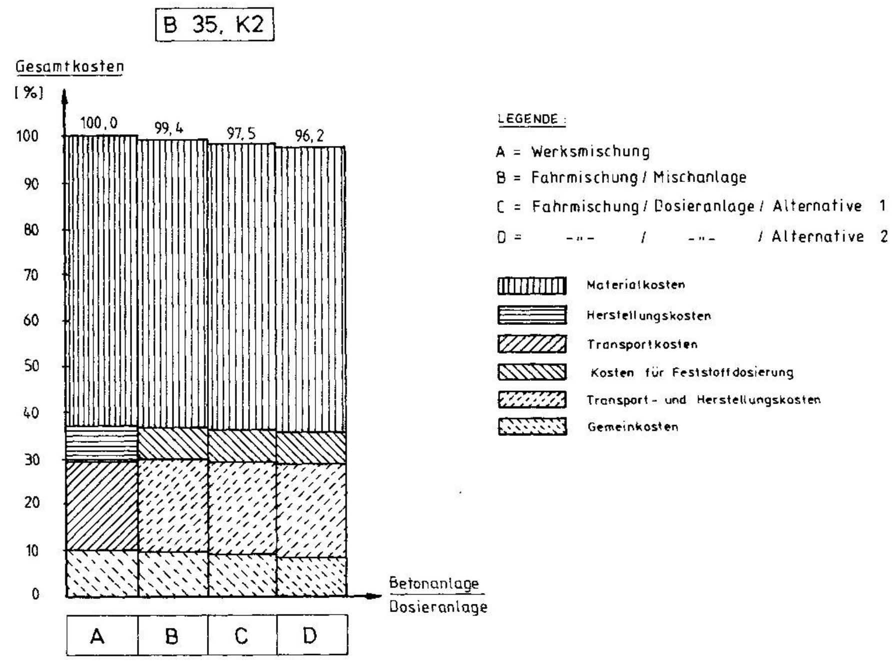

Bild 3 Kostenerfassung für Transportbeton

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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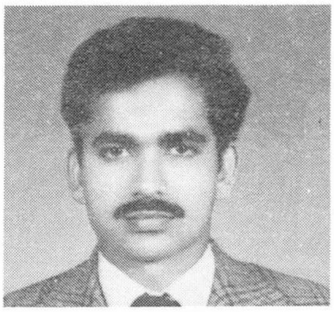

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SUMMARY
In this paper, the possibility of using sulphur concrete in foundations has been examined. The early gain in strength, high strength, low permeability and high electrical resistivity of sulphur concrete are all in favor of its use as a foundation 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°C (293°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°C (340 to 380°F) and then mixing these constituents with modified sulphur cement. A hot homogeneous mixture in the temperature range of 127 to 149°F (260 to 300°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°C (260 to 300°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.

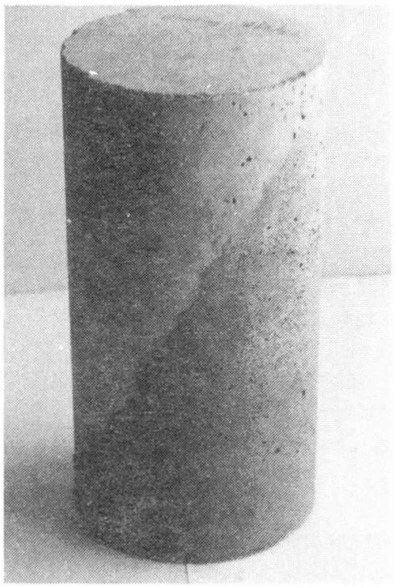

Fig. 1 Composite Cylinder of SC and PCC used for bond test

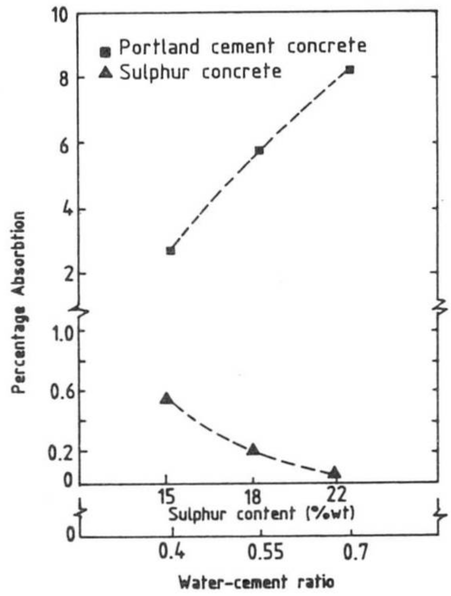

Fig. 2 Absorption of SC and PCC

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.

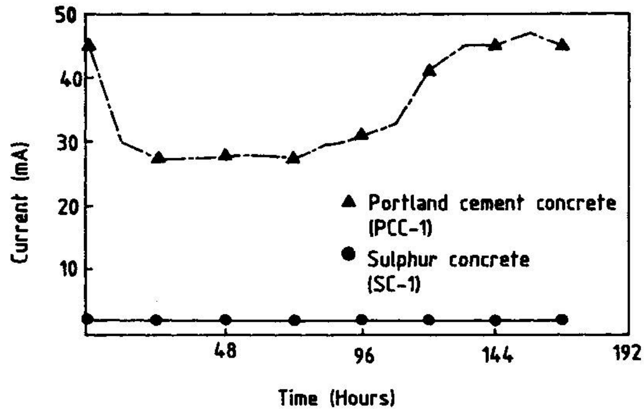

Fig. 3 Variation of current with time in SC and PCC specimens in impressed voltage testing

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/cm² (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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REFERENCES

1. MCBEE, W.C., SULLIVAN, T.A. and JONG, B.W., Modified Sulfur Cements for use in Concretes, Flexible Pavings, Coatings and Grouts. Research Report RI 8547, U.S. Bureau of Mines, 1981, 24 pp.

2. MCBEE, W.C., SULLIVAN, T.A. AND FIKE, H.L., Sulfur Construction Materials. Bulletin 678, U.S. Bureau of Mines, 1985, 31 pp.

3. VROOM, A.H., Sulfur Cements, Process for Making Same and Sulfur Concretes Made Therefrom. U.S. Pat. 4,058,500, November 15, 1977.

4. GILLOT, J.E., JORDAAN, I.J., LOOV, R.E. AND SHRIVE, N.G., Sulphur Concretes, Mortars and the Like, U.S. Pat. 4,188,230, February 12, 1980.

5. PICKARD, S.S., Sulphur Concrete at AMAX Nickel: Project Case History. Sulphur Research and Development, Vol. 7, 1983, pp. 19-24.

6. FOLEY, A.S., Repairing Foundations with Sulphur Concrete. Sulphur Research and Development, Vol. 9, 1985, pp. 4-5.

7. SULPHUR RESEARCH AND DEVELOPMENT, Sulphur Concrete used for Highway Repair. Sulphur Research and Development, Vol. 9, 1985, pp. 6-7.

8. SULLIVAN, T.A., Corrosion Resistant Sulfur Concretes - Design Manual. The Sulphur Institute, Washington, D.C., U.S.A., 1982, 18 pp.

9. MUIR, D.R., Sulphur Supply/Demand and its Relevance to New Sulphur Products: An Examination of their Economic Viability. Proceedings of International Conference on Sulphur in Construction, CANMET, Energy, Mines and Resources, Ottawa, Canada, 1978, pp. 1-16.

10. AL-TAYYIB, A.J. and KHAN, M.S., Testing Techniques for Reinforcement Corrosion in Portland Cement Concrete - A Review. Corrosion (submitted for publication).

11. STRATFULL, R.F., How Chlorides Affect Concrete Used with Reinforcing Steel, Material Protection, Vol. 7, No. 3, March 1968, pp. 29-34.

12. AL-TAYYIB, A.J., TEWFIK, M.F., BALUCH, M.H., AZAD, A.K. and KHAN, M.S., Effect of Filler Type on the Durability of Sulphur-Sand Composites in the Arabian Gulf Countries. The Arabian Journal for Science and Engineering, Vol. 11, No. 3, July 1986, pp. 225-234.

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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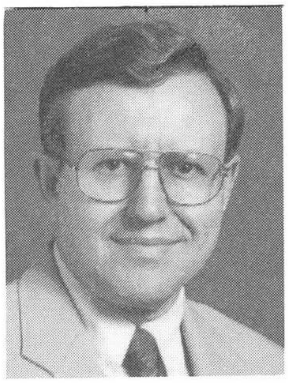

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SUMMARY
The use of concrete-polymer materials for bridge deck construction and repair has increased significantly. Polymer concrete repairs and overlays have proven to be fast, durable, and cost effective. The use of polymers for sealing cracks has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

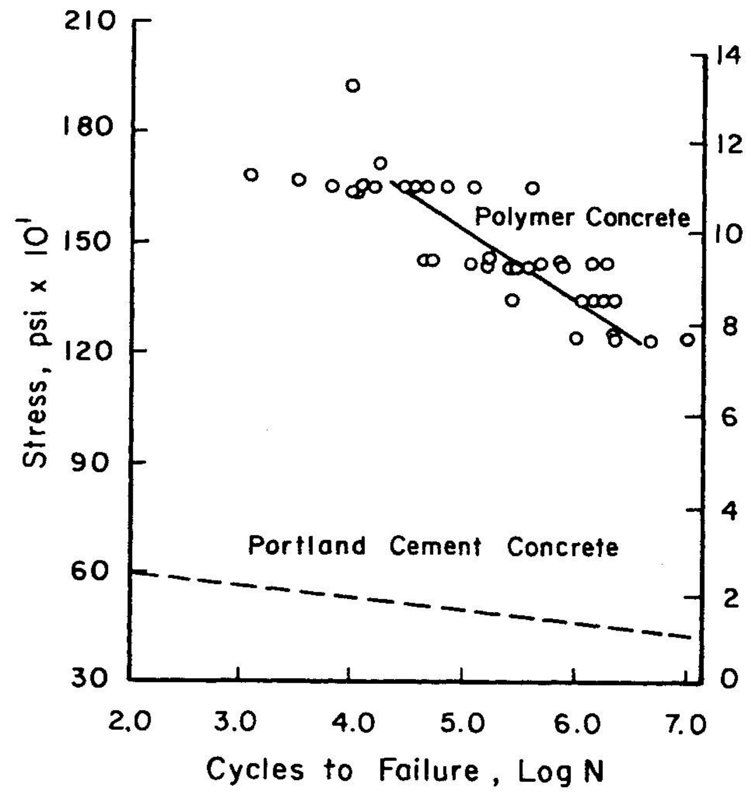

Fig. 1: Comparison of Fatigue Strength of Beams for R = 0.05 and R = 0.25

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.

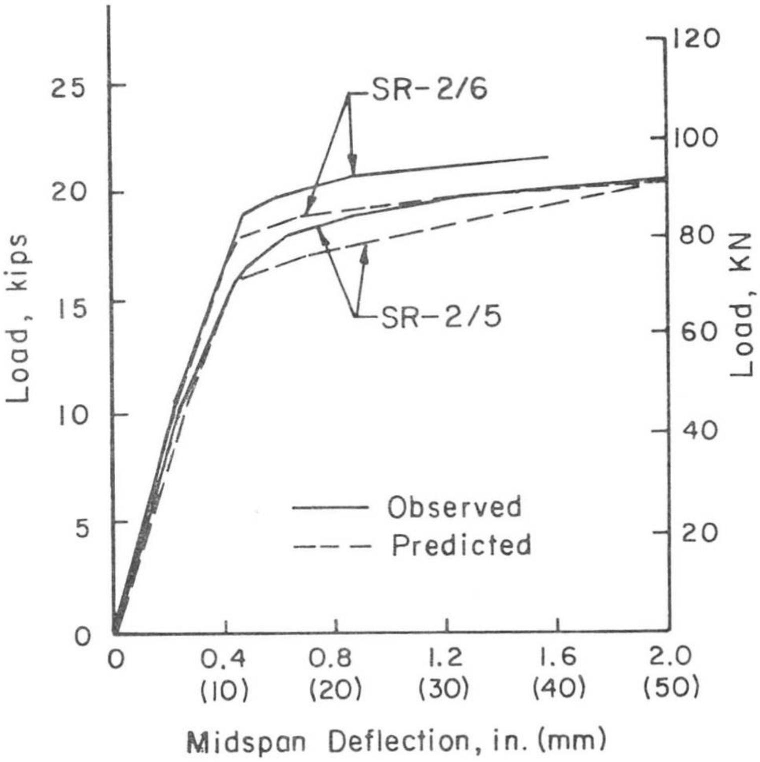

Fig. 2: Load-Deflection Responses for Beams (SR-2/5 has 4.0% steel and SR2/6 has 5.6% steel)

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.

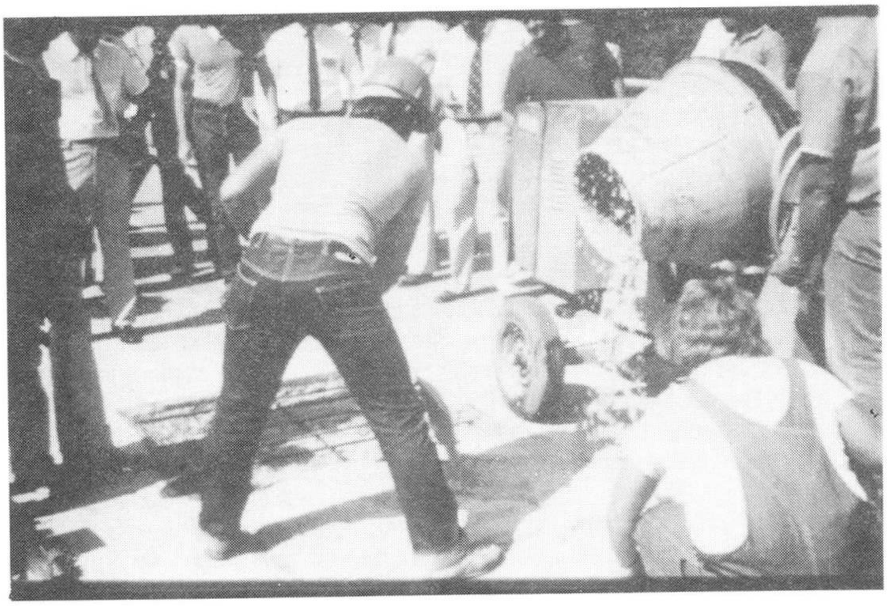

Fig. 3 Placing PC into Repair Area on Bridge

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].

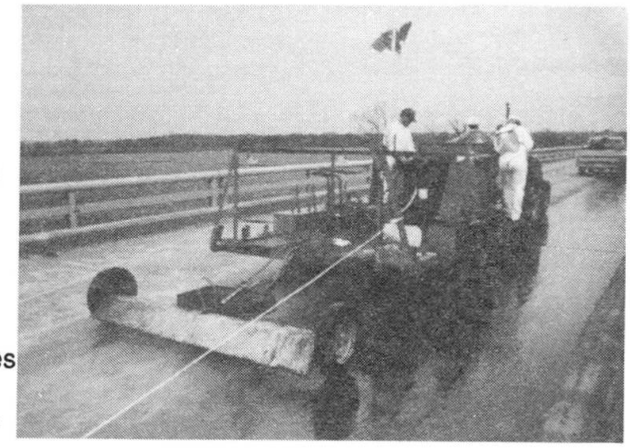

Fig. 4. Polyester Resin Applied by Spray Bar for Construction of Overlay

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].

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.

REFERENCES

1. Kuhlman, L. A., The Application and Use of Styrene-Butadiene Latex Modified Concrete, ACI Journal (in press), presented at the 1986 ACI Fall Convention, Baltimore, MD.
2. Hsu, M. and Fowler, D. W., Creep and Fatigue of Polymer Concrete, American Concrete Institute, SP-89, 1985.
3. Fowler, D. W., and Hsu, H-T., Static and Cyclic Behavior of Polymer Concrete Beams, Proceedings of the Fourth International Congress on Polymers in Concrete, September, 1984, Darmstadt.
4. Technical Manual, Adhesive Engineering Co., San Carlos, CA., May, 1983.
5. Sprinkel, M. M., Polymer Concrete Overlay on the Big Swan Creek Bridge, Final Report, Virginia Highway and Transportation Research Council, VHTRC 86-R37, June 1986.
6. Scarpinato, Emanuel S., "Epoxy Bridge Deck Overlays and the Brooklyn Bridge," Polymers in Concrete, ICPIC '84, Darmstadt, pp. 351 - 358.
7. "Alcoa Bridge Deck System," Aluminum Company of America, Pittsburgh, PA.
8. Fontana, Jack J., "Electrically Conductive Polymer Concrete Coatings," SP-99, American Concrete Institute, 1987.
9. Fowler, D. W., Use of High Molecular Weight Methacrylate for Repairing Cracks in Concrete, Adhesion Between Polymers and Concrete, RILEM Symposium, 1986, Chapmand and Hall, New York.

High Performance Fiber Reinforced Cement Composites

Bétons de fibres à haute performance

Hohe Festigkeit Faserverstärkter Betone

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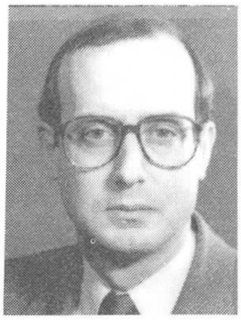

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SUMMARY
A brief synthesis of the behavior and mechanics of fiber reinforced cement based composites in tension, compression, and flexure is presented with particular emphasis on their stress-strain, stress-elongation, or load-deflection relationships. 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

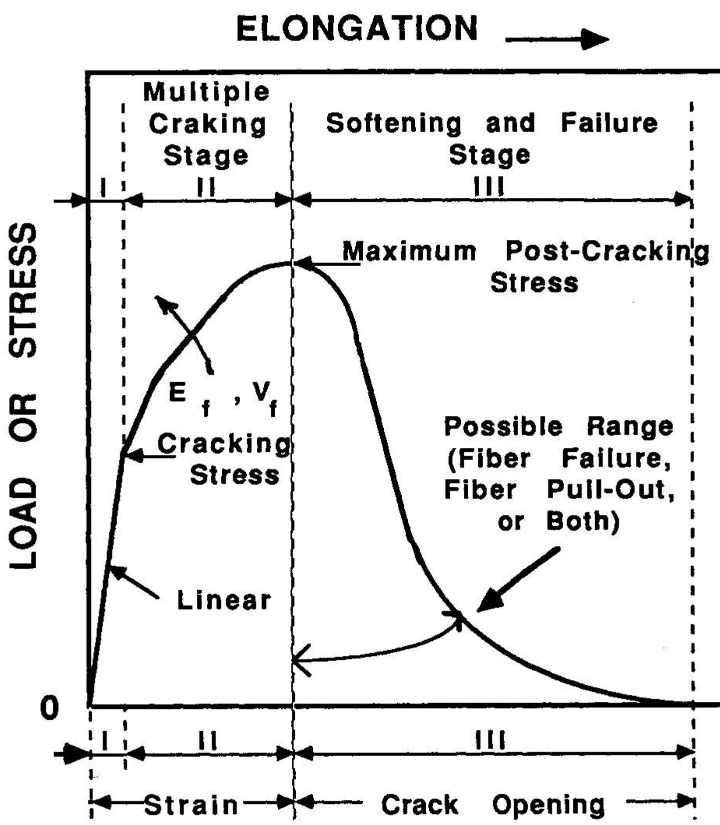

Figure 1. Typical Load Elongation Response in Tension of a High Performance FRC Composite Such as SIFCON.

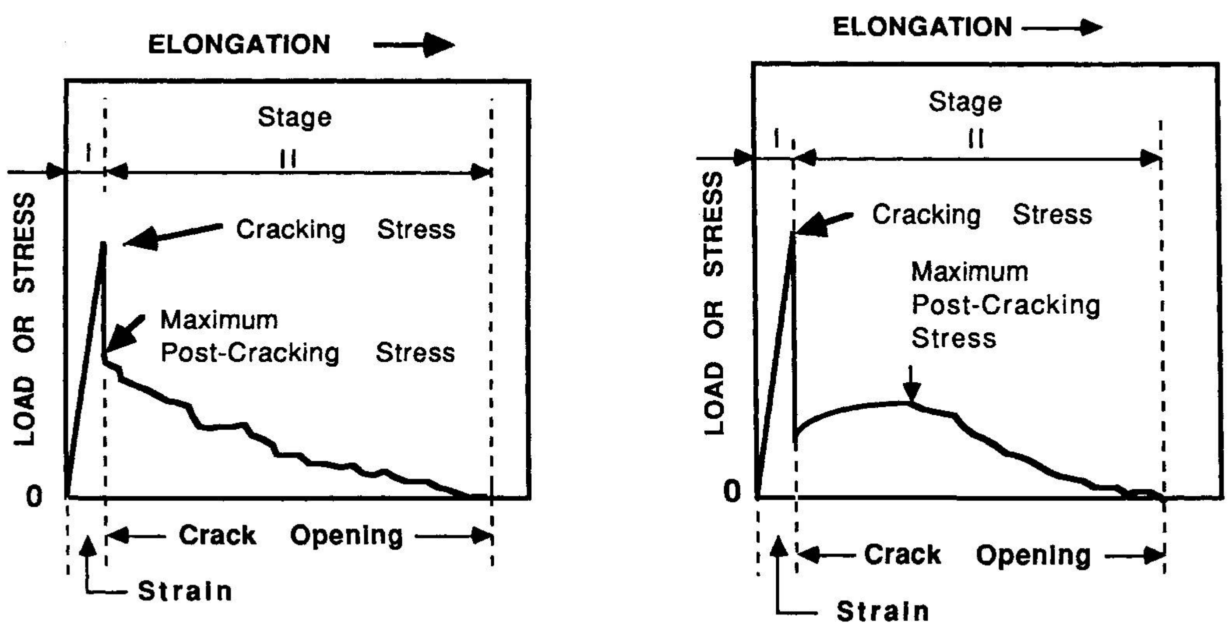

Figure 2. Typical Load Elongation Response in Tension of Fiber Reinforced Concrete: a) Using Premixed Steel Fibers, and b) Using Premixed Polypropylene Fibers.

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]:

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

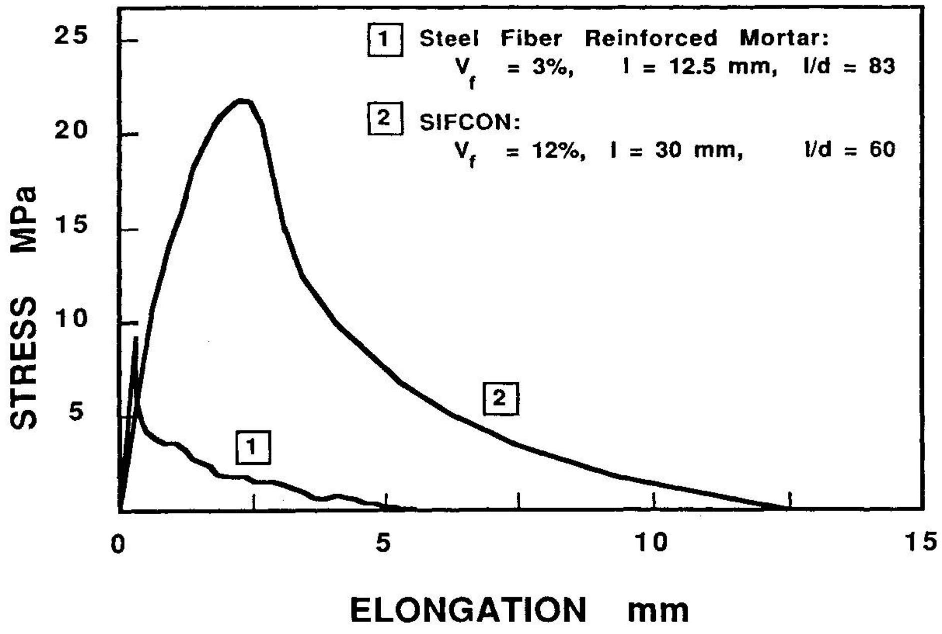

Fig.3 Typical Stress-Elongation Curves of Conventional Fiber Reinforced Mortar and SIFCON.

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:

$$E_c = E_m V_m + E_f V_f \qquad (2)$$

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:

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

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:

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

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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REFERENCES

1. NAAMAN A.E., MOAVENZADEH, F., and McGARRY, F., Probalistic Analysis of Fiber Reinforced Concrete. ASCE Journal of the Engineering Mechanics Division, April 1974.

2. VISALVANICH K., and NAAMAN A.E., A Fracture Model For Fiber Reinforced Concrete. ACI Journal, March-April 1983.

3. NAMMUR G., and NAAMAN A.E., Strain Rate Effects on Tensile Properties of Fiber Reinforced Concrete. Materials Research Society, Symposium Volume 64, Pittsburgh, 1986.

4. FANELLA D., and NAAMAN A.E., Stress-Strain Properties of Fiber Reinforced Mortar in Compression. ACI Journal, July-August 1985.

5. NAAMAN A.E., and HOMRICH J., Properties of High Strength Fiber Reinforced Concrete. ACI Special Publication SP-87, 1985.

6. OTTER D., and NAAMAN A.E., Steel Fiber Reinforced Concrete Under Static and Cyclic Compressive Loading. Proceedings of the Third International Symposium on Developments in Fiber Reinforced Cement and Concrete, University of Sheffield, July 1986.

7. ABSI E., and NAAMAN A.E., Modele Rheologique Pour les Betons de Fibres. Proceedings of the Third International Symposium on Developments in Fiber Reinforced Cement and Concrete, University of Sheffield, July 1986.

8. DELVASTO S., and NAAMAN A.E., Effect of Pressure After Casting on High Strength Fiber Reinforced Mortar. International Journal of Cement Composites and Lightweigth Concrete, August 1986.

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

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

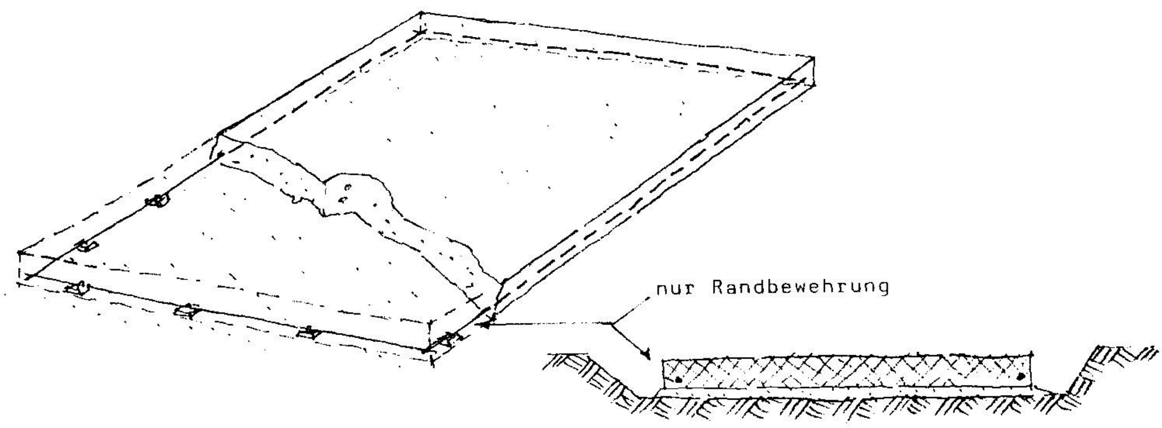
Zugabe der Fasern:	Fasern entweder direkt durch die Reinigungsöffnung nach der Zementzugabe in den Mischer geben oder, was meist einfacher ist, zusammen mit Kies oder Sand in den Waagebehälter schütten.
Empfehlung:	Eine Trockenmischzeit von ca. 30 Sekunden von Kies, Sand, Zement oder Fasern verbessert 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

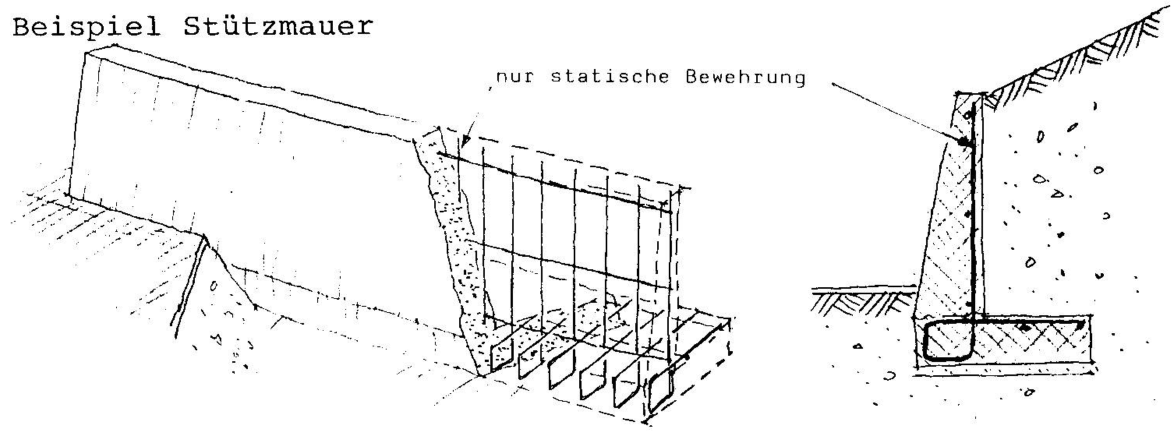
4.1 Ortsbeton

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.

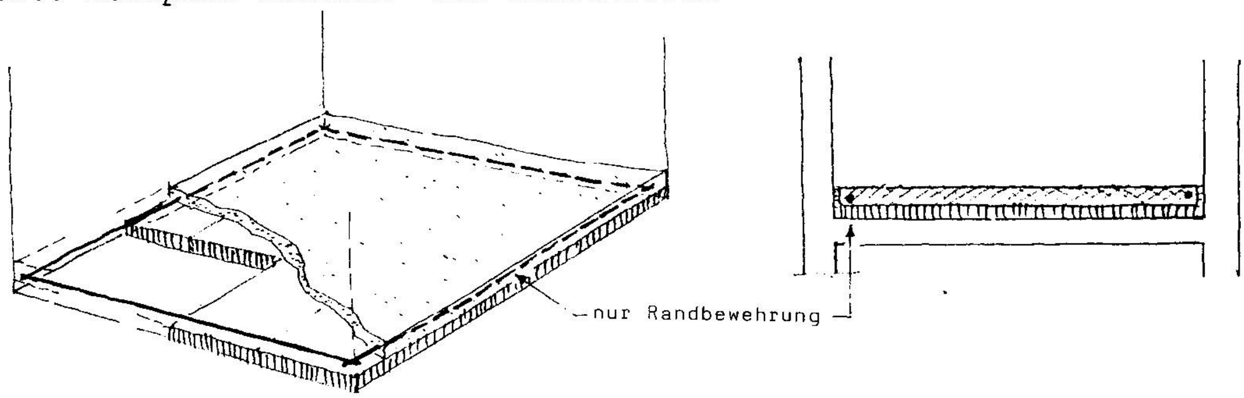
4.1.1 Beispiel Bodenplatte



4.1.2 Beispiel Stützmauer



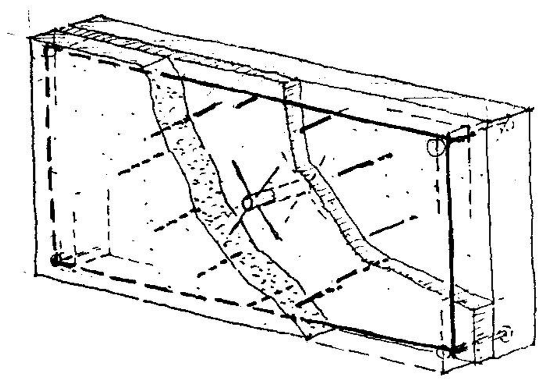
4.1.3 Beispiel Gefälls- und Schutzbeton

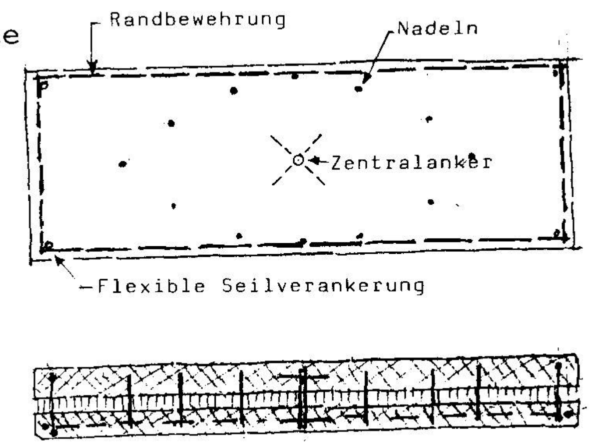


4.2 Vorfabrikation

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.

8. LITERATURVERZEICHNIS

[1] Staatliche Prüfanstalt EMPA "Untersuchungen der Frühschwindrissbewehrung durch Forta-Fibre Kunststoffaserarmierung"

- Prof. R.F. Zollo "Collated Fibrillated Poylpropylene Fibres in FRC", American Concrete Institute, 17. Sept. 1980
- L. Vironnaud "The Early Stages of Concrete", National Research Concil of Canada, Ottawa 1962
- Rud. Enzler, Bauingenieur "Neue Erkenntnisse in der Betontechnologie, März 1977
- Schweiz. Bauzeitung "Kunststoff-Faserarmierung von Beton", Nr. 44 vom 30. Okt. 1975
- T. Steiner, Bauingenieur "Forta-Fibre, Die Kunststoffaser für resistenten Beton", Romanshorn, 26. März 1985

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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François de Larrard prépare actuellement une thèse sur la formulation et les applications des bétons à très hautes performances.

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.

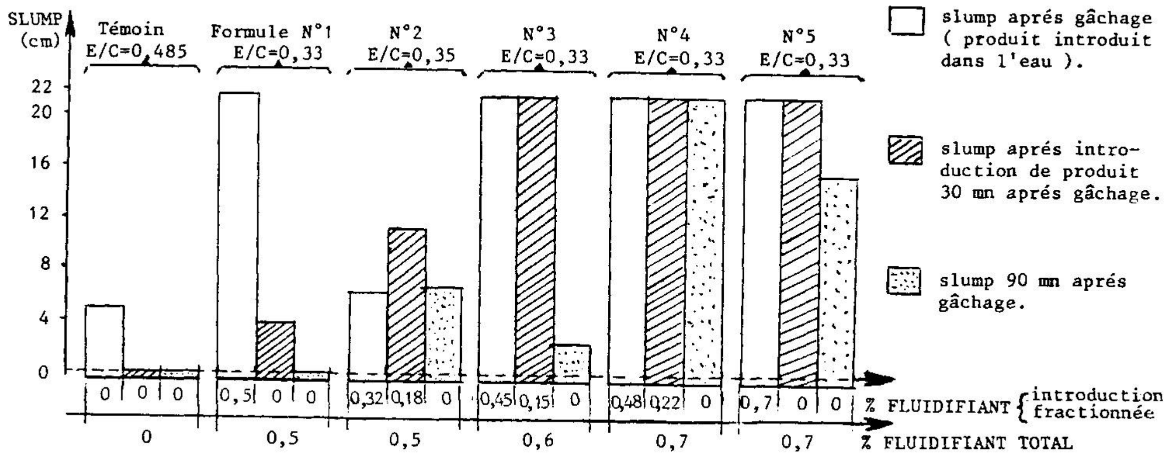
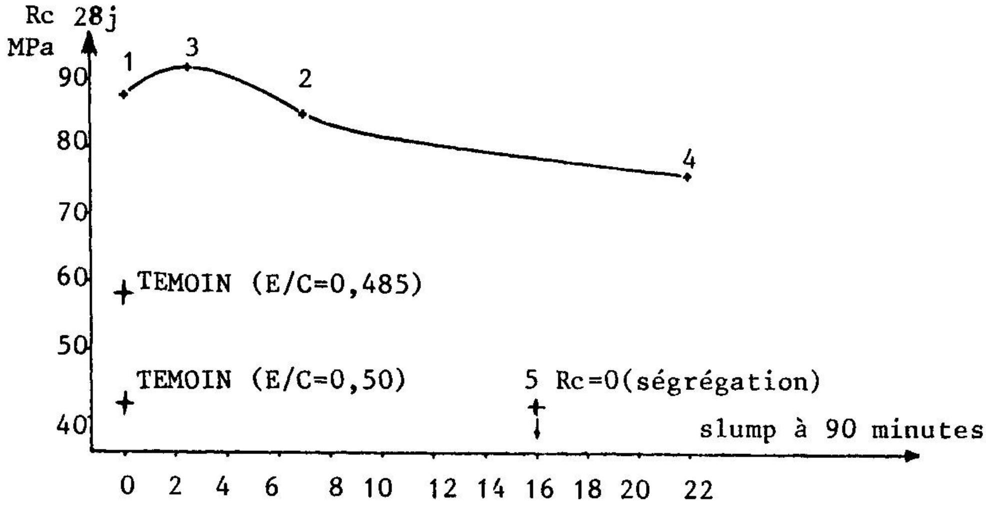


Figure 1 : Affaissement au cône d'Abrams (slump) d'un béton fluidifié en fonction du dosage en fluidifiant, de son mode d'introduction et du temps.



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 à haute turbulence : 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.

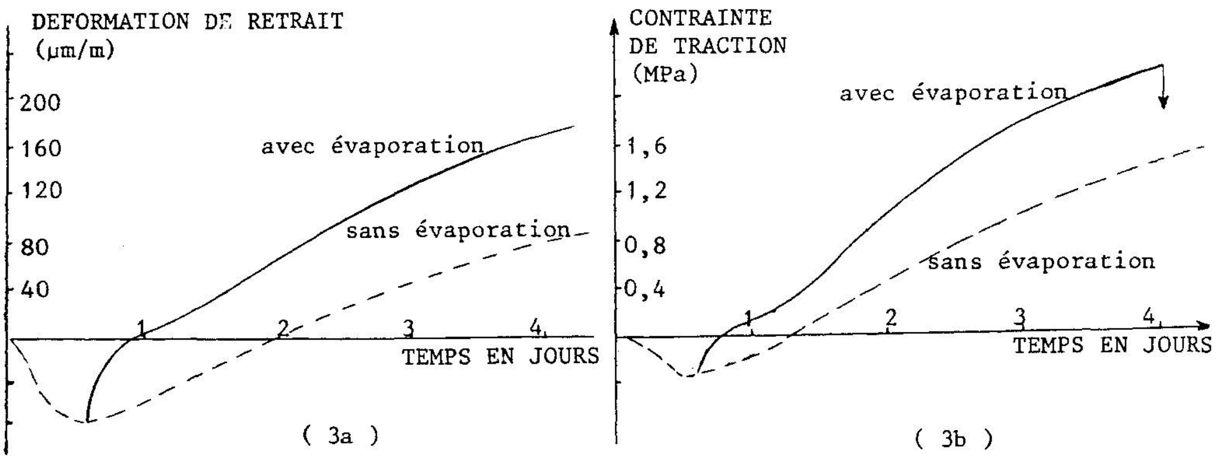


Figure 3 : Evolution des déformations libres et des contraintes dans l'essai au banc de fissuration pour un béton traditionnel avec évaporation et sans évaporation.

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).

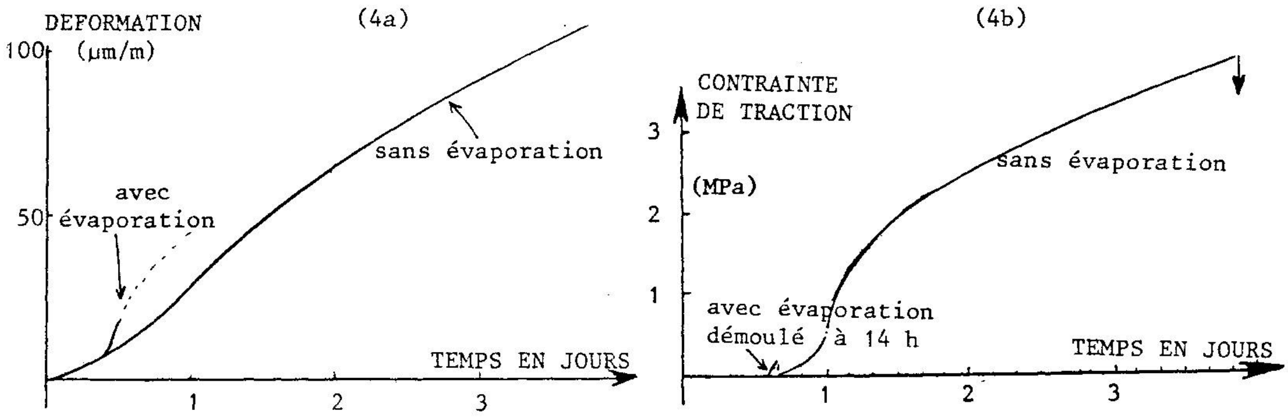


Figure 4 : Evolution des déformations libres et des contraintes dans l'essai au banc de fissuration pour un béton à très hautes performances avec fumées de silice, avec évaporation et sans évaporation.

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.

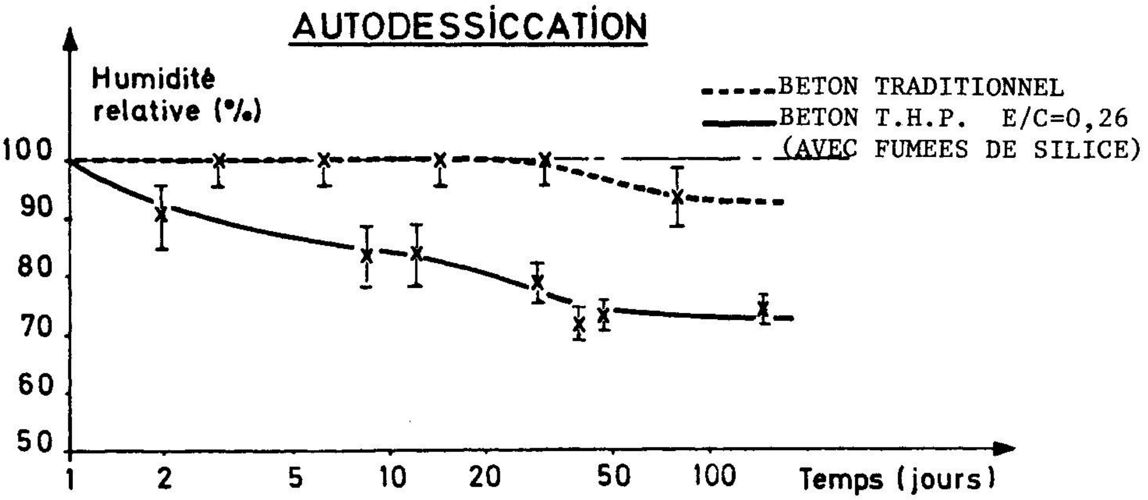
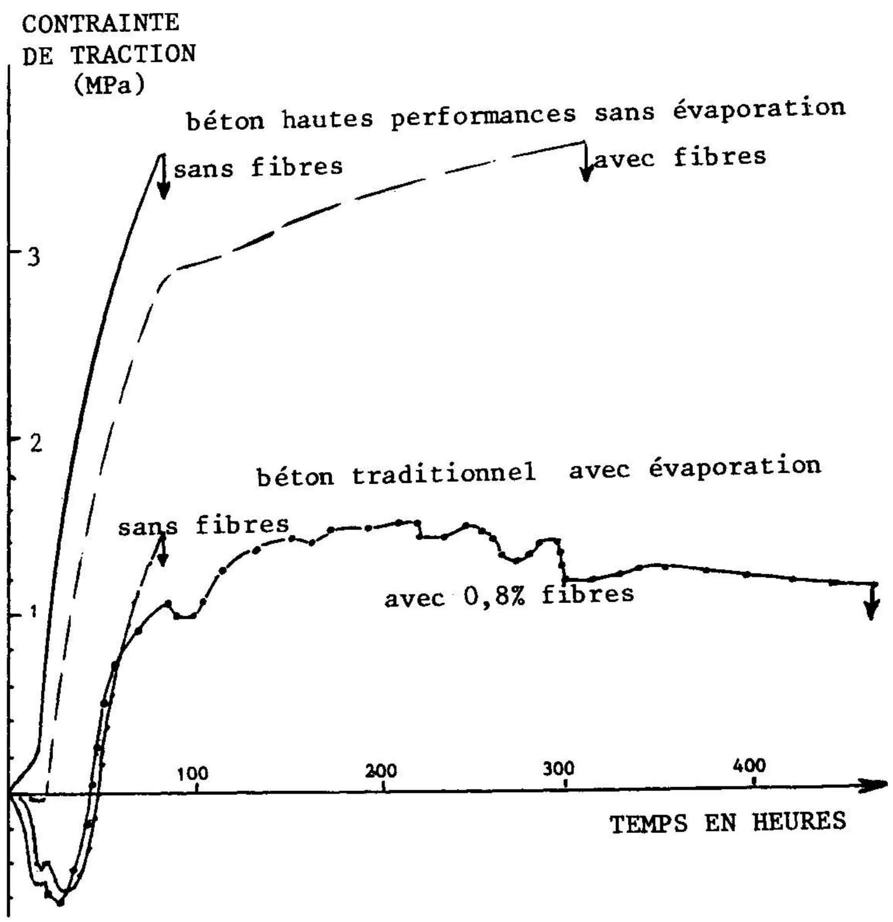


Figure 5 : Evolution de l'humidité interne avec le temps d'hydratation pour un béton traditionnel et pour un béton à très hautes performances avec fumées de silice (éprouvettes protégées de l'évaporation).



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.

BIBLIOBRAPHIE

1. CANMET-ACI (1986) Proceedings Second International Conference en Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete - Madrid. 2 vol.
2. A.M. PAILLERE, Ph. BRIQUET (1980) - « Influence des résines de synthèse fluidifiantes sur la rhéologie et la déformation des pâtes de ciment avant et en cours de prise » - 7e Congrès International sur la Chimie des Ciments - Paris.
3. M. BUIL (1979) « Contribution à l'étude du retrait de la pâte de ciment durcissante ». Rapport de recherche LCPC n° 92, Laboratoire Central des Ponts et Chaussées, Paris.
4. A.M. PAILLERE et J.J. SERRANO (1976) « Appareil d'étude de la fissuration du béton ». Bull. Liaison Labo. P. et Ch. n° 83, pp. 29-38.
5. A.M. PAILLERE, M. BUIL et J.J. SERRANO (1987) « Durabilité du béton à très hautes performances : incidence du retrait d'hydratation sur la fissuration au jeune âge », à paraître, Communication au 1er Congrès International « de la Science des Matériaux au Génie des Matériaux de Construction ». Versailles, 7 - 11 sept. 1987.

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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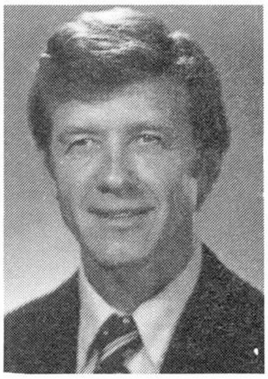

The author received his BS degree from Stanford University, MS from Cornell, and PhD from the University of California. Prior to joining the faculty at Cornell he was engaged in professional practice. He is the author of two texts on concrete design, as well as numerous technical papers.

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.

REFERENCES

1. CARRASQUILLO R.L., SLATE F.O., and NILSON A.H., Microcracking and Behavior of High-Strength Concrete Subject to Short-Term Loading. ACI Journal, May/June 1981.

2. NGAB A.S., SLATE F.O., and NILSON A.H., Microcracking and Time-Dependent Strains in High Strength Concrete. ACI Journal, July/August 1981.

3. SMADI M.M., SLATE F.O., and NILSON A.H., High-, Medium-, and Low-Strength Concrete Subject to Sustained Overloads -- Strains, Strengths, and Failure Mechanisms. ACI Journal, September/October 1985.

4. CARRASQUILLO R.L., NILSON A.H., and SLATE, F.O., Properties of High-Strength Concrete Subject to Short-Term Loads, ACI Journal, May/June 1981.

5. SLATE F.O., NILSON A.H., and MARTINEZ S., Mechanical Properties of High-Strength Lightweight Concrete, ACI Journal, July/August 1986.

6. NGAB A.S., NILSON A.H., and SLATE F.O., Shrinkage and Creep of High-Strength Concrete, ACI Journal, July/August 1981.

7. PASTOR J.A., NILSON A.H., and SLATE F.O., Behavior of High-Strength Concrete Beams, Report 84-3, Dept. of Structural Engg, Cornell Univ., February 1984.

8. ELZANATY A.H., NILSON A.H., and SLATE F.O., Shear Capacity of Reinforced Concrete Beams Using High-Strength Concrete, ACI Journal, March/April 1986.

9. ELZANATY A.H., NILSON A.H., and SLATE F.O., Shear Capacity of Prestressed Concrete Beams Using High-Strength Concrete, ACI Journal, May/June 1986.

10. LUEBKEMAN C.H., NILSON A.H., and SLATE F.O., Sustained Load Deflection of High-Strength Concrete Beams, Report 85-2, Dept. of Structural Engg., Cornell Univ., February 1985.

11. MARTINEZ S., NILSON A.H., and SLATE F.O., Spirally Reinforced High-Strength Concrete Columns, ACI Journal, September/October 1984.

12. NILSON A.H., Design Implications of Current Research on High-Strength Concrete, Special Publication SP-87: High-Strength Concrete, Amer. Conc. Inst., 1985.

F˙ Path of cracks

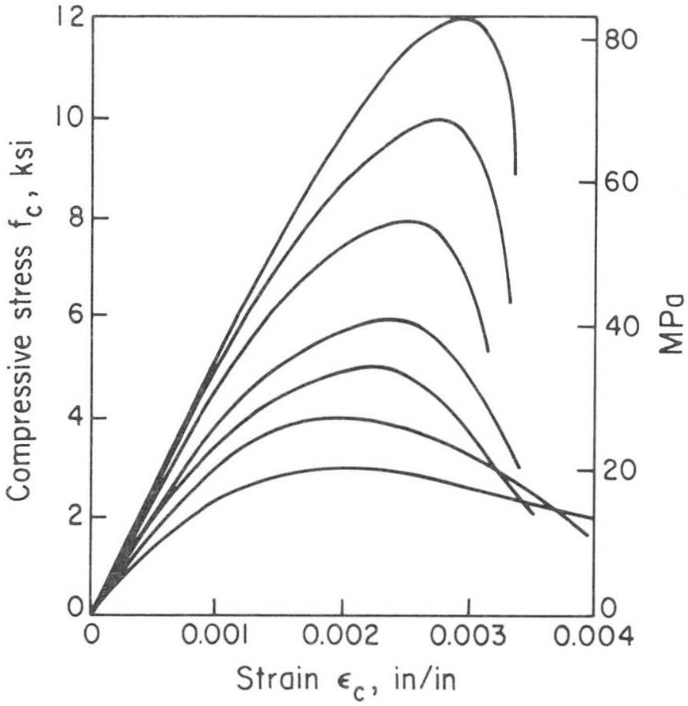

Fig. 2 Typical compressive stress-strain curves

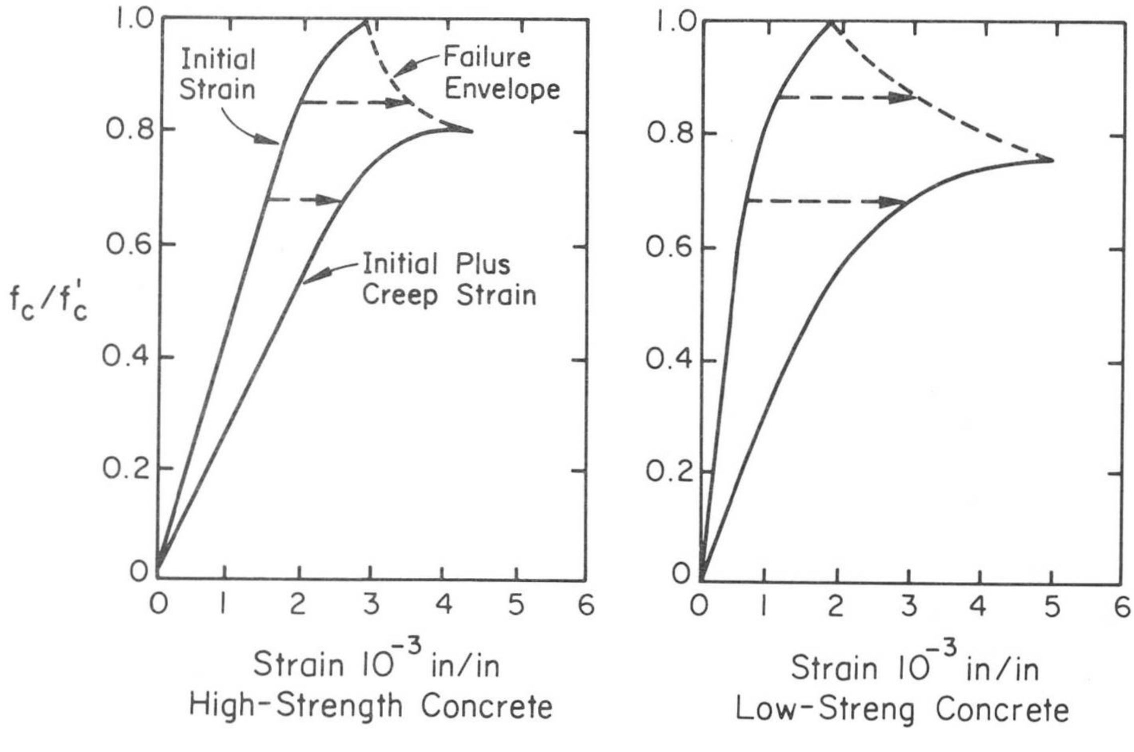

Fig. 3 Sustained load stress strain curves

Mechanical Properties of High Strength Concrete

Propriétés mécaniques du béton à haute résistance

Mechanische Eigenschaften vom hochfesten Beton

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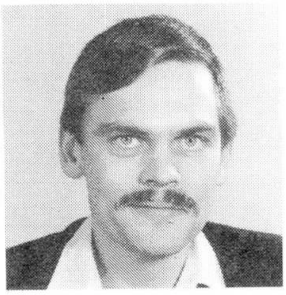

SUMMARY
This report deals with experimental data on high strength concrete. Compressive stress-strain relationship, ductility, lap splice strength, and fire resistance are discussed.

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:

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

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

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

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

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

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

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.

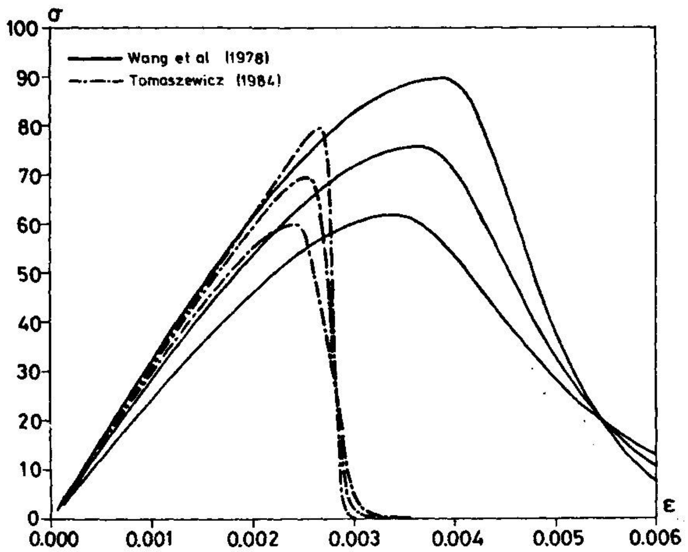

Fig. 1: Stress-strain curves from uniaxial compression tests.

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.

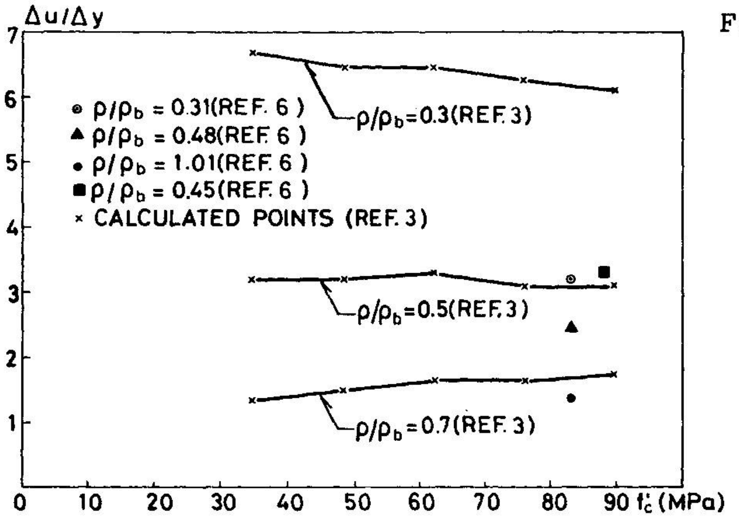

Fig. 2: Relationship between deflection ductility and compressive concrete strength.

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.

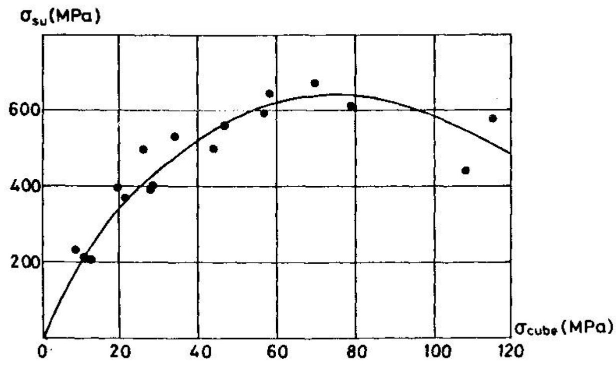

Fig. 3: Relationship between compressive concrete strength and splice strength represented by σ_{su}.

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).

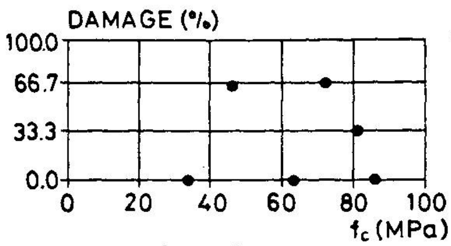

Fig. 4a.

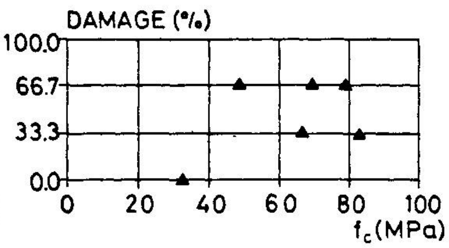

Fig. 4b.

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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REFERENCES

1. WANG, P.T., SHAH, S.P., and NAAMAN, A.E., "Stress-Strain Curves of Normal and Lightweight Concrete in Compression", ACI Journal, Proceedings V. 75, No. 11, November 1978, pp. 603-611.
2. TOMASZEWICZ, A., "Betongens Arbeidsdiagram", SINTEF, STF65, A84065, Trondheim, 1984.
3. AHMAD, S.H. and SHAH, S.P., "Structural Properties of High Strength Concrete and its Implications for Precast Prestressed Concrete", PCI Journal, Nov.- Dec. 1985, pp. 92-119.
4. ACI Committee 363, "State-of-the-Art Report on High Strength Concrete", ACI Journal, Proceedings V. 81, No. 4, July-August 1984, pp. 364-411.
5. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)", American Concrete Institute, Detroit, Michigan, 1983.
6. BERNHARDT, O.J., "Forsøk med bjelker i høyfast betong", STF65, A850211, SINTEF, Trondheim, 1985.
7. TEPFERS, R., "A Theory of Bond Applied to Overlapped Tensile Reinforcement Splices for Deformed Bars", Publication 73:2, Division of Concrete Structures, Chalmers Tekniska Högskola, Göteborg, 1973.
8. HERTZ, K., "Heat-Induced Explosion of Dense Concretes", Report 166, Institute of Building Design, Technical University of Denmark, Lyngby 1984.

Béton à hautes performances

Hochfester Beton

High-Strength Concrete

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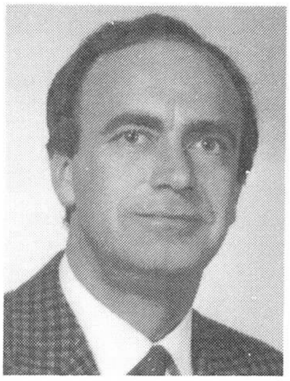


Gaël Cadoret, né en 1949, obtient son diplôme d'Ingénieur à l'INSA de Lyon en 1972. En 1976, il reçoit le titre de Docteur Ingénieur de l'Université Pierre et Marie Curie. Après avoir exercé une activité de laboratoire et de Contrôle sur chantier, il intègre Bouygues en 1983 où il est actuellement Chef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

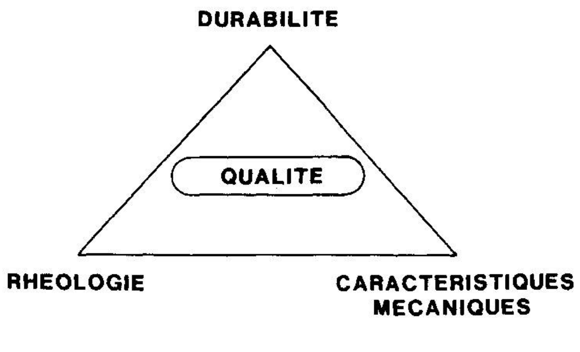


FIGURE 1 - LA QUALITE DU MATERIAU

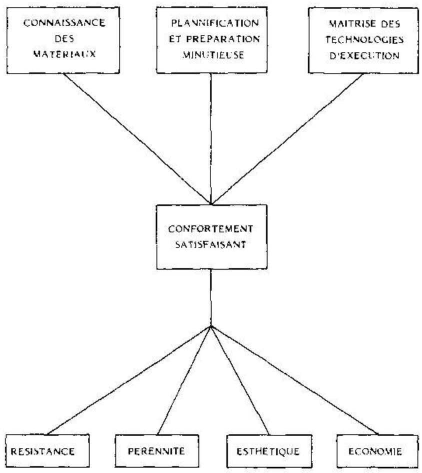


Fig. 2 - La qualité de la réalisation

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.

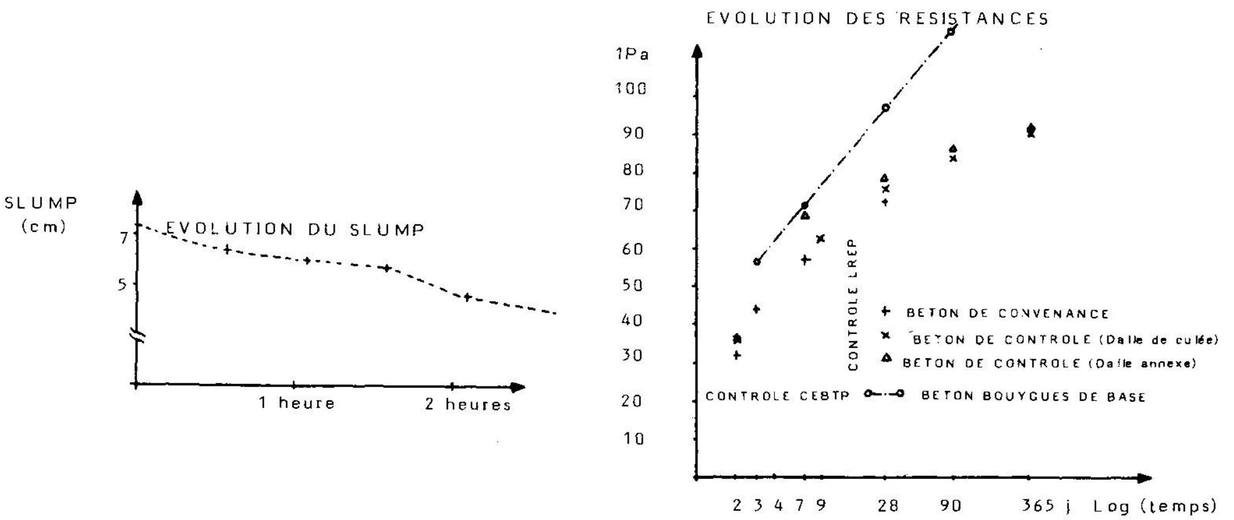


Fig. 3 : CHANTIER PLM A86 - Suivi du béton

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) :

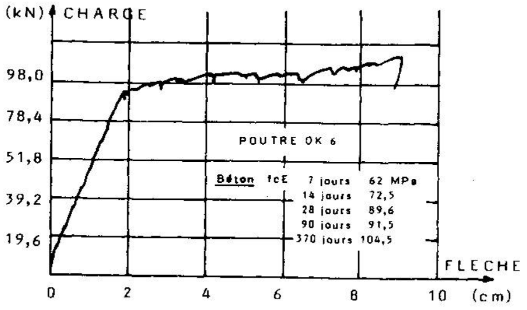
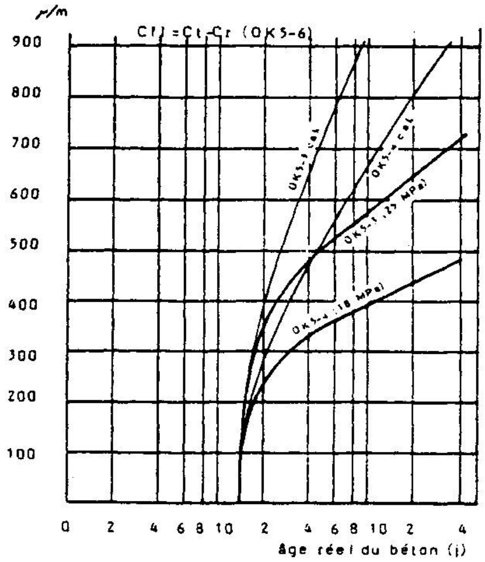


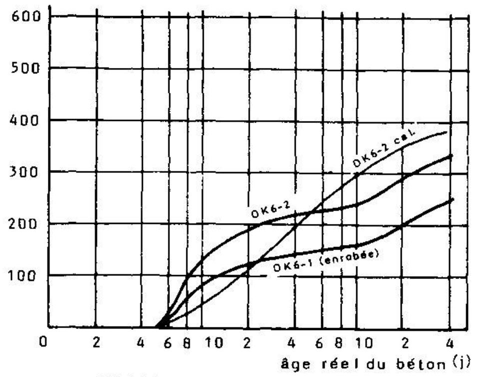
FIGURE 4 - DIAGRAMME CHARGE FLECHE AU CENTRE POUR OK 6



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

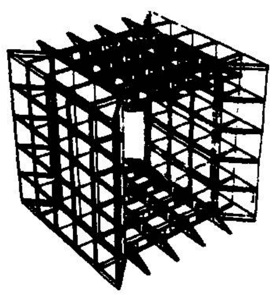


FIGURE 7

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.

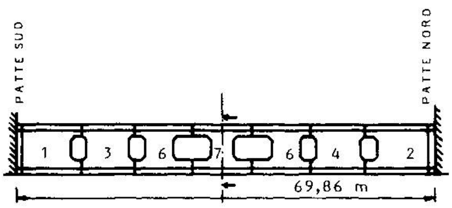
Pour les poutres principales et leurs noeuds ainsi que pour les chapiteaux des 12 appuis, il était nécessaire d'obtenir un béton de résistance caractéristique de 50 MPa.

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

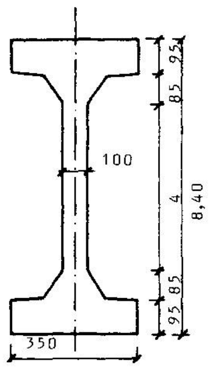


FIGURE 8

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.

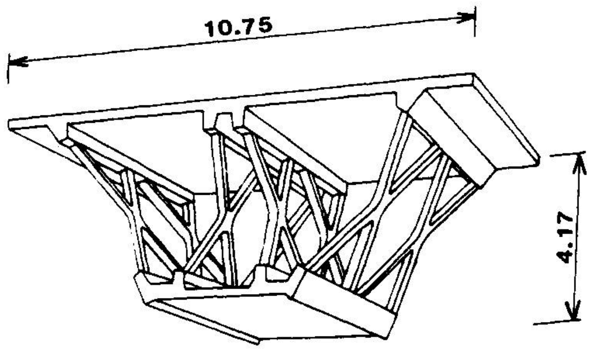


FIGURE 9

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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Jaime Moreno received his Bachelor of Science in engineering from the University of the Andes in Bogota, Colombia, and his master's degree in material engineering from the University of Illinois. His present responsibility is the optimization of concrete structures.

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.

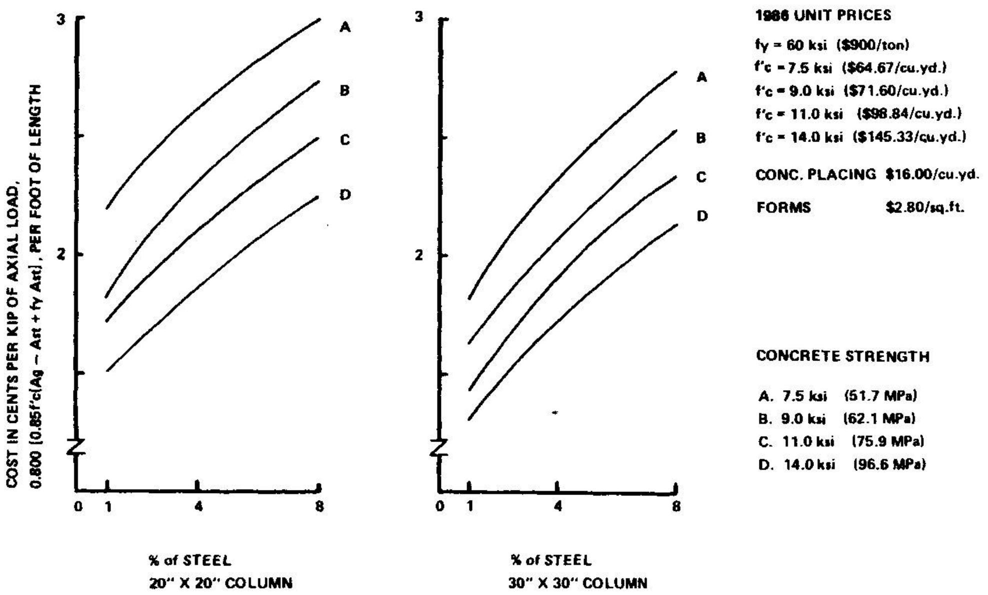
Recent developments in concrete technology have allowed the ready-mix industry to improve and optimize the mixtures shown in this table. The new mixtures for high-strength concrete are proprietary and this type of concrete is being sold as perfomance concrete.

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

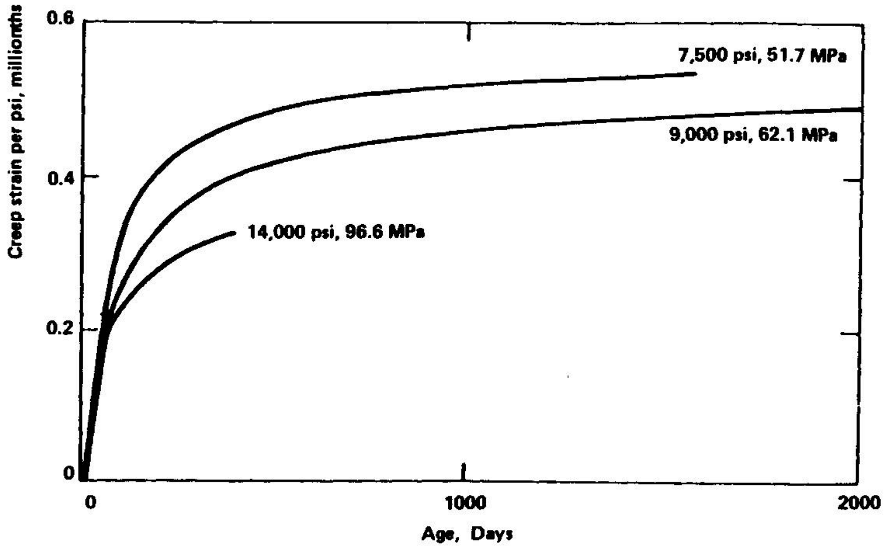


FIGURE 2

MEASURED CREEP IN 6 X 12 INCH CYLINDERS

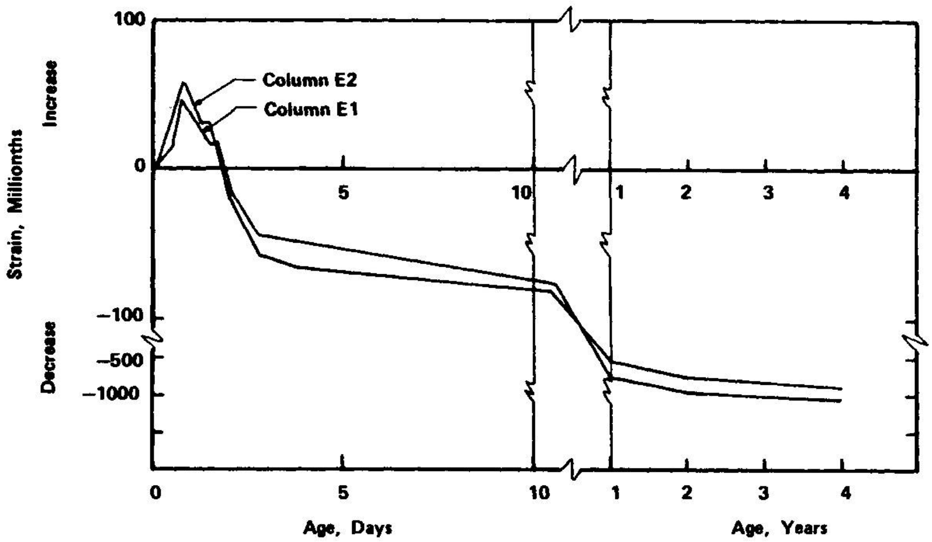


FIGURE 3

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

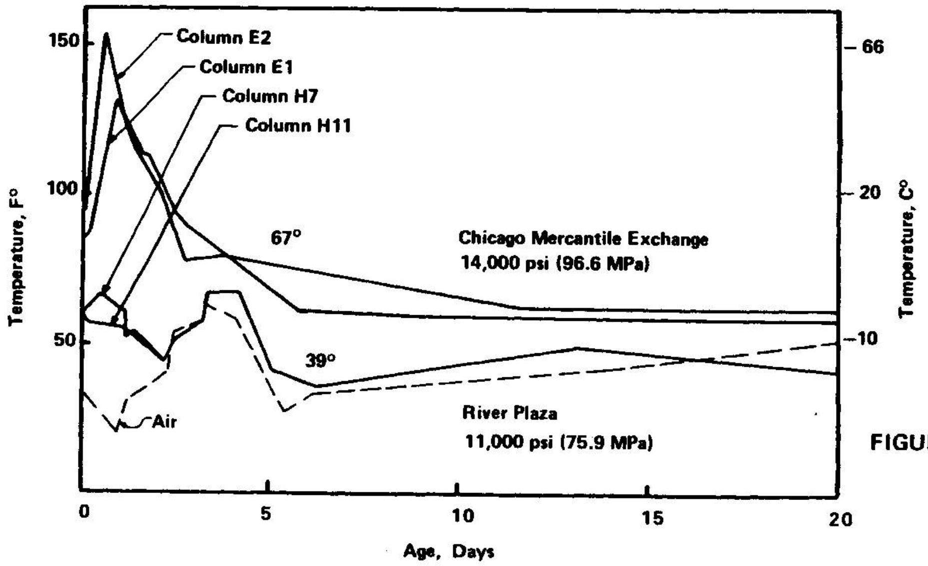


FIGURE 4

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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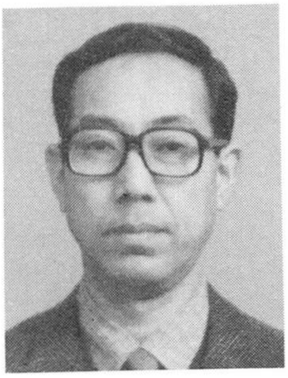

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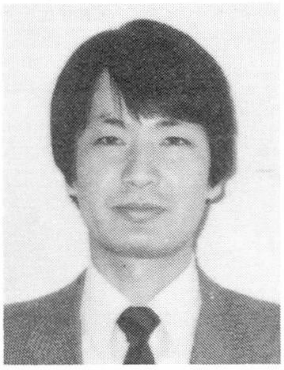
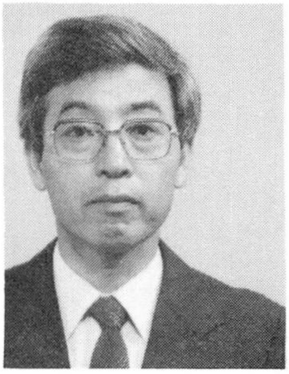




SUMMARY
The characteristics of a new material developed to replace reinforcing steel bars are introduced together with experimental results. This non-corrosive, lightweight new material is made of fiber reinforced plastics. It was formed in two – or three-dimensional grid shape and was developed for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

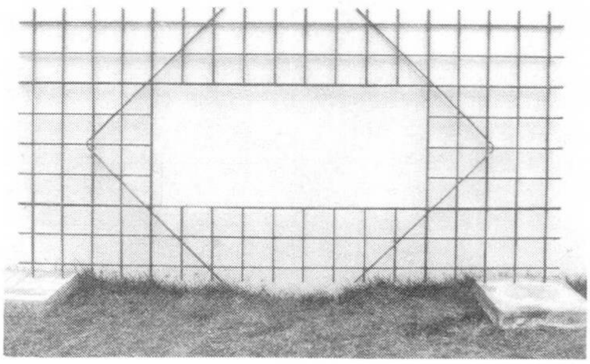


Fig. 1 Sample of NEFMAC

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.

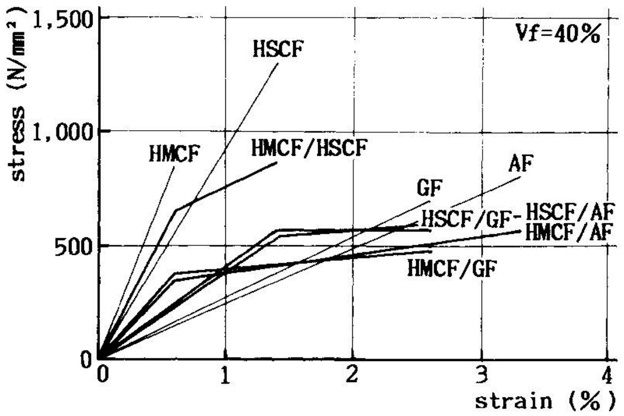
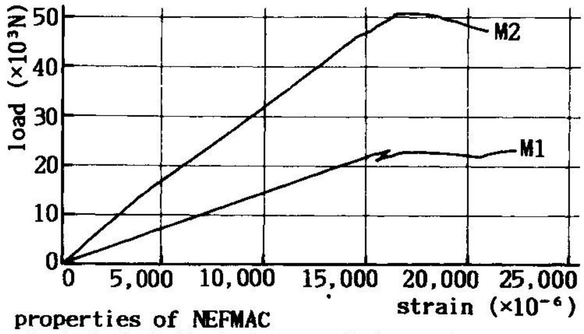


Fig. 2 Schematic stress - strain relationship of NEFMAC



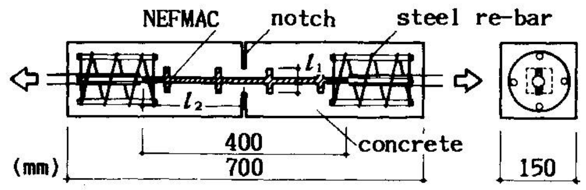
properties of NEFMAC

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

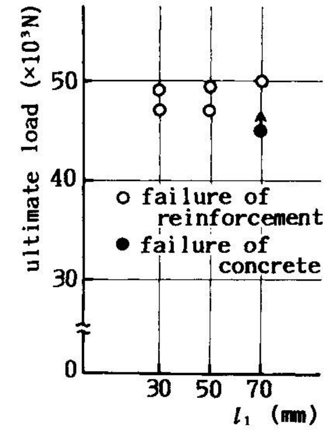
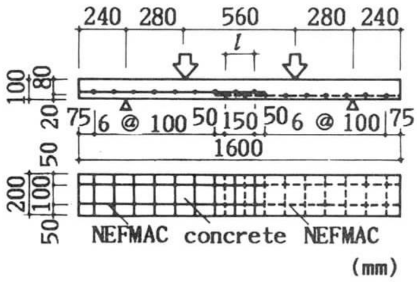


Fig. 5 Ultimate loads of the tensile tests

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 (mm²)	Vf (%)	tensile load (N)	modulus (N/mm²)
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

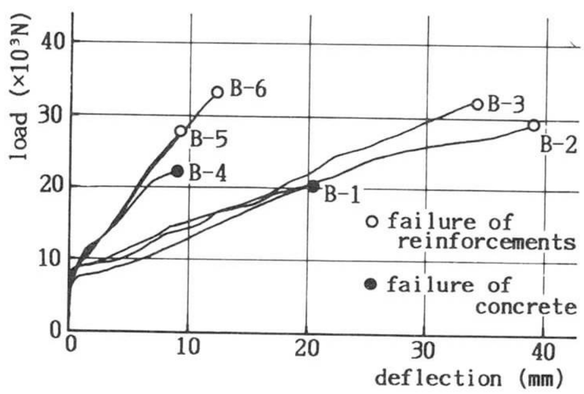
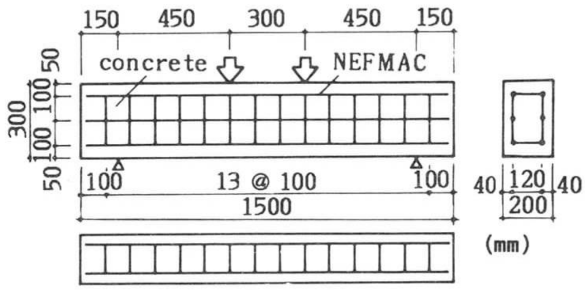


Fig. 7 Load - deflection curves of the bending tests

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 (mm²)	Vf (%)	tensile load (N)	modulus (N/mm²)
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

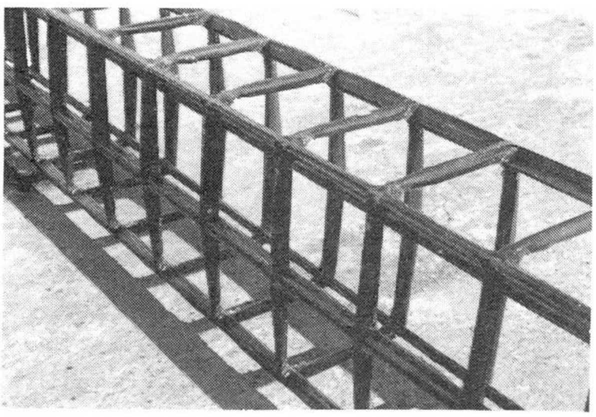


Fig. 9 Sample of three-dimensional shaped NEFMAC

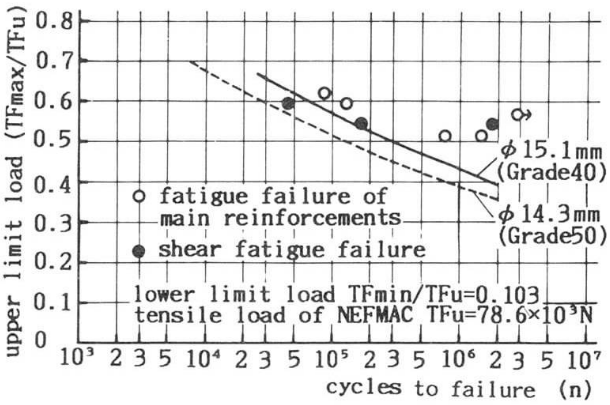
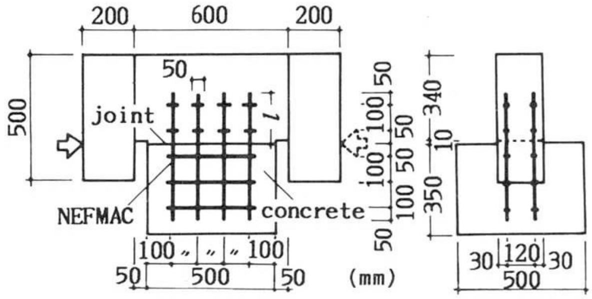


Fig.10 Fatigue strength of NEFMAC

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 (mm²)	Vf (%)	tensile load (N)	modulus (N/mm²)
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

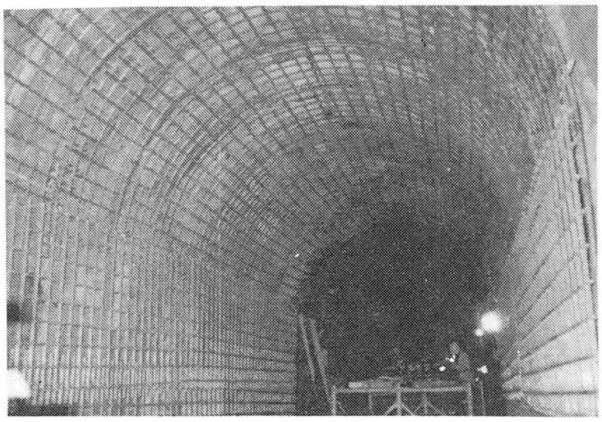


Fig.13 Reinforcements for arch of tunnel

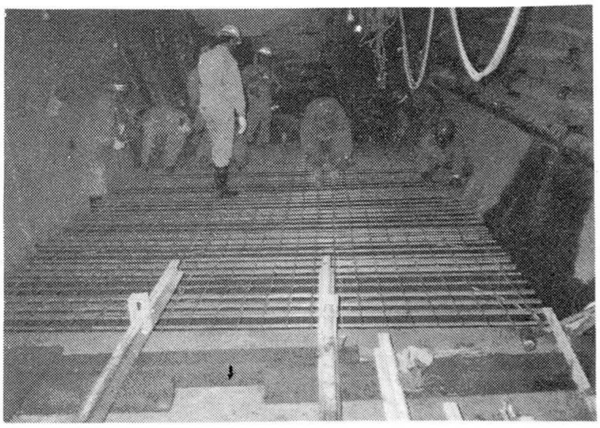


Fig.14 Reinforcements for invert of tunnel

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

The authors would like to express their deep gratitude to Professor Seiji Kokusho and Research Associate Katsumi Kobayashi, Tokyo Institute of Technology for kind guidance and advice on this paper.
The authors also wish to show their profound appreciation to President Hideo Futagawa, Dai Nihonglass Industry Co., Ltd., Minoru Sugita and Teruyuki Nakatsuji, Shimizu Construction Co., Ltd., and the other project members.

REFERENCE

1. Japan Society of Civil Engineers, Standard Specifications for Concrete, 1986.

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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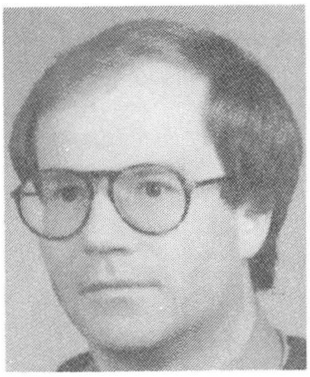

Reinhard Wolff, geboren 1945. Promotion an der TH Darmstadt. Mehrere Jahre im Techn. Büro der Strabag Bau-AG, Köln u. Düsseldorf. Seit 3 Jahren Leiter der Hauptabteilung Technik und Projektbearbeitung der Strabag Hauptverwaltung in Köln.

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).

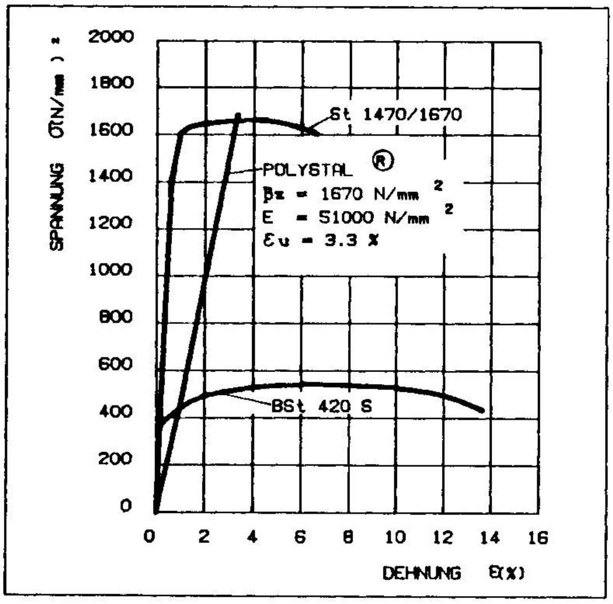

Bild 1 Spannungsdehnungsdiagramm von POLYSTAL-Stäben im Vergleich mit Betonstahl und Spannstahl

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.

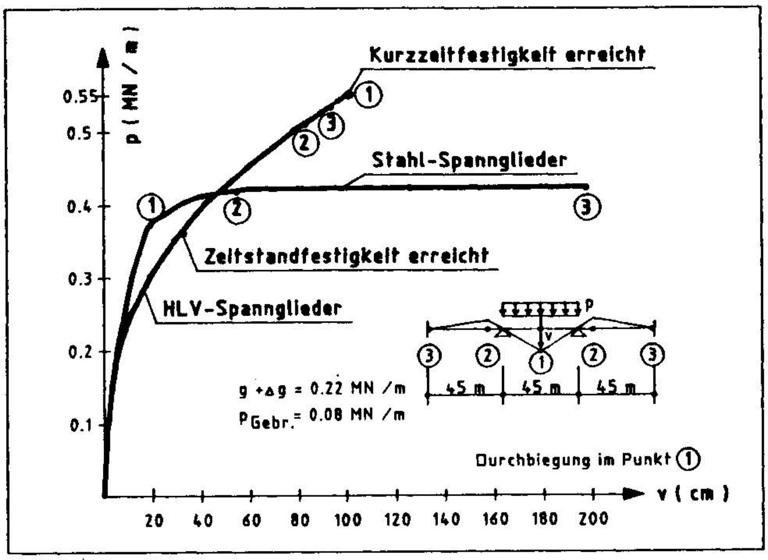

Bild 2 Last-Verformungs-Verhalten bei Überlastung eines Innenfeldes eines Durchlaufträgers

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.

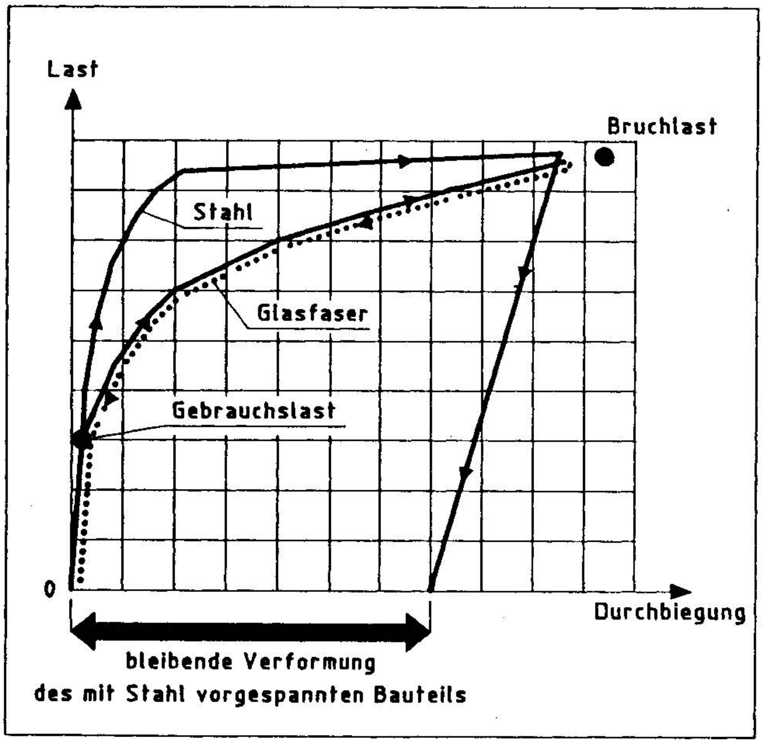

Bild 3 Last-Durchbiegungsbeziehung eines mit POLYSTAL vorgespannten Bauteils zu einem mit Stahl vorgespannten Bauteil.

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).

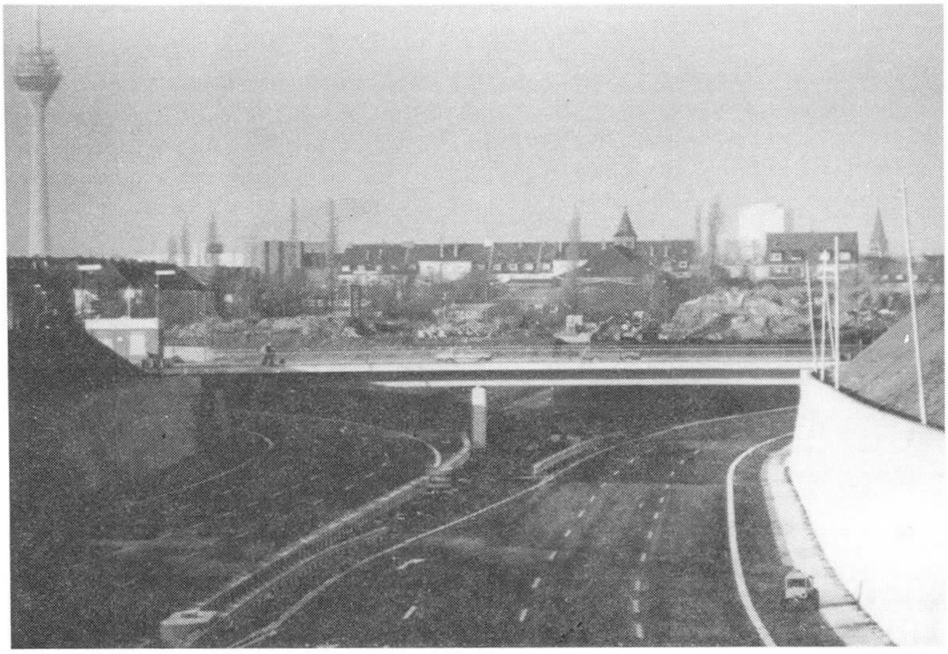

Bild 4 Gesamtaufnahme Brücke Ulenbergstraße, Düsseldorf

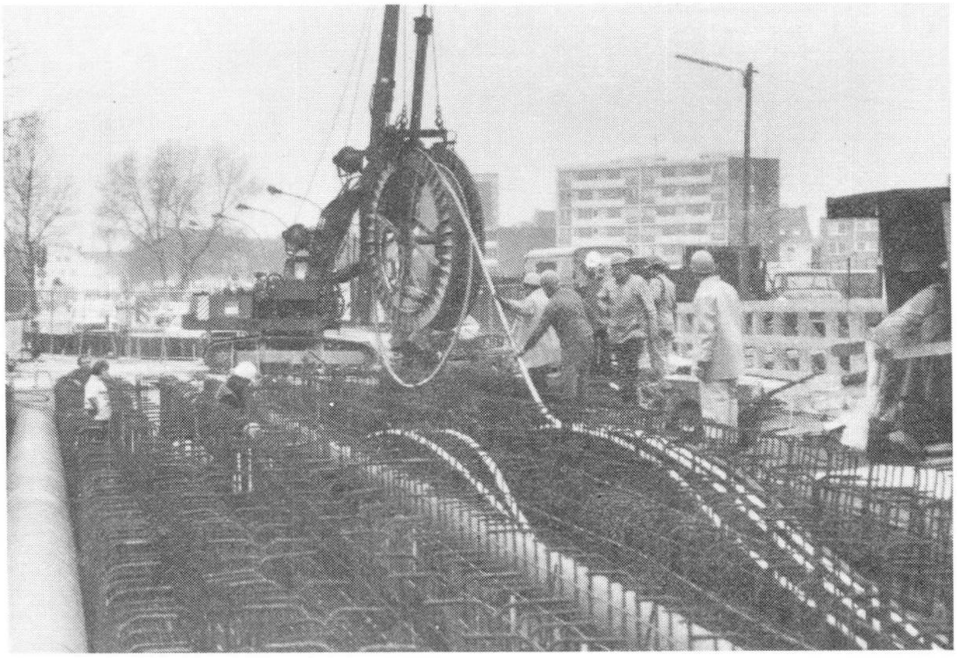

Bild 5 Verlegen der HLV-Spannglieder

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.

LITERATURVERZEICHNIS

1. WEISER, M.; PREIS, L.: Kunstharzgebundene Glasfaserstäbe - eine korrosionsbeständige Alternative zum Spannstahl, Sonderdruck aus: Fortschritte im konstruktiven Ingenieurbau, Gallus Rehm zum 60. Geburtstag. Verlag Wilhelm Ernst & Sohn, Berlin 1985.

2. WAASER, E.; WOLFF, R.: Ein neuer Werkstoff für Spannbeton, "beton" 36 (1986), H. 7, S. 245 - 250.

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

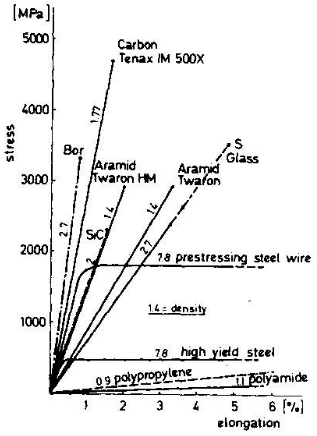

Fig.1 Stress-strain diagram of reinforcing materials

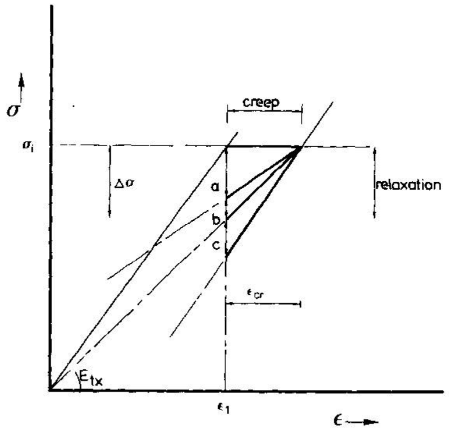

Fig.2 Possible relations between creep and relaxation at a chosen level

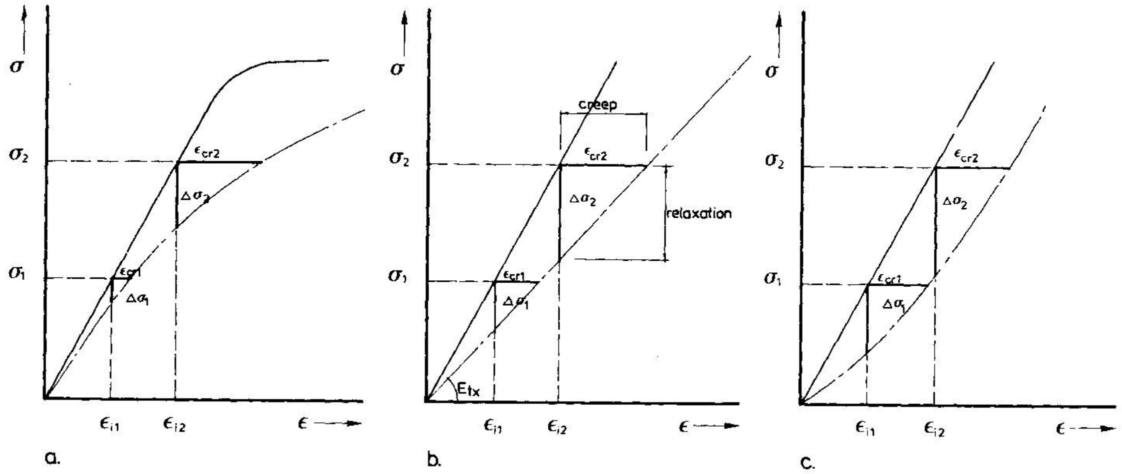

Fig.3 Schematic creep/relaxation behaviour at different initial stress-levels

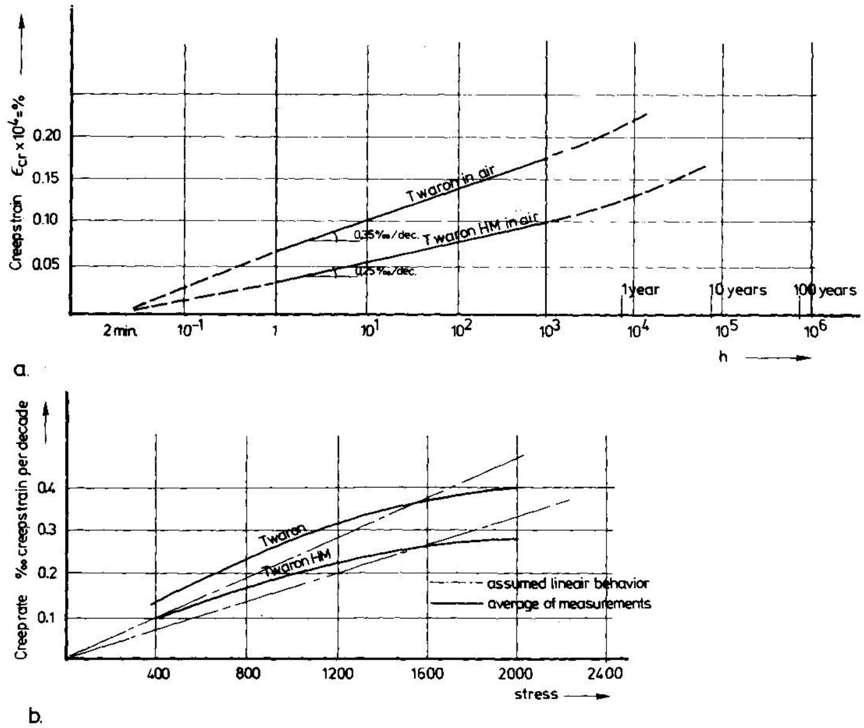

Fig.4 a Creep data of Arapree based on Twaron HM and Twaron in air, measured at $0.5f_u$ (ca. 1500 N/mm^2)

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 $$\Delta\sigma_{p,1} = n \cdot \Delta\sigma_{p.1000}\left(1 - m\,\frac{\Delta\sigma_{p,r+\phi}}{\sigma_{po}}\right) \qquad (1)$$

A pretensioned prestressed column 220 mm square ($A_c = 484\times10^2$ 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] → E-mod = $33{,}5\times10^3$ N/mm² → $\varepsilon_{elast.} = \dfrac{10}{33.5 \times 10^3} = 0.33\times10^{-3}$
assume ϕ creep = 2 → $\varepsilon_\phi = 0.66\times10^{-3}$
assume shrinkage $\varepsilon_{cs} = 0.25\times10^{-3}$ $\qquad \underline{0.91\times10^{-3}}$
Concrete deformation: $\qquad 1.24\times10^{-3}$

$\Delta\sigma_{p,r+\phi}$ becomes:
Twaron HM → $0.91\times10^{-3}\times130\times10^3 = 118$ N/mm²
Twaron → $0.91\times10^{-3}\times 70\times10^3 = 64$ N/mm²
Steel → $0.91\times10^{-3}\times200\times10^3 = 182$ N/mm²

Losses due to relaxation:

		Twaron HM	Twaron	Steel (type I)	Steel (type II)	
Twaron HM	$\Delta\sigma_{p,1} = 2\times10(1-2\times\frac{118}{1500}) =$	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	$\Delta\sigma_{p,2} = 1.24\times10^{-3}\times130\times10^3 = 161$ N/mm²	11.5				%
Twaron	" $= 1.24\times10^{-3}\times 70\times10^3 = 87$ N/mm²		6.2			%
Steel	" $= 1.24\times10^{-3}\times200\times10^3 = 248$ 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	%

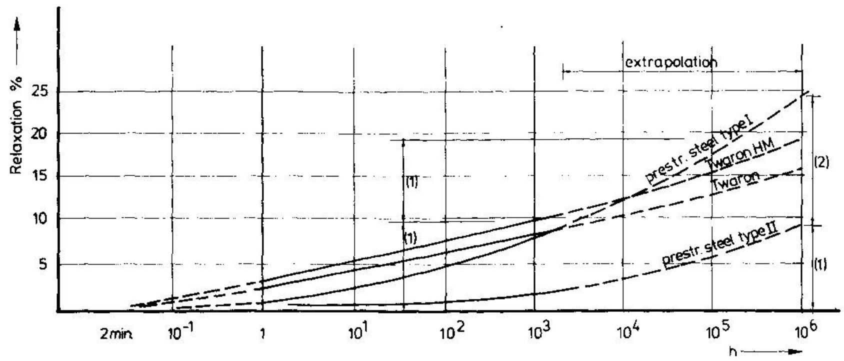

Fig.5 Relaxation behaviour

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

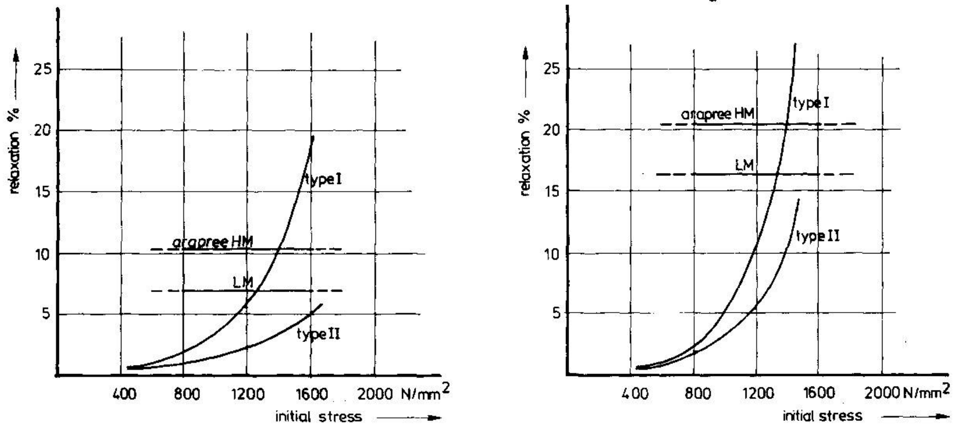

Fig.6 Comparative relaxation behaviour at different initial stress-levels

a Measured at 10^3 h b Expected at 10^6 h

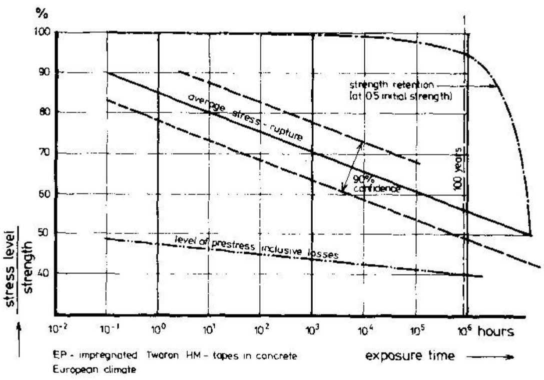

Fig.7 Safety prognosis

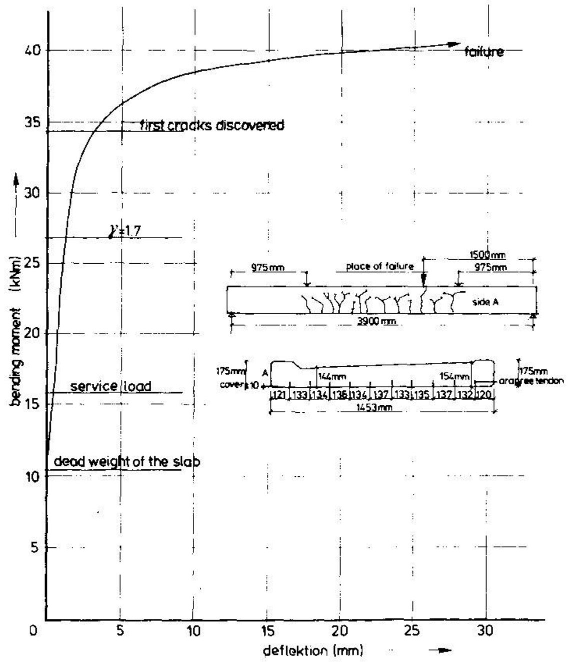

Fig.8 Graphical representation of a load-test on a balcony slab

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.

8. REFERENCES

1. Prestressing with aramid tendons. A. Gerritse and H.J. Schürhoff FIP, 10th Congress New Delhi

2. Aramid reinforced concrete (ARC). H.J. Schürhoff and A. Gerritse Rilem Symposium Sheffield (FRC'86)

3. Kunstharzgebunden Glasfaserstäbe, eine korrosionsbeständige Alternative für Spannstahl. M. Weiser, L. Preis zum 60 Geburtstag von G. Rhem, Forschritte in Konstruktiven Ingenieurbau; 1984

4. Prestressing with Parafil tendons. C.J. Burgoyne, J.J. Chambers; Concrete, Oct. 1985.

5. Creep of aromatic polyamide fibres. R.H. Eriksen. Polymers, 26 May 1985

6. Creep op Kevlar 49 fibre and a Kevlar 49 cement composite. P.L. Walton and A.J. Majumdar.Journal of Materials Science 18 (1983).

7. The Creep of Kevlar 29 Fibers. M.H. Lafitte and A.R. Bunsell. Polymer Engineering and Science, Febr. 1985.

8. FIP-Recommendations on Practical Design.

9. Brochure NDI 1976. Characteristics of Prestressing Steels.

10. Euronorm 138; Prestressing Steel.

11. Spannungrisskorrosion von Spannstählen-Prüfung im Ammonrhodanid versuch G. Hampjes. FIP, 10th Congress New Delhi, (Australian bundle no. 4).

12. Residual-Strength determination in Polymer materials. R.M. Christensen Journal of Rheology, 25 (5), 1981

13. Kruipgedrag van aramide. R.A.G.M. Sandbergen. Internal Report. HBG R&D department. 1987.

14. Load test on a balcony slab prestressed with Arapree tendons. Report nr: B-86-666/62.6.3036. TNO-IBBC. May 1987. (Not publicly available)