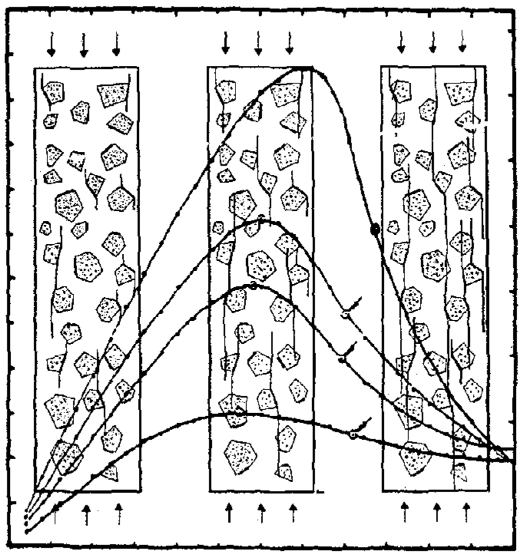
PB80-210057

HIGH STRENGTH CONCRETE

Proceedings of a Workshop Held at the
University of Illinois at Chicago Circle
December 2-4, 1979

Sponsored by

National Science Foundation



S. P. Shah, Editor

Steering Committee: C. Babendreier, Z. Bazant (Co-chairman), G. Frohnsdorff, A. Naaman, H. Russell, S. Shah (Chairman), F. Young, P. Zia

PROCEEDINGS OF THE WORKSHOP

ON

HIGH STRENGTH CONCRETE

Edited by
S.P. Shah

Department of Materials Engineering
UNIVERSITY OF ILLINOIS AT CHICAGO CIRCLE
Chicago, Illinois U.S.A.

December 2-4, 1979

Sponsored by
NATIONAL SCIENCE FOUNDATION

Steering Committee

C. Babendreier, National Science Foundation
Z. Bazant, Co- Chairperson, Northwestern University
G. Frohnsdorff, National Bureau of Standards
A. Naaman, University of Illinois at Chicago Circle
H. Russell, Portland Cement Association
S. Shah, Chairperson, Universityof Illinois at Chicago Circle
F. Young, University of Illinois at Urbana-Champaign
P. Zia, North Carolina State University

ia

PREFACE

High-strength concretes represent a rather recent development which is now finding rapidly increasing use, especially in the construction of tall buildings. However, the scope and extent of the applications of these high-strength concretes are still limited, partly because the knowledge of the distinct features of the mechanical behavior of high-strength concretes is not clearly established. This is natural to expect in the case of a very recent development. Improved understanding of the behavior of concrete is also needed to make certain that no serious deficiencies or failures appear in the high strength structures that are built. Therefore, it is of paramount importance to analyze the material behavior as accurately as the current state of theoretical mechanics permits, to identify the major gaps in knowledge, and to formulate the approaches to overcome them. In particular, the questions of current code provisions, shear-transfer, ductility, brittleness of failure, fracture mechanics, and triaxial loading call for far deeper examinations than have been carried out to date.

One obvious undesirable feature in the development of high-strength concrete has been the gap in communication among material scientists, structural engineers, and mechanics theorists. As for the development of this new material per se, it is the material scientists and cement physicists who deserve the credit. They however, often examine only the most elementary mechanical properties of the material, such as the uniaxial strength and elastic modulus, whereas for a careful and reliable evaluation of the performance of high-strength concrete structures, the multiaxial nonlinear behavior and fracture propagation must be studied as well. It is true that these problems have been more or less empirically and intuitively addressed by structural engineers, but an in-depth examination must come from materials engineers and structural mechanicists.

This workshop was held to partly remedy the situation, establish communication, and identify major uncertainties and avenues of approaching them. The purposes of the workshop were to: 1) develop statements on research needs and aspects related to concrete and high strength concrete, 2) establish a dialogue among materials scientists, materials engineers, researchers with an interest in mechanics and structural engineers, and 3) identify and synthesize various research approaches and different levels at which concrete and concrete structures are examined (see Table).

The five sessions of the workshop (see program) were held during two days, December 3 and 4, 1979. For each session, except the last concluding session, a reporter and a discussor orally summarized their written contributions which are included in these proceedings. Following this, extensive and lively discussion orchestrated by the moderator took place. These floor discussions are briefly summarized for each of the five sessions by the corresponding recorders.

There were a total of about seventy participants (see the list of the attendees) whose interest ranged from physical chemistry to practical structural design. The participants came from different parts of the world, including U.S.A., Canada, Mexico, Sweden, Norway, Germany, Italy, England, Holland, China, and Japan.

The workshop was sponsored by the National Science Foundation. The active support and encouragement of C. Babendreier and R. Ayre is gratefully acknowledged. I would also like to express my gratitude to the members of the steering committee, and the staff of the Department of Materials Engineering for their invaluable help. These proceedings were reproduced at the Portland Cement Association, Skokie, Illinois; their assistance is greatly appreciated.

S. P. Shah
April 1980

Levels at which material is considered / Research Approaches and tools	Concrete (Constituents)				Reinforcing Systems								Structural		
	Cement	Aggregates	Additives	Interface	Concrete	Reinforced Concrete	Prestressed Concrete	Fiber-Reinforced Concrete	Polymer-Concrete	Modified Concrete	Interface (bond-corrosion)	Additives	Frames	Wall-Panels	ETC....
Physico-chemical															
Material Properties															
Experimental Mechanics															
Theoretical Mechanics															
Structural Mechanics															
Structural Engineering															

Table - Subjects and Disciplines

TABLE OF CONTENTS

PROGRAM

NSF HIGH STRENGTH CONCRETE WORKSHOP

All the events, unless otherwise mentioned, will be held in the Chicago Circle Center (Polk and Halsted) on the campus of the University of Illinois at Chicago Circle.

Sunday, December 2

7:00 - 9:00 P.M.	Reception-Registration, Belmont Room, Pick Congress Hotel

Monday, December 3

8:00 A.M.	Bus leaves Pick-Congress Hotel for UICC
8:30 - 9:00 A.M.	Registration Room 509-510 CCC
9:00 - 9:30 A.M.	Introduction and welcome addresses

Introduction: S. P. Shah
Welcome Address: Paul Chung, Dean
College of Engineering
Robert Ayre
Civil & Mechanical Engrg. Division
National Science Foundation

9:30 - 12:30 P.M. SESSION I: MICROMECHANICS OF ACHIEVING HIGH STRENGTH CONCRETE
Moderator: G. Frohnsdorff
Reporter: F. Wittman
Discussor: C. Brown
Recorder: S. Diamond

(Includes a 30-minute coffee break at 10:30 A.M.)

12:30 - 2:00 P.M. Lunch, Room 605 CCC

2:00 - 5:30 P.M. SESSION II: MATERIAL BEHAVIOR UNDER VARIOUS TYPES OF LOADING
Moderator: K. Pister
Reporter: K. Gerstle
Discussor: Z. Bazant
Recorder: Wai-Fah Chen

(Includes a 30-minute coffee break at 3:30 P.M.)

Monday, December 3 (cont'd)

5:30 - 7:30 P.M.	Wine and Cheese Party, Room 509 CCC
7:30 P.M.	Bus leaves for Pick-Congress Hotel

Tuesday, December 4

8:00 A.M.	Bus leaves Pick-Congress Hotel for UICC
8:30 - 11:30 A.M.	SESSION III: INELASTIC BEHAVIOR OF STRUCTURAL ELEMENTS AND STRUCTURES Moderator: S. Uzumeri Reporter: V. Bertero Discussor: W. Schnobrich Recorder: J. Jirsa

(Includes a 30-minute coffee break at 10:00 A.M.)

11:30 - 1:00 P.M.	Lunch, Room 605 CCC
1:00 - 4:00 P.M.	SESSION IV: SPECIAL DESIGN FEATURES APPROPRIATE FOR HIGH STRENGTH CONCRETE Moderator: F. Robles Reporter: P. Zia Discussor: A. Naaman Recorder: A. Nilson

(Includes a 30-minute coffee break at 3:00 P.M.)

4:00 - 6:30 P.M.	SESSION V: CONCLUDING SESSION Moderator: S. P. Shah Reporters: S. Diamond, W. Chen, J. Jirsa, A. Nilson Recorders: F. Young and H. Russell
7:00 - 10:00 P.M.	Imperial Banquet (Mandar Inn)
10:00 P.M.	Bus leaves for Pick-Congress Hotel

A SUMMARY

by

S. P. Shah
Department of Materials Engineering
University of Illinois at Chicago Circle
Chicago, Illinois 60680

INTRODUCTION

It is difficult to summarize the extensive and excellent presentations of the reporters and the discussors as well as the involved and intricate floor discussions. Such a summary is likely to be both partial and personal. In addition, this summary may be even more unsatisfying since the presentations covered a wide ground; from materials science to structural engineering and from micromechanics to continuum mechanics. Nevertheless, I hope that this review gives some idea of the problems and prospects of high strength concrete.

WHAT IS HIGH STRENGTH CONCRETE?

High strength concrete is a relative term; what is considered high strength in Houston may be considered a normal strength concrete in Chicago. For the purpóse of this presentation, it might be convenient to agree with the definition proposed by Bertero:

High strength concrete is considered to be concrete with compressive strength higher than 6000 psi. for normal weight aggregates and 4000 psi. for lightweight aggregates.

This definition seems justified in light of the following two arguments:

1. With the conventional methods of production and materials of construction, the bulk of concrete a ready-mix supplier delivers is in the range of 3000-6000 psi. Using the same materials and methods of production he may be able to deliver concrete of much higher compressive strength. However, to produce concrete above 6000 psi (for normal weight aggregates) more stringent quality control, use of admixtures (plasticizers, fly ash, etc.), and careful selection of the blends of cement and the type and size of aggregates are essential. Thus, to distinguish concrete of above 6000 psi compressive strength, it may be termed high strength.

2. The current design practice is based among other things, on experiments made with concrete of compressive strength in the range of 3000-6000 psi. Additional considerations, modifications of the empirical equations, and new tests may be necessary before a satisfactory procedure for the design of structures made with concrete of compressive strength significantly higher than 6000 psi. is developed. Thus, concrete of compressive strength higher than 6000 psi. can be considered in a separate class as high strength concrete.

There are many ways to produce high strength concrete. The method which is commercially common differs from normal strength concrete only in certain details reflecting the quality of its components and of the mix proportions. In particular, high strength concrete generally comprises a low water-cement ratio made possible by water reducing admixtures, high cement content, portland cement of superior strength-producing capability, fly ash and strong, stiff aggregates.

There are other ways of producing high strength concrete. One is using different cementitious material such as low porocity cement mentioned by Diamond. Compressive strength of up to 25,000 psi, or higher has been achieved when the capillary pores in the cement matrix of the cured and dried concrete are filled with solid polymer or sulfur (polymer impregnated concrete). Reinforced concrete beams made with concrete of compressive strength of up to 23,000 psi. were recently tested by Italian researchers as mentioned by Naaman. The high compressive strength was achieved by using quartzite aggregates, portland cement and silica powder and by combination of low and high pressure steam curing.

The measure of high strength concrete is generally the uniaxial compressive strength. For concrete with the normal compressive strength range (less than 6,000 psi), it is generally assumed that the higher the strength, the higher the overall qualities of concrete. Many other properties of concrete such as tensile, shear, and bond strength are expressed in terms of the compressive strength. It was pointed out by Wittmann that one cannot always assume, especially for high strength concrete, that higher strength also means superior other properties (for example, ductility). Thus it may be misleading to use uniaxial compressive strength alone as the parameter when discussing high strength concrete.

The economic advantages of using high strength concrete for the columns and shear walls of high-rise buildings have been demonstrated already in buildings such as the Water Tower Place in Chicago. In general, high strength concrete can be and has been advantageously used for:

1. columns and shear walls of high-rise buildings,
2. elevated structures,
3. precast and or prestressed products, and
4. construction where durability (low porocity) is critical.

STRESS-STRAIN RELATIONSHIP

One of the most important indications of mechanical behavior of any structural material is its stress-strain curve. For concrete it is becoming increasingly clear that for rational predictions of inelastic behavior of structural members, both the ascending and the descending parts of the stress-strain curve are significant. This need to know the complete stress-strain curve was clearly elaborated by Bertero. The most commonly observed stress-strain relationship for concrete is that obtained under uniaxial compression. Even under this apparently straightforward condition, there are many variables which can influence the observed stress-strain curve. These are summarized below.

Type of Specimen

To stimulate the compression zone of reinforced and prestressed concrete structural members the so-called C-shaped specimens subjected to eccentric compression, pioneered by Hognestad, have been used to obtain stress-strain curves of concrete of differing compressive strengths. The current ACI-Code ultimate strength design parameters are based on the information obtained from testing this type of specimen. Although such specimens do simulate strain gradients present in structural members there are severe disadvantages associated with them: (a) The stress-strain relationship cannot be directly obtained from the test results, (b) The strain rate and strain gradient are continuously varying during the test and their effects cannot be separated, and (c) it is difficult to obtain the descending portion. It seems that more basic information is obtained by testing cylindrical or prismatic specimen subjected to concentric compression at a constant rate of axial strain.

Difficulties in Obtaining Descending Part

It is generally agreed that with increasing compressive strength, the strain at the peak stress (peak strain) increases and the slope of the descending part becomes more steep. It is not clear at what value of strength the material becomes classically brittle and the slope of the descending part becomes vertical and therefore not practically usable in design. While at the University of Illinois - Chicago Circle (UICC) researchers have obtained a reproducible descending part for normal weight concrete of compressive strength up to 12,000 psi and for lightweight concrete up to 8,000 psi, the Cornell researchers report brittle behavior beyond

7,000 psi. Part of the reason for such conflicting information may have to do with the influence of testing variables which were mentioned by Naaman and Bazant. Even for an identical concrete, one may obtain differing descending portions depending on: (1) size and shape of the specimen, (2) rigidity of the testing machine, (3) frequency response of a closed-loop testing machine, (4) method of controlling strain in a constant strain rate test, and (5) the rate of strain.

VARIOUS TYPES OF LOADING

The present state of the art for both normal and high strength concrete subjected to various types of loading was well summarized by Gerstle. Very little is known about the behavior of high strength concrete subjected to multiaxial state of stress, creep loading and cyclic loading. For normal strength concrete, the influence of different types of loading is often expressed empirically in terms of parameters which are related to the uniaxial compressive strength (f'_c). For example the effect of lateral confining pressure ($\sigma_3 = \sigma_2$) on the axial compressive strength ($\sigma_1 u$) is often expressed by the equation: $\sigma_1 u = f'_c + 4\sigma_3$. Similarly, the fatigue strength, tensile strength and modulus of rupture are often related to uniaxial compressive strength. It is not clear whether a similiar approach will be valid for high strength concrete, and if so, whether the same empirical constants will be applicable. Higher strength can be achieved by changing the water-cement ratio, the aggregate-cement ratio, using different types of aggregates or by changing curing conditions. Would the behavior of high strength concrete of a given compressive strength be the same regardless of how the higher strength was achieved? Is an approach which was valid for 3,000-6,000 psi. range also applicable when one extends the range to say 12,000 psi.? Additional research is necessary before such questions can be answered.

Confinement

It is economically advantageous to use high strength for construction of columns of high rise buildings. Since the columns are the most vulnerable elements of a building subjected to earthquake loading, it is important to know whether high strength concrete columns can have adequate ductility under seismic conditions. Since the ductility in columns is achieved primarily by lateral confinement provided by hoops or stirrups, it is vital to study the behavior of confined high strength concrete.

A continuing study at the UICC showed that the comparison between confined normal and high strength concrete is somewhat similar to that between confined normal and lightweight concrete, The effect of hoop reinforcement or the confinement index decreased with increasing compressive strength. Similarly a given reinforcement was less effective for lightweight concrete of the same compressive strength as compared to that for normal weight concrete. As was pointed out by Professor Bertero a desired ductility can be achieved using a higher volume of closely spaced hoop reinforcement made with high strength steel. An alternate approach may be to use fiber reinforcement as was suggested by Zia.

Cyclic Loading

To predict the behavior of reinforced high strength concrete columns subjected to earthquake type loading, behavior of confined concrete subjected to cyclic loading needs to be known. Researchers at UICC have initiated a research project aimed in that direction. The results ontained so far indicate that the concept of envelope curve put forward by Jirsa as well as by Gerstle seem to apply to lightweight concrete, high strength concrete, both confined and unconfined concrete and at both static and dynamic strain rates.

CODE MODIFICATIONS

Many of the current code provisions and suggested methods may have to be modified to include high strength concrete. This was pointed out by both Professors Zia and Bertero. The following is a partial list of code provisions which may need attention:

1. the method of computing P-M strength interaction curve for columns,
2. redistribution of negative moment in continuous flexural members,
3. confinement index for adequate ductility or the criteria for minimum spiral reinforcement,
4. method of predicting modulus of elasticity, tensile strength, bond strength, deflection and crack width,
5. shear strength, and
6. maximum allowable reinforcing ratio and index.

The critical examination of the code provisions for their validity regarding high strength concrete may also lead to more rational provisions as well as a better understanding of the current empirical formulas. For example, currently the ACI code suggests a constant value of 0.003 for the ultimate strain of concrete. This ultimate strain value may be defined as the strain of the extreme fiber of the compression zone of a structural member at the ultimate (maximum) load. This value of ultimate strain in reality is not a constant value but depends on the properties of the cross-section of a structural member and the type of loading.

It is not often realized that the stress strain curve of concrete alone is inadequate to determine the ultimate strain value as well as the ultimate strength design parameters (β_1, k_1k_3, etc.). However, from the knowledge of the complete stress-strain curve, the properties of concrete compression block for different types of cross-sections and structural members can be generated. Thus, as pointed out by Bertero and Naaman, the knowledge of the complete stress-strain curve of high strength concrete is essential for developing possible code modifications.

IS HIGH STRENGTH CONCRETE MORE BRITTLE?

The answer appears to be both yes and no. It is necessary to distinguish between material, sectional, and structural ductility. At the material level, the ductility of plain concrete seems to decrease with the increasing compressive strength, but the same is not necessarily true at the sectional level. It has been shown that the ductility ratio of a reinforced concrete section subjected to flexure increases with increasing compressive strength provided that the amount of steel is kept constant. However, if the amount of steel is maintained as a constant fraction of the balanced amount of steel, then the ductility ratio is independent of the compressive strength of concrete. For columns, the reduced material ductility of high strength concrete may be a problem for loads above the balanced load unless the section is adequately confined. At the structural level, the effects of load reversals involving high shear needs to be studied for high strength concrete.

MICRO MACRO AND FRACTURE MECHANICS

For normal strength concrete made with normal weight aggregates, the fracture path is predominantly around the aggregates. With increasing compressive strength of concrete the aggregate-fracture becomes more noticeable. Partly as a result, high strength concrete, similar to lightweight aggregate concrete shows a more linear behavior, a more brittle mode of fracture and less volumetric dilation. This may mean that linear elastic fracture mechanics is a more appropriate tool to characterize the resistance to crack propagation for high strength concrete. Cracking predominantly through aggregates means smooth cracks. The effect of smooth cracks on transfer of shear stresses in reinfroced concrete beams must be evaluated. The reduced volume dilation for high strength concrete may mean different behavior under multiaxial stresses from that observed for normal strength concrete. This was already mentioned when effects of confining reinforcement were considered. To make the optimum use of aggregates of different strengths, it is necessary to understand the interaction between the two components: paste and aggregates, both at micro and macro levels as well as to establish what conditions favor cracking around the aggregates and through the aggregates.

RESEARCH NEEDS

It is necessary to:

1. Obtain experimental information on mechanical characteristics of concrete and to determine how these characteristics are related to the properties of the matrix, aggregates, and interface,
2. Develop theoretical models to predict the composite behavior from those of its constituents. Possible approaches include micro, macro, continuum and fracture mechanics as suggested by Brown, Bazant, Chen and Wittmann. Such models might help optimize the use of available component materials.
3. Determine the mechanical characteristics (under monotically increasing and cyclic loading) of confined high strength (and lightweight) concrete and formulate models to predict constitutive behavior for such concrete.
4. Improve knowledge about mechanical behavior of structural elements made with high strength concrete subjected to static as well as earthquake types of loading.
5. Examine the applicability of the current ACI code methods for predicting:
    1. limiting amount of tensile reinforcement,
    2. P-M diagram,
    3. shear strength,
    4. minimum amount of hoop reinforcement,
    5. moment redistribution for continuous beams,
    6. serviceability (deflection, cracking, etc.).
6. Analyze relationships among different kinds of cementitious materials, their potential strength development, microstructure of hydrated products, and the rate of heat evolution.

SESSION I - REPORT

MICROMECHANICS OF ACHIEVING HIGH STRENGTH AND OTHER SUPERIOR PROPERTIES

by

Folker H. Wittmann
Delft University of Technology
Department of Civil Engineering
Materials Science Group
Stevinweg 1 postbox 5048
2600 GA Delft The Netherlands

ABSTRACT

First of all it is pointed out that strength of a material is dependent on loading conditions. High strength, low shrinkage, low permeability, and low thermal dilation may be defined as superior properties. Low creep may be advantageous in prestressed concrete members, in other cases high creep may prevent cracking. Therefore concrete with superior creep properties must be optimized with respect to creep. The performance concept may be used to describe criteria and materials behaviour in a systematic way. With respect to micromechanics, the structure of concrete may be subdivided into three different levels. On the micro-level we have to deal with the porous matrix of hardened cement paste. The structure of mortar is built up by sand inclusions, microcracks, and capillary pores. The mortar structure is defined to be the mezzolevel. Finally the structure of concrete with coarse aggregates, compaction pores, and interfacial cracks is dealt with on the macrolevel. Crack propagation in a homogeneous infinite plate with one cylindrical hole is described. This approach is extended to cover crack formation in porous materials. Then crack formation and crack propagation in a composite material is investigated. The influence of properties and geometrical arrangement of interfaces is discussed. Failure of normal and high strength concerete is studied with a computer simulation method. The application of micromechanics is further discussed by means of two examples, failure under high sustained load, and influence of rate of loading on strength. It is concluded that porosity and pore size distribution are decisive factors for achieving high strength and other properties. On the basis of the micromechanical approach described here, characteristic properties of high strength concrete may be predicted.

1. ON THE MEANING OF HIGH STRENGTH

Before we can deal with elements of micromechanics of composite materials such as concrete, the meaning of the term "high strength" has to be discussed. It is obvious that compressive strength at 28 days and determined under conventional conditions is not sufficient to characterize or classify a given type of concrete. An observed strength value does strongly depend on series of influences among which the loading condition plays a dominant role.

High compressive strength does not necessarily mean that a corresponding high tensile strength is reached and vice versa. In many practical cases, concrete is under multiaxial state of stress. Does high uniaxial compressive load guarantee superior performance under two- or tri-axial state of stress?

All strength measurements are strongly influenced by rate of loading. Does high strength concrete, als classified according to conditions of conventional materials testing, have sufficient strength under impact loading conditions and is the behaviour under sustained load satisfying? What is the influence of varying load on the load bearing capacity?

The maximum load achieved during testing is often defined as strength. In general the load bearing capacity, however, is described more specifically by taking the complete stress-strain-diagram into consideration. Brittle failure under comparatively high load may be less desirable than ductile behaviour if higher total fracture work is achieved at lower maximum stress.

Finally the variability of strength values recorded does severely influence the quality of a given concrete. A high mean value has little advantage, if low strength values have a high enough probability of occurence. In fact, concrete with a lower mean value but a comparatively narrow strength distribution may turn out to be superior. This may be quantitatively shown by a sensitivity analysis (1).

Thus it is obvious that the term "high strength" can only be used meaningfully, if it is indicated clearly for what provided loading condition a given structural element is designed.

2. OTHER MECHANICAL PROPERTIES

In most practical cases, a high elastic modulus may be regarded as benificial. With respect to creep, the situation may not be described in similarly simple terms. In a prestressed concrete member, low creep certainly is assumed to be a superior property. Undesirable cracking of undeterminate structures, however, is limited or avoided by sufficient creep. Thus an optimum value has to be found.

All different volumetric changes caused by the interactions with water such as chemical (2), capillary (3), and drying shrinkage should be as small as possible. Therefore many methods to reduce or to compensate shrinkage have been developed.

Thermal stresses may cause cracking in a similar way as shrinkage. Both heat of hydration and coefficient of thermal dilatation preferably must be kept low to achieve concrete with superior properties. In addition, coefficients of thermal dilatation of aggregates and matrix should be close to one another.

3. NON-MECHANICAL PROPERTIES

Of course non-mechanical properties are not independent of the examples mentioned above of mechanical performance. For the sake of completeness some important aspects of non-mechanical behaviour shall be mentioned briefly.

One important property of concrete is its durability under a given climatic environment. Gases in the air such as CO_2, SO_2 and NO_x may react with concrete and thus limit its serviceability. In addition to this, several ions dissolved in water may attack concrete. Diffusion of gases or ions is the rate determining process of the chemical attack just mentioned. Diffusion is essentially influenced by porosity and pore size distribution. As will be shown in this report, that provides a link between mechanical and non-mechanical properties on a micro-mechanical basis.

Artificially added compounds may acclerate the hardening process which is desirable. But if the risk for corrosion is severely increased at the same time one can hardly speak of a material with superior properties.

A common cause for degradation of properties of concrete is frost action. By now much is known about the different elementary processes and therefore superior properties may be maintained under severe climatic conditions.

All properties and influences mentioned above may be listed in a systematic way and they may be supplemented by other relevant influences. This would finally lead to a complete description of criteria for concrete with high strength and other superior properties. The performance concept may be applied to achieve this goal. As we have to concentrate further on micromechanics these incomplete introductry remarks may serve as a useful basis.

4. CRACK PROPAGATION IN A POROUS MATERIAL

With respect to crack formation and crack propagation the structure of concrete may be subdivided into different levels. Hardened cement paste is a porous material with a high internal surface. This system may be described in terms of microlevel. A characteristic hydraulic radius of the pore system is of the order of magnitude of 20 Å. These micropores may be neglected in micromechanics of fracture of concrete because there are plenty of larger pores in the material. Therefore hardened cement paste may be looked upon in good approximation to consist of a homogenous material containing capillary pores.

Capillary pores are at least ten times larger than gel pores and their density as well as their characteristic pore radius depends on the water-cement ration and on the age of a specimen.
Fine aggregates in mortar are bound together by hardened cement paste. In this way a structure similar to natural sand stones is formed. The porosity of the porous mortar system depends essentially on the cement content. This structure will be dealt with at mezzolevel. Besides a characteristic pore size distribution crack arresting by aggregates is typical for a micromechanics approach on the mezzolevel.

Finally compaction pores and cracks formed by bleeding, capillary shrinkage and shrinkage have to be taken into consideration on the macrolevel. Crack formation and crack propagation on the macrolevel is dominated by the interaction of cracks with aggregates. We shall deal with crack formation on the three different levels just mentioned consecutively.

In Fig. 1 a typical pore size distribution of normal concrete is shown by means of a solid line. With dashed lines two pore size distributions as determined at samples of hardened cement paste with differing water-cement-ratio are shown. The capillary pores of hardened cement paste can be easily recognized in the total pore size distribution of concrete. At higher radii the pores of the mortar structure (mezzolevel) can be seen, and finally compaction pores of concrete appear at a radius of about 10^{-5}m and above.

We shall start to describe crack propagation through a porous material with the easiest case, i.e. one circular hole in a homogeneous infinite plate. The well-known stress distribution that occurs if an internal compressive stress is applied is shown in Fig. 2. At the two poles where tensile stress is created, a crack may occur as soon as materials strength is reached. If a tensile load were applied, catastrophic crack propagation would be observed according to the Griffith relation:

$$\sigma = \sqrt{\frac{2E\gamma}{\pi l}} \qquad (1)$$

(All symbols have their usual meaning). Under compression, however, stable crack growth takes place. The crack length increases as the load is increased. It is useful to relate crack length l tot the radius of the pore r:

$$\lambda = \frac{l}{r} \qquad (2)$$

It can be shown (4,5) that the related crack length λ is dependent on the load q and can be described by the following equation:

$$q = \sqrt{\frac{\pi E\gamma}{r}} \sqrt{\frac{(1+\lambda)^7}{2\,(1+\lambda)^2-1}} \qquad (3)$$

If we consider an idealized material with one single pore, the first root in equation (3) becomes a materials constant:

$$C = \sqrt{\frac{\pi E\gamma}{r}} \qquad (4)$$

Now the load q may be related to the materials properties C and one gets the following implicit expression for related crack length as function of related load q^*:

$$q^* = \frac{q}{C} = \sqrt{\frac{(1+\lambda)^7}{2\,(1+\lambda)^2-1}} \qquad (5)$$

Relation (5) is graphically shown in Fig. 3. It is obvious that from relation (5) no failure criterion may be derived. There is no critical crack length. It may be seen, however, from equation (3) that with increasing pore radius the necessary load to create a crack with a given length decreases.

In a real porous material pores are distributed at random. The pore size may be approximated by an extreme value distribution

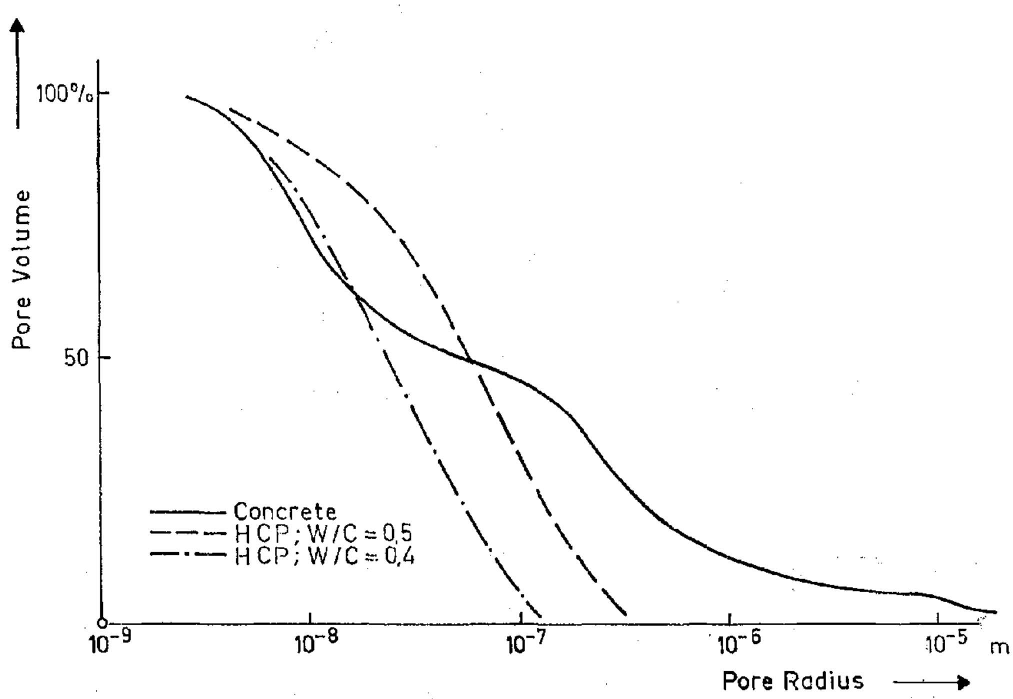


Fig. 1: Pores size distribution of concrete (solid line). For comparison corresponding distribution functions of hardened cement paste (HCP) with w/c-ratio 0,4 and 0,5 are shown by means of dashed lines.

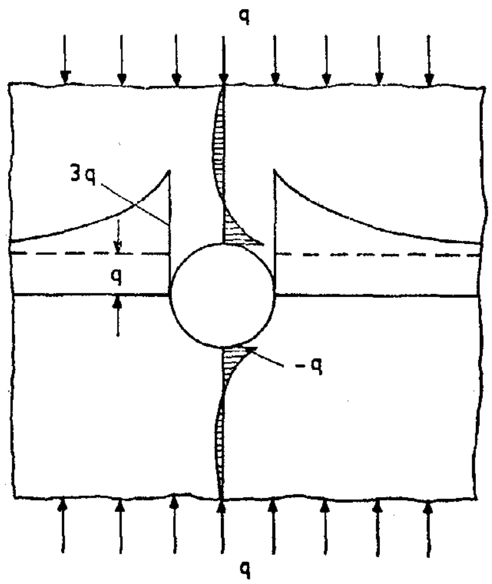


Fig. 2: Stress distribution around a circular hole in an infinite plate under undirectional compressive load.

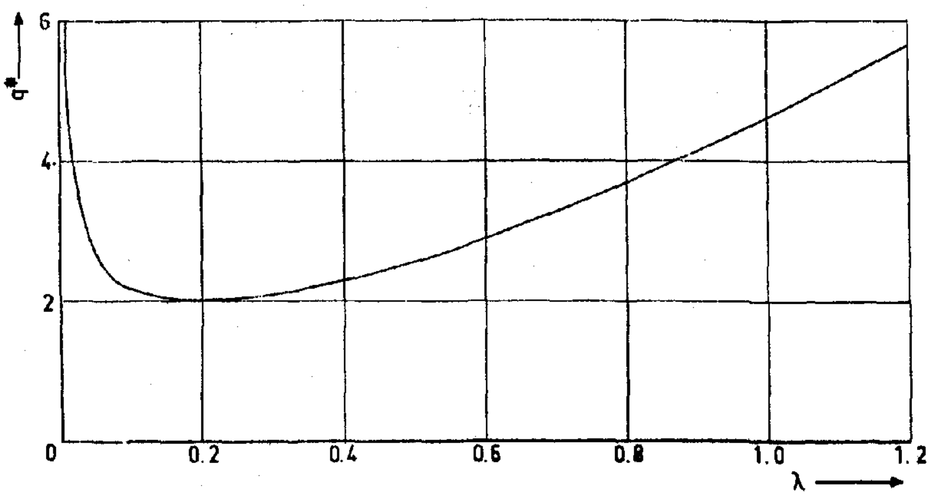


Fig. 3: Graphical representation of relation (5).

function and the distance may be looked upon to be normally distributed. If we neglect for a moment the pore size distribution we may simulate a porous structure in a simplified way by pores distributed at random and arranged on a line. In Fig. 4 an example is given. Pores have an average distance of c and the distribution function of the distance has a standard deviation of Δc. If an external load is applied above a characteristic stress, cracks will propagate from all pores. The crack length will be described in the initial stage by equation (5). If two cracks come close to one another, they interact. It can be shown (4) that they attract one another and finally they coalesce. This brings about a sudden discontinuous jump in crack length. In Fig. 5, the calculated crack length of a computer simulation is shown.
In fact the sum of all crack lengths:

$$S = \sum_{i=1}^{n} (2l_i - 2r) \tag{6}$$

is plotted versus the related load ($r = 1$; $n = 100$). Dashed lines represent the undisturbed situation according to equation (5). But now the crack length increases more rapidly and finally a critical state is reached. At a certain critical total crack length further crack propagation is unstable. The corresponding load may be defined to be an ultimate load and thus a failure criterion is introduced.

In the way just described here, porous structures may be studied in a systematic way. In case the pores are not assumed to be arranged along one line the interaction of approaching cracks becomed much more difficult. For a given porosity optimum pore distributions can be obtained. In concrete materials this is not of primary interest, however, because cracks are influenced by aggregates. Therefore crack propagation in a composite material shall be described in the following section.

5. CRACK PROPAGATION IN A COMPOSITE MATERIAL

In a homogeneous porous material cracks can develop in an arbitrary way. The direction and the crack length are exclusively determined by the external state of stress. In mortar and concrete as in all other composite materials crack propagation is also influenced by the structure of the material. The comparatively high strength of normal aggregate causes crack arresting and crack deviation. As a result, composite materials with high strength aggregates become more ductile than the plain matrix.

As mentioned above in mortar and concrete there are usually a-priori cracks present. These cracks are caused by bleeding, capillary or drying shrinkage or by thermal stresses. Unsufficient compaction and deformation at early age also may cause cracks. If an external load is applied cracks may propagate. Therefore on the mezzolevel and on the macrolevel extension of existing cracks has to be studied.

First of all we have to investigate branching cracks starting from an arbitrarily inclined crack in a homogenous plate. In Fig. 6 a crack with length $2\,l_1$ and an inclination α with respect to the direction of an external load is shown. The shear stress T acting along the inclined crack can be expressed in the following way:

$$T = 2\,l_1 \cdot q\,(\sin\alpha\cos\alpha - \rho\sin^2\alpha) \tag{7}$$

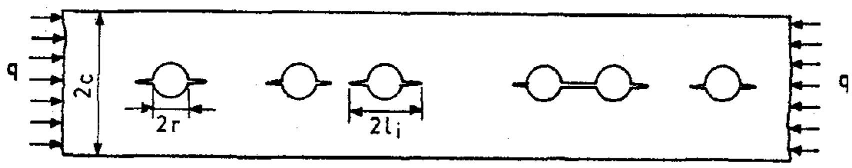


Fig. 4: Simplified simulation of a porous structure with randomly distributed pores.

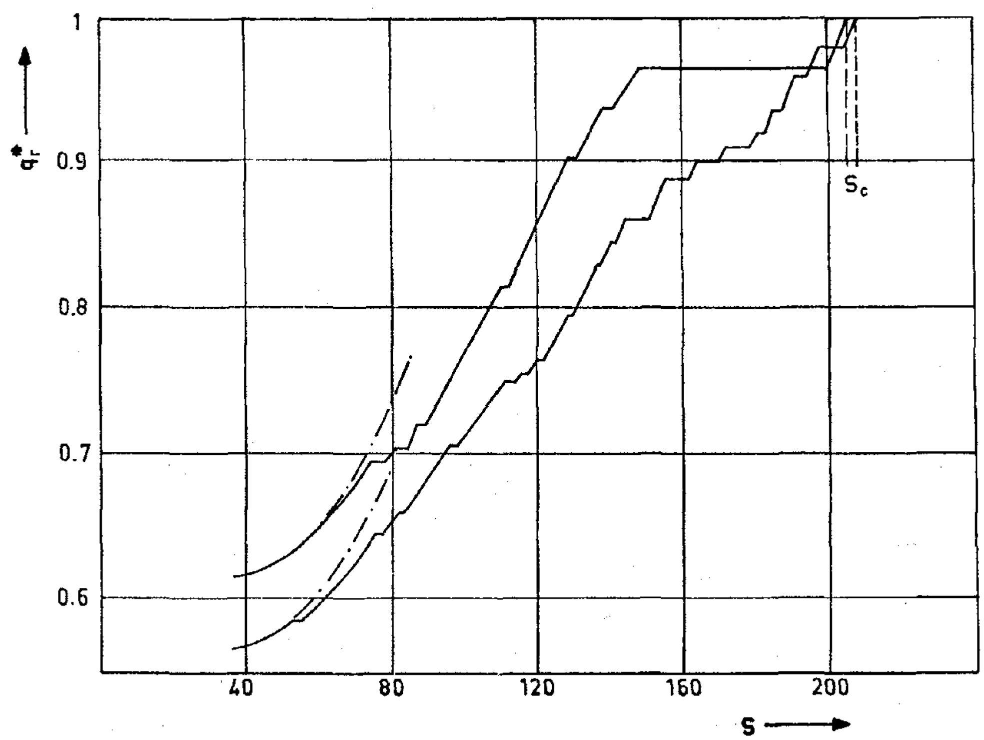


Fig. 5: Total crack length S (see equ. (6)) as function of load. Two runs of randomly distributed pores are shown. The means distance of the pores has been chosen to be four times the radius (r=1) and a sample with 100 pores has been studied by this computer simulation.

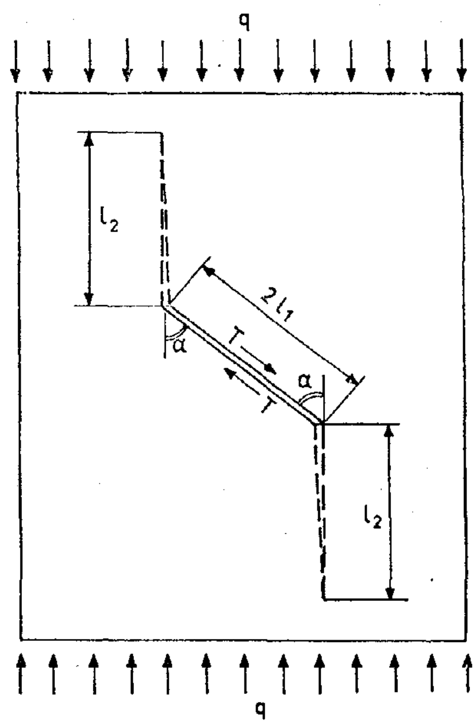


Fig. 6: Schematic representation of development of branching cracks and definitions of symbols used in corresponding equations.

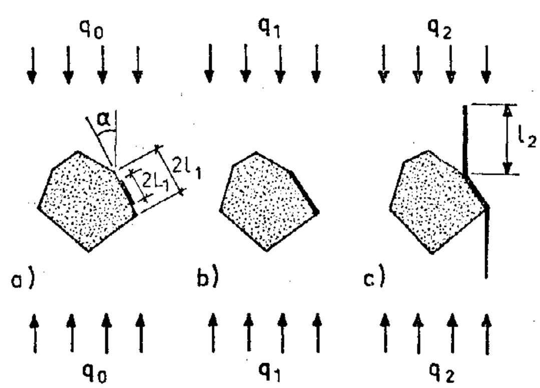


Fig. 7: Schematic representation of crack growth starting from an interface: (a-b) initial crack grows unstable along the interface: (b-c) stable branching cracks are developed.

where ρ represents the coefficient of friction. The x component (perpendicular to the direction of stress) of the shear stress T at the ends of the crack is:

$$P = T \cdot \sin \alpha \tag{8}$$

and with equation (7) P becomes:

$$P = 2\, l_1\, q(\sin^2\alpha \cos \alpha - \rho \sin^3 \alpha) \tag{9}$$

It is useful to introduce the following abbreviation:

$$A(\alpha,\rho) = \sin^2 \alpha \cos \alpha - \rho \sin^3 \alpha \tag{10}$$

With this expression the horizontal component P may be rewritten:

$$P = 2\, q\, l_1\, A(\alpha,\rho) \tag{11}$$

By introducing some simplifying assumptions it may be shown that the length of the two branching cracks l_2 can be expressed as function of P by the following implicit equation:

$$P = K_{IC} \sqrt{\pi\, l_2} \tag{12}$$

From equations (11) and (12) follows

$$q = \frac{K_{IC}}{A(\alpha,\rho)} \; \frac{1}{2\, l_1} \; \sqrt{\pi\, l_2} \tag{13}$$

or in a slightly modified form:

$$q = \sqrt{\frac{l_2}{l_1}} \; \frac{K_{IC}}{2A(\alpha,\rho)} \; \sqrt{\frac{\pi}{l_1}} \tag{14}$$

Equation (14) has been found to be in good agreement with experimental findings (6). Crack propagation of this type may be characteristic for failure of a structure on the mezzolevel i.e. mortar with preexisting cracks. In concrete, however, the interface between matrix and aggregate has to be taken into consideration.

In Fig. 7 a polygonial aggregate is supposed to be embedded in a homogeneous matrix. There exists a crack with length $2l_1$ on one side. As has been shown earlier (7), in this case the crack will spread along the interface if a critical load q^{IF} is reached. This crack extension is unstable and follows mode II. The critical load essentially depends on stress intensity factor of the interface K_{IIC}^{IF}:

$$q^{IF} = \frac{K_{IIC}^{IF}}{B(\alpha,\rho)} \; \frac{1}{\sqrt{\pi\, l_1}} \tag{15}$$

where B (α,ρ) is given by:

$$B\,(\alpha,\rho) = \sin \alpha \cos \alpha - \rho \sin^2 \alpha \tag{16}$$

The now created crack with length L_1 (see Fig. 7) will further behave like an inclined crack in a matrix. Branching cracks will propagate into the matrix in a stable way if the load is increased. By using equation (13) this condition may be written as follows:

$$q = \frac{K_{IC}}{A(\alpha,\rho)} \frac{1}{2L_1} \sqrt{\pi l_2} \tag{17}$$

In concrete, cracks which penetrate into the matrix may meet another aggregate very soon. In Fig. 8 this situation is shown schematically. When the crack reaches the second inclusion further crack growth is dependent on both the inclinations of the first and second interface. The conditions for opening (I) and shear (II) mode for crack propagation can be given as follows:

$$q_I = \frac{2K_{IC}^{IF} \sqrt{\pi l_2}/L_1}{A(\alpha,\rho)\left[3\cos{}^{\beta}/_{2} + \cos\frac{3\beta}{2}\right] - 3C(\alpha,\rho)\left[\sin{}^{\beta}/_{2} + \sin\frac{3\beta}{2}\right]} \tag{18}$$

and

$$q_{II} = \frac{2K_{IIC}^{IF} \sqrt{\pi l_2}/L_1}{A(\alpha,\rho)\left[\sin{}^{\beta}/_{2} + \sin\frac{3\beta}{2}\right] + C(\alpha,\rho)\left[\cos\frac{\beta}{2} + 3\cos\frac{3\beta}{2}\right]} \tag{19}$$

where $C(\alpha,\rho)$ has the following meaning:

$$C(\alpha,\rho) = B(\alpha,\rho) \, . \cos\alpha \tag{20}$$

It is important to note that further crack growth depends also on the sign of β because sign β appears in equations (18) and (19). In fact it turns out that crack propagation is favoured if the inclinations of α and β have the same sign.

But a crack, meeting an inclusion, has a third possibility; the crack may extend through the inclusion. Whether a crack penetrates an inclusion or whether it is deviated along the interface depends on equations (18) and (19) and the condition for straight crack formation through the aggregate (see also equation 13):

$$q_I^A = \frac{K_{IC}^A}{A(\alpha,\rho)} \frac{1}{2L_1} \sqrt{\pi l_2} \tag{21}$$

With the formulae (18), (19) and (21) it is possible to calculate crack fromation in a composite material. In this connection, computer simulation methods proved to be successful.

6. CRACK PROPAGATION IN NORMAL CONCRETE

In a computer, a random structure of concrete can be simulated. In Fig. 9 an example is shown. The aggregates are distributed at random. Also the size and geometry of the polygonial aggregates are generated by means of a stochastic process. At each particle there is one interfacial crack at a randomly chosen side.

If an external load is applied and increased above a certain level according to the formulae mentioned in the previous section some a-priori cracks start growing. Cracks extend into the matrix and the merge. As the aggregates are assumed to be stronger than the matrix all cracks run along interfaces:

$$K_{IC} < K_{IC}^{A} \qquad (22)$$

Therefore condition (21) never becomes critical. Finally an inclined crack runs through the specimen. This situation is defined to be materials failure. The inclination of the fracture line is based on conditions (18) and (19) and has been experimentally verified very often. A typical crack pattern is shown in Fig. 10.

7. CRACK PROPAGATION IN HIGH STRENGTH CONCRETE

In the previous section it was supposed that the matrix is much weaker than the aggregates. Therefore cracks are deviated. In high strength concrete this is not the case. Here strength of matrix and strength of aggregate are of the same order of magnitude:

$$K_{IC} \approx K_{IC}^{A} \qquad (23)$$

Under these conditions a crack may penetrate a particle. Whether it actually penetrates or not is mainly dependent on the geometrical arrangement. The same computer generated structure as used in the example of crack formation in normal concrete is studied again. In Fig. 11 an example of crack formation in high strength concrete is shown. As can be seen with increasing load, an increasing number of aggregates is split by growing cracks. Failure is again defined from the crack pattern i.e. when the first crack is running through the whole specimen. In high strength concrete, cracks are more likely to extend along the axis of applied load. It may be mentioned here that a similar crack pattern is obtained for lightweight aggregate concrete.

8. ALTERNATIVE APPROACHES

So far we have only dealt with crack formation and propagation in concrete. There are of course other approaches to deal with micro-mechanics. Some of the alternative approaches shall be briefly mentioned.

A stochastic theory for concrete fracture has been published by Mihashi and Izumi (8). In this approach a concrete specimen is supposed to consist of many elements and each element may crack independently. The tensile stress in the material varies locally and is taken into consideration by a probability of density function.

The stochastic model indicates the probability of failure of

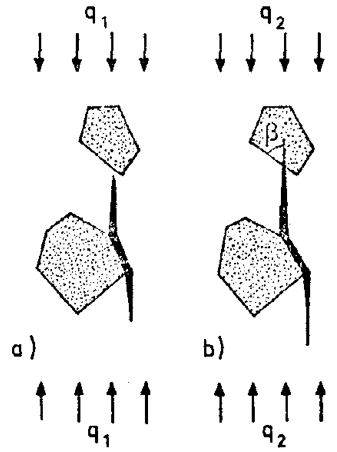


Fig. 8: A crack as created according to Fig. 7 meets a second inclusion.

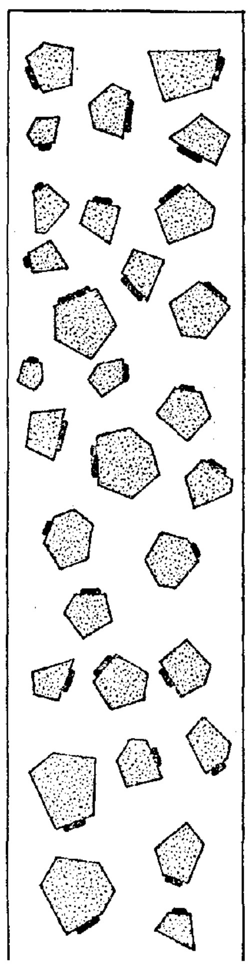


Fig. 9: Computer Simulation of random structure of concrete.

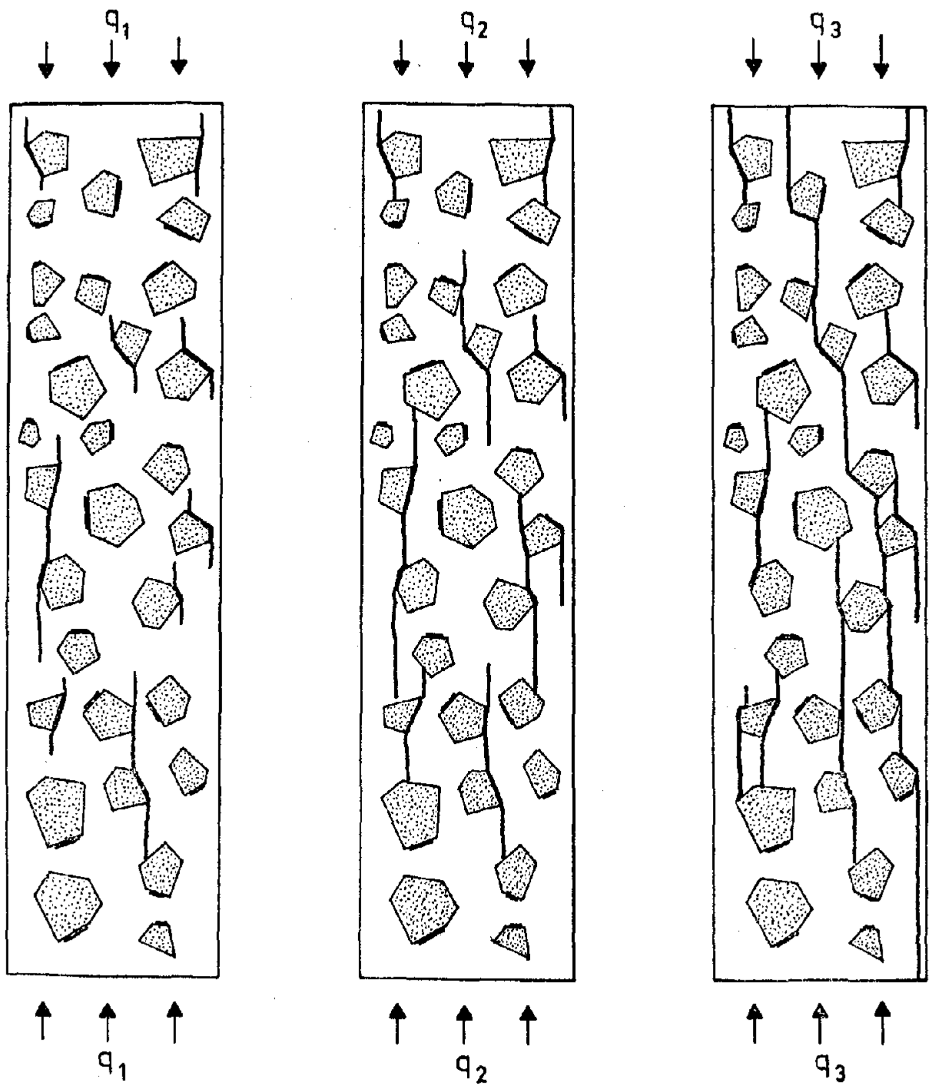


Fig. 10: Crack formation as calculated for normal concrete.

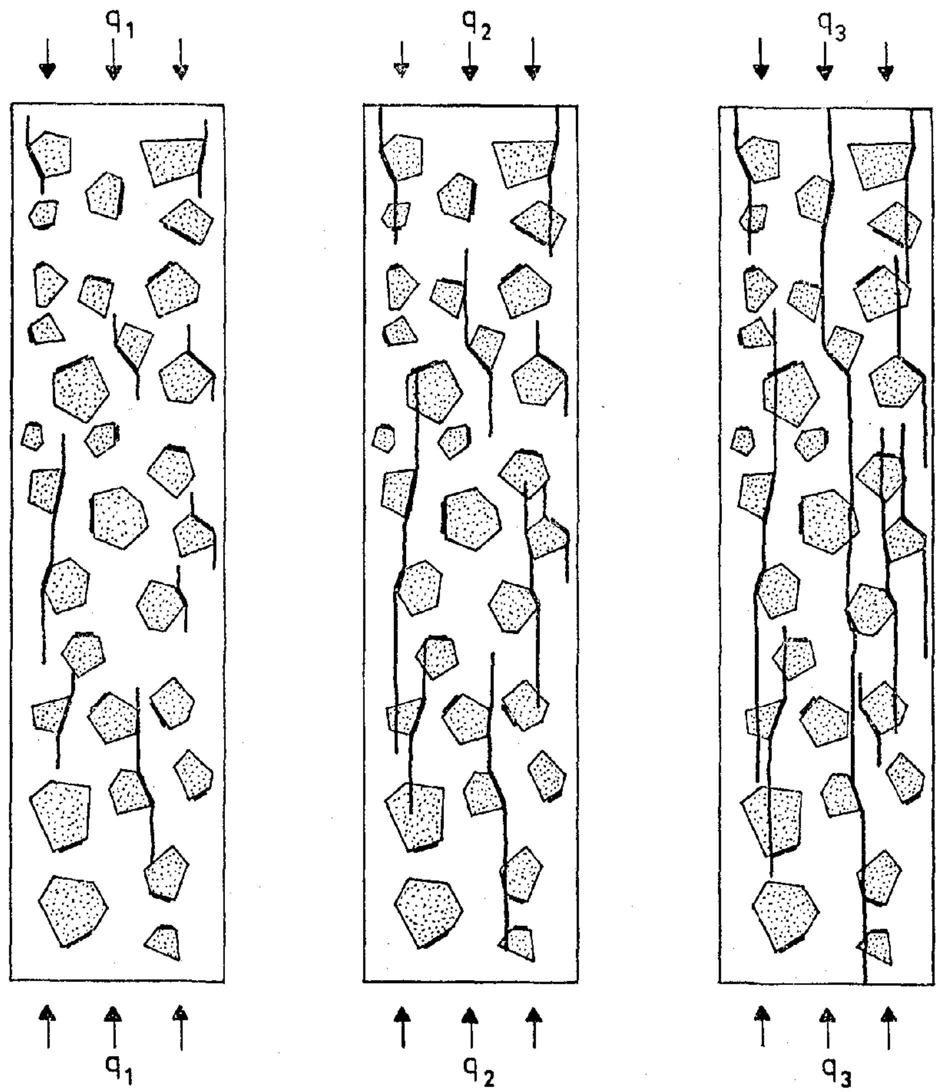


Fig. 11: Crack formation as calculated for high strength concrete.

hardened cement paste and concrete. In this way, the influence of rate of loading, temperature, and size effect on the mean value of strength can be described. In addition to this, the variance of strength can be directly linked with the structure of the material. As an example of the application of this theory the influence of rate of loading shall be briefly discussed in the following section. A comprehensive experimental study to verify the theoretical approach of Mihashi shall be published soon (9).

Based on the fact that in composite materials such as concrete many microcracks are distributed all over the loaded material far below failure, Bazant and Cedolin have suggested the method of blunt crack band propagation (10). Instead of investigating individual crack formation and crack growth, a smeared crack band with a blunt front is introduced. This approach seems to be highly efficient in finite element analysis. Cracks are smeared over a finite zone. This zone retains only the capability of transmitting stresses parallel to the crack direction. It is possible to link this analysis with classical fracture mechanics.

Another method with combined fracture mechanics and finite element analysis has been published by Hilleborg et. al. (11,12,13). Similar to the model of Barenblatt they assume that near a crack tip a plastic zone is created. Within this plastic zone stress can be transferred. The actual materials behaviour may then be characterized by choosing an appropriate function for the variation of stress with crack width. Possibly this method is especially suitable to describe failure process of fibre reinforced materials.

Mainly for the investigation of temperature induced cracks, Rösli and Harnik studied the effect of stress gradients (14,15). In a similar approach, stress relaxation has been taken into consideration (16,17). Podvalny (18) studied crack formation in a mortar layer surrounding a coarse aggregate particle. His analysis is especially suitable to study crack formation as a consequence of differential thermal expansion and due to differential shrinkage.

This is by no means a complete list of alternative approaches. But different trends in recent development have been touched.

9. EXAMPLES OF APPLICATION OF MICROMECHANICS TO CONCRETE

Two examples of application of micromechanics to investigate or describe concrete properties shall briefly be dealt with. First there will be an example to show how crack theory as described in sections 6 and 7 can be used to determine the lifetime of concrete under high compressive load. In the second example, the influence of rate of loading shall be described by the stochastic approach developed by Mihashi (8,9).

Until now we have neglected time dependence on crack growth. In Fig. 5 a critical crack length S_c is shown. If the corresponding load is reached, cracks will spread in an unstable way without further increase of load. If the load is kept constant on a level slightly below the critical load the overall crack length increases as function of time due to creep in the vicinity of crack tips. Stress corrosion and other mechanics may also contribute to crack extension. In this way after some time a critical crack length is reached below the "short term" critical load. The lifetime under high load is thus governed by the viscoelastic properties of the material and by the level of the applied load.

If the hardening process of concrete while under load may be neglected the following formula for stress f(t) which causes failure after a lifetime t, related to the short term strength f_o can be derived:

$$\frac{f(t)}{f_o} = \sqrt{\frac{t}{1+\phi(t)}} \qquad (24)$$

In this equation $\phi(t)$ stands for the creep number. If the creep deformation is high, a comparatively low lifetime under high load may be expected. In Fig. 12 the theoretical prediction is compared with some experimental results.

In the second example the influence of rate of loading shall be dealt with. After introducing some simplifying assumptions the stochastic approach mentioned above (8,9) leads to the following simple equation:

$$\frac{f_d}{f_s} = \left(\frac{\dot{\sigma}}{\dot{\sigma}_o}\right)^{1/(1+\beta)} \qquad (25)$$

f_d and f_s stand for strength under high rate of loading (dynamic) and for low rate of loading (static) respectively. The corresponding rates of loading are called $\dot{\sigma}$ and $\dot{\sigma}_o$. β is a materials constant. In (19) some experiments to determine bending strength of mortar bars are described. Results are shown in Fig. 13. β turns out to be strength dependent, it is bigger for specimens with high strength than for low quality material. That means that low strength concrete shows higher strength gain as the rate of loading increases. This is in accordance with theoretical prediction and also in accordance with extrapolations of experimental findings of lifetime under high load.

The performance of concrete in a structure is not only dependent on the mean strength. High strength is fully beneficial only, if the strength distribution is sufficiently narrow (20,21). It can be shown (1) that lower average strength together with small variability may result in more reliable structures.

10. OTHER SUPERIOR PROPERTIES

It has been shown that mechanical properties depend on the porosity of the hardened cement paste and on the structure of interfaces. In fact most other properties depend on the same parameters. Durability of high strength concrete is essentially determined by the diffusion of gases or ions through the pore structures. Therefore low permeability leads to a resistant and superior material.

There are several ways to achieve low porosity hardened cement paste. The use of water reducing agents possibly is most widely spread nowadays (22). Other ways to achieve low porosity are compacts (23) and special cements (24,25). With respect to thermal durability it should be mentioned that again pore size distribution and in addition to this water content play a dominant role. But an additional factor is differential thermal expansion of different components in composite materials, This has been studied in detail by Podvalny (18).

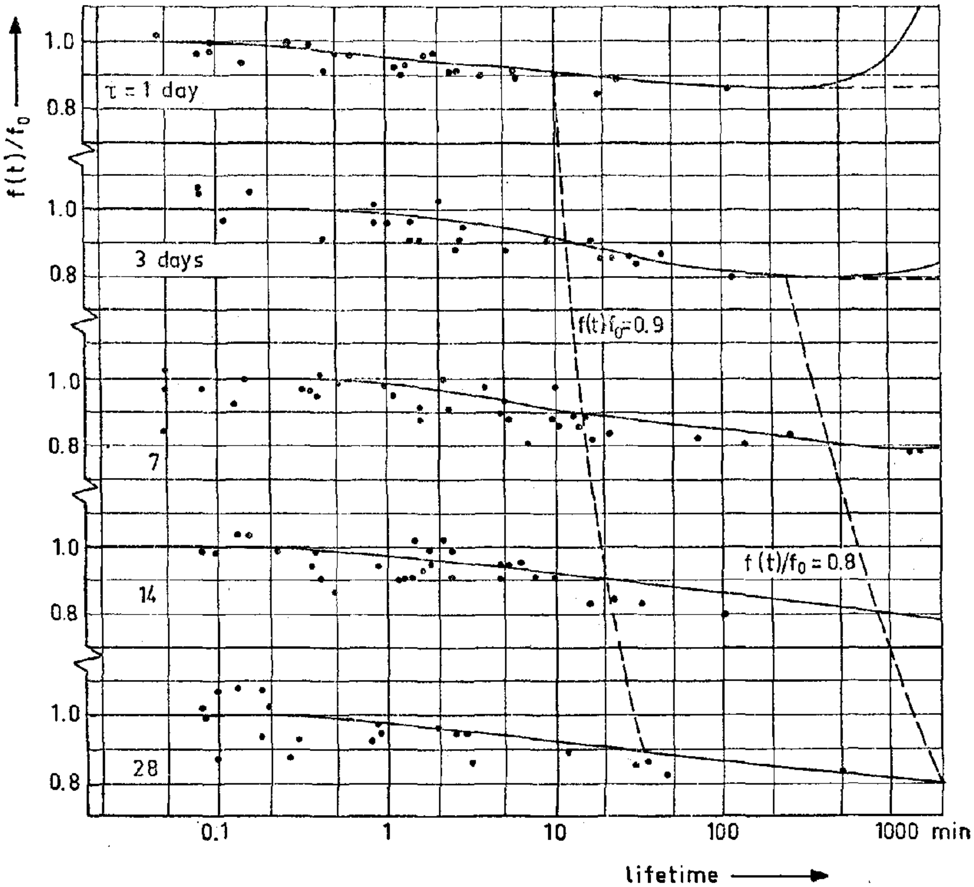


Fig. 12: Comparison of theoretical prediction of lifetime under high load with experimental results.

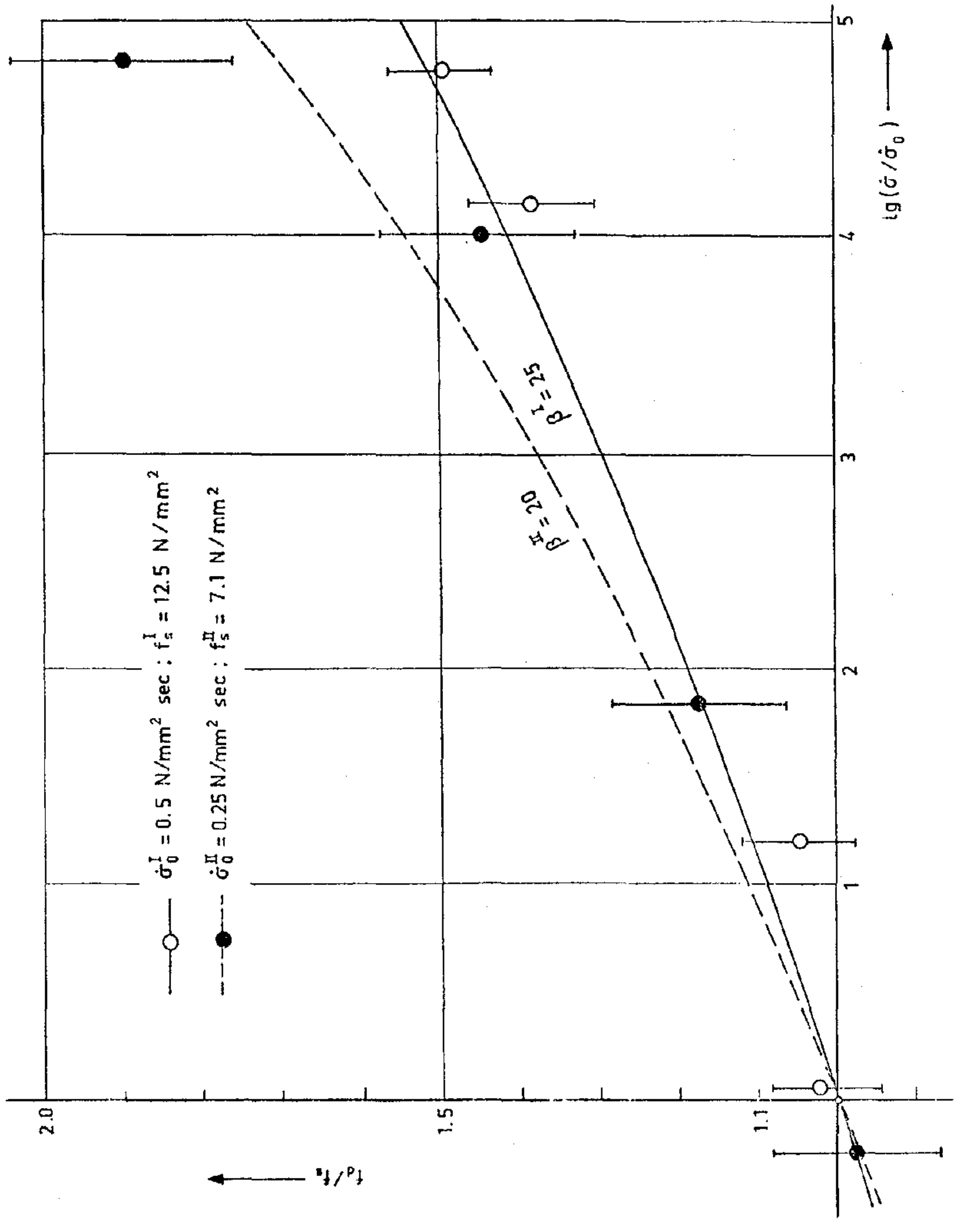


Fig. 13: Strength of two types of mortar bars with differing strength as function of related rate of loading.

Superior properties of this kind can be expected if the coefficient of thermal volume change of matrix and aggregate come close to one another.

In some cases the ultimate load is not necessarily the most important property. In fact a material which is able to absorb a lot of energy until complete failure may be superior in many applications. By adding fibres, the stress strain diagram can be changed in such a way that a basically almost brittle material becomes very ductile. Micromechanics of fibre reinforced concrete, however, have been omitted in this contribution.

Finally one superior property of concrete has to be mentioned, and that is the comparatively low price. There is, however, a drawback to this advantage because the low price hampers development. Most changes in the microstructures being reached by admixtures or by alterations in the technological process increase the price of the material considerably. Thus moderate advances often have no practical significance.

11. DISCUSSIONS AND CONCLUSIONS

It has been shown that failure of concrete is caused by crack formation in the structure of the material. Crack theory is a powerful tool to investigate crack formation and final degradation of a composite material. The observed load bearing capacity is dependent on the pore size distribution within the matrix and the structure of the interface.

Micromechanical methods can be used in two different ways. First of all it is possible to investigate and simulate materials behaviour in a realistic way. This approach finally leads to a better description and understanding of experimental findings. Secondly micromechanical analysis can be used to optimize composite materials. This latter mentioned possibility may turn out to be most important for the development of new high strength materials.

The well-known relation between strength and water-cement ratio can be interpreted by means of pore size distribution. At very low water-cement ratios the expected high strength is not reached because an increasing number of compaction pores is created due to insufficient workability. Superplasticising agents avoid the development of compaction pores at comparatively low-water-cement ratios and thus the extreme value distribution of pore size in a given sample is drastically changed. In this way high strength and other superior properties may be obtained.

Another method to reduce porosity and to avoid the presence of large pores is mechanical compaction. Compacts with strength up to 250 N/mm^2 can be easily prepared.

As durability and strength are sensitively influenced by the corresponding pore system many authors have tried to change pore size distribution by impregnation. Among other materials sulphur and a variety of polymers has been used. By impregnation the durability may be increased. Therefore impregnated concrete may be used under conditions of severe chemical attack such as bridge decks and offshore structures. Polymers have also been added to the fresh concrete. The effect usually is moderate. In some cases, however, compressive strength of up to 120 N/mm^2 has been reached.

Crack propagation is dependent on porosity and, as has been shown, on the geometrical arrangement of interfaces. Based on crack an optimization of size and shape factors of aggregates may be achieved. If it is possible to improve strength of interfaces, somewhat higher strength values may be reached (27). But the influence of properties of interfaces must not be overestimated.

Until now high strength concrete usually has been prepared on the basis of experimental test seris. A better understanding of failure process of concrete will lead to more systematic and more effective experimental procedures. At the same time an optimization based on a realistic theory may open new possibilities and indicate the inherent limits.

Based on results of a micromechanical approach some additional conclusions concerning characteristic properties of high strength concrete can be drawn:

- Strength of aggregates are decisive for ultimate load bearing capacity. In normal concrete most aggregates have sufficient strength. For use in high strength concrete aggregates have to be tested carefully.
- Crack arresting by inclusions as occurring in normal concrete are less pronounced. Therefore, until ultimate load, approximately linear elastic behaviour and nearly constant Poisson's ratio can be expected. Finally more brittle failure will occur and fracture planes will run parallel to external load.
- Low porosity hardened cement paste and comparatively small amounts of hardened cement paste will cause small creep and shrinkage deformation of high strength concrete. Because of the same reason the damping capacity will be reduced and static as well as dynamic fatigue load will be increased. Durability will be increased.
- High rate of loading increases strength of high strength concrete not to the same extent as that observed in normal concrete. The probabilistic approach to describe micromechanics of porous materials clearly points this out.

Computerized numerical analysis demands a better understanding of materials properties. A phenomenological description has to be replaced by analytical expressions. High strength concrete is a comparatively new material which is being manufactured for special applications. Micromechanics of concrete at this moment is not yet fully developed, but a significant contribution in achieving high strength and other superior properties may be expected.

LITERATURE

1. B. Zech and F.H. Wittmann, "Probabilistic Approach to Describe Behaviour of Materials", Nucl. Eng. Design 48, 575-584 (1978)

2. W. Czernin, Zementchemie für Bauingenieure 3rd edition, Bauverlag (1977)

3. F.H. Wittmann, "On the Action of Capillary Pressure in Fresh Concrete", Cem. Concr. Res. 6, 49-56 (1976)

4. F.H. Wittmann and J. Zaitsev, "Verformung und Bruchvorgang poröser Baustoffe bei kurzzeitiger Belastung und Dauerlast", Deutscher Ausschuß für Stahlbeton, Report No. 232 (1974)

5. J. Zaitsev, Cem. Concr. Res. 1, 123, 329 and 437 (in Russian)

6. F.H. Wittmann and J. Zaitsev, (to be published)

7. J. Zaitsev and F.H. Wittmann, "Crack Propagation in a Two-Phase Material such as Concrete", Fracture 1977, ICF 4 Waterloo, Vol. 3, 1197-1203 (1977)

8. H. Mihashi and M. Izumi, "A Stochastic Theory for Concrete Fracture", Cem. Concr. Res. 7, 411-422 (1977)

9. F.H. Wittmann and H. Mihashi, "Stochastic Approach to Study the Influence of Rate of Loading on Strength of Concrete" (to be published in Heron 1980)

10. Z.P. Bazant and L. Cedolin, "Blunt Crack Band Propagation in Finite Element Analysis", Journal of Eng. Mech. Div., Proc. ASCE 105, No. EM 2, 297-315 (1979)

11. A. Hillerborg, M. Modéer and P.E. Petersson, "Analysis of Crack Formation and Crack Growth in Concrete by Means of Fracture Mechanics and Finite Elements", Cement. Concr. Res. 6, 773-782 (1976)

12. A. Hillerborg, "A Model for Fracture Analysis", Division of Building Materials, The Lund Institute of Technology, Report TVBM-3005 (1978)

13. M. Modéer, "A Fracture Mechanics Approach to Failure Analysis of Concrete Materials", Division of Building Materials, The Lund-Institute of Technology, Report IVBM-1001 (1979)

14. A. Rösli and A.B. Harnik, "Temperaturmessungen beim Auftauen von vereisten Betonplatten mittels Tausalz", EMPA-Untersuchungsbericht Nr. 31, (1974)

15. A.B. Harnik and A. Rösli, "Temperaturschock und Eigenspannungen in Beton unter Frost-Tausalzeinwirkung", Bericht der Abteilung für Materialwissenschaften der ETH Zürich (1974)

16. F.H. Wittmann, O.W. Knappe and A. Schuhbauer, "Temperature Induced Stresses in Concrete", Cem. Concr. Res. 8, 703-710 (1978)

17. F.H. Wittmann, H. Schwanke and A. Schuhbauer, "Influence of Composition on Temperature Induced Stresses in Asphalt Concrete Mixtures", Cem. Concr. Res. 9 , 497-500 (1979)

18. A.M. Podvalny, Zeitschrift für Ingenieurphysik 25, 361 (1973)

19. B. Zech and F.H. Wittmann, "Influence of Rate of Loading on Strength of Concrete", (to be published)

20. M. Collepardi and M. Corradi, "High Strength and Reliable Concretes", Silicates Industriels 44, 13-22 (1979)

21. O.W. Knappe, G.I. Schuëller and F.H. Wittmann, "On the Probability of Failure of a Reinforced Concrete Beam in Flexure", Proc. Second. Int. Conf. Application of Statistics and Probability in Soil and Structural Engineering, Aachen, Sept. 1975, Vol. 1 pp. 153-170

22. F.H. Wittmann, "Influence of Water Reducing Admixtures on the Rheological Behaviour, Capillary Shrinkage and Structure of Cement Paste, Silicates Industriels 44, 5-12 (1979)

23. D.M. Roy and G.R. Gouda, "Optimization of Strength in Cement Pastes", Cem. Concr. Res. 5, 153-162 (1975)

24. S.O. Oyefesobi and D.M. Roy, "Effect of Admixtures on Hot-Pressed Cements", Cem. Concr. Res. 7, 165-172 (1977)

25. S. Brunnauer, M. Judenfreund, I. Odler, R.Sh, Mikhail, J. Skalny, J. Hagymassy, E.E. Bodor, and K.M. Hanna, "Hardened Portland Cement Pastes of Low Porosity" Part I-VI, Cem. Concr. Res. 2, 313-330, 331-348, 463-480, 731-743 (1972); 3, 129-147 (1973)

26. S. Diamond and C. Gomez-Toledo, "Consistency, Setting and Strength Gain Characteristics of a "Low Porosity" Portland Cement Paste", Cem. Concr. Res. 8, 613-622 (1978)

27. G. Rehm, P. Diem and R. Zimbelmann, "Technische Möglichkeiten zur Erhöhung der Zugfestigkeit von Beton", Deutscher Ausschuß für Stahlbeton, Heft 283 (1977)

SESSION I - DISCUSSION

MICROMECHANICS OF ACHIEVING HIGH STRENGTH AND OTHER SUPERIOR PROPERTIES

by

Colin B. Brown
Department of Civil Engineering
University of Washington
Seattle, Washington 98195

ABSTRACT

The use of fracture mechanics in the study of superior concrete is dealt with in terms of the local phenomena and the ability to utilize local results to predict phenomological properties. The matrix cracking with the load increases and difficulties in the measurement of fracture parameters may make the modelling for the application of conventional fracture mechanics a difficult process. Alternative approaches suggested by ice mechanics are presented as well as the use of entropy in the statistical description of the crack density. The gross properties such as stiffness, strength and creep should be predictible from the micromechanics. However, without successful modelling of the matrix such predictability proves to be elusive. In particular, the relation between the probability distribution of the crack density and the phenomonological parameters has to be worked out.

INTRODUCTION

At least two perspectives on the use of micromechanics in the study of high strength concrete are possible. An introspective view allows the comparison of local phenomena with results for normal concrete and other brittle materials. Here the paper by Professor Wittmann shows the less devious fracture routes in high strength concrete with the cracking through both matrix and inclusions.

A relationship between the micro-structure and the gross, phenomonological parameters can also be discerned from the study of micromechanics. Professor Wittmann concludes that the reduction of the porosity and the tightening of the initial pore size distribution are critical in achieving excellent concrete properties. In this manner direct ways of improving the gross parameters by the modification of the matrix should be predictable quantitatively and testable against experimental observations.

In this discussion these two approaches to micromechanics are explored in relation to high strength concrete. Also, the definition of the term high strength concrete is discussed.

INTROSPECTIVE VIEW

We can start this view by examining the development of fracture mechanics. It seemed to begin with Griffith's study of the rupture and flow of glass in 1921. Here he was concerned with a criterion for rupture and hypothesised the existence of flaws, one of which grows. Due to such growth the change of potential energy dominates the change in surface energy. Thirty years later the study was noticed and modified for application to ductile materials where the criterion for the existence of a running crack became the rate relationship

$$\delta W + \delta U \geq \delta U_s + \delta U_p \tag{1}$$

where δW is the increase in external work
δU is the decrease in potential energy
δU_s is the increase in surface energy
δU_p is the increase in plastic energy dissipation

For appropriate boundary condition $\delta W = 0$, and for metals, $\delta U_p >> \delta U_s$. However, in ceramics, including concrete, the surface and dissipative energies are of the same order. The right hand side of (1) is considered as a constitutive property of the material and we need not be too fussy in distinguishing between δU_p and δU_s. The left hand side of (1) can be obtained by the solution of appropriate boundary value problems and hence, for an equality, $\delta U_s + \delta U_p$ can be expressed in terms of K, the stress intensity factor. The solution to the boundary value problem gives the stress, σ, around the crack as

$$\sigma = K\, f(x,y,z) \tag{2}$$

where f is a function of the location of the crack in the body with the crack tip as the co-ordinate origin. A loading change alters the crack geometry and increases K, but the crack is stable for all $K < K_c$, the crack toughness factor. At $K = K_c$ the crack propagates with no load increase.

Clearly, the measurement of K_c for various crack modes would be of interest in concrete technology. Up to now, it appears that a wide scatter in K_c is reported from tests. Apparently, the critical crack has a propagating precursor region of stable micro-cracks and the large spread of results may be associated with the specimen dimensions. For reproducible results the specimen dimensions must be large compared to this region. This may be analagous to the plastic effects at the crack tip which occur in sea ice. In such a situation the plastic zone radius is small compared to the ice thickness and fracture mechanics predicts the usual failure modes [1,2]. An additional step by Mukherji [3] has modelled the sea ice in a finite element scheme where he works out δU in (1) and successfully predicts experimental values of K_c. Needless to say, the thermal gradient and ablation problems make ice a more tricky material than mature concrete.

The ideas of using fracture mechanics to study the local behavior of brittle materials appear to be reasonably hopeful - at least they are a success in ice. The real trouble seems to be associated with the modelling rather than the analytical operations. The presence of initial flaws so necessary for the application of Griffith's work has been established by the Cornell studies. However, in normal concrete the crack density increases in regions of tensile stress when the loading state is about 60% of failure. These new cracks have to be modelled in any incremental study of critical crack growth. Additionally, many potentially critical cracks in the matrix are snubbed off by the inclusions and the final fracture surface adopts a devious path. For high strength concrete Professor Wittmann's paper suggests that the cracks through the inclusions are as likely as in the matrix and that the fracture surface adopts a direct path. In this sense high strength concrete can be modelled as a homogeneous material. These differences in the concrete paths for the two concretes are observed in experiments.

To model the changing load dependent crack characteristics of normal concrete in a micro-study appears to be frustrating. An effort to average the crack density rather than consider individual micro-cracks has proved fruitful in ice [3] and has been utilized by Hawkins, Wyss and Mattock [4] for the prediction of failure characteristics in concrete beams. It does mean dealing with changes of potential energy, δU, in obtaining K and eventually K_c in the form

$$K = \sqrt{\frac{\delta U}{\delta A}}\, L \tag{3}$$

where δA is the increase of the critical crack surface area and L is arrangement of mechanical properties of the material. This averaging approach may upset the fracture mechanics purist but it does seem to predict actual observations and may be applicable in high strength concrete. However, in such high strength concrete the load associated with a change in crack density is amplified to some 90% of the failure state. Professor Wittmann outlines a scheme to account for the crack change in the matrix which depends on the pore density and distribution. This may lead to the same consequences as the cruder modelling in [4].

Any effort to discuss the pore and crack densities requires the use of some probability distribution in the modelling. Such a use may be prescribed from experience or other subjective insight. For unbiased probabilities the methods of Jaynes as developed by Jowitt and others [5,6] should prove attractive. Essentially they ask for an unbiased distribution p_i, for various moments and limits of the countable data, say crack density, The unbiased distribution is the one that maximizes the entropy

$$E = - \sum_i p_i \operatorname{Ln} p_i \tag{4}$$

subject to the moment and limit constraints. At least by using this procedure, modelling does not lean on experience and, given the same crack counts, should lead to the same probabilities for all investigations.

GROSS PROPERTIES

Probably even more important than improving the strength of concrete has been the increase of the reliability. One of the troubles with ceramic materials is that with coefficients of variation of 0.2 to 0.3 little use can be made of high mean values of strength. In concrete this figure has been lowered to below 0.1 and the economic advantages of high strength can be realized in locations where excellent aggregates are available. It is reasonable to ask how the study of micromechanics can be utilized to attain superior quality control? This requires at least that the gross properties be related to the micro properties and, hopefully, that long-term gross properties be predicted from micro studies of the wet or green concrete. It seems that these gross, phenomological properties still have to be determined from conventional cylinder tests and that the micro studies have not proved quantitatively too helpful.

The mechanics of composite materials includes the prediction of gross properties from knowledge of the characteristics of the constituents. This approach has been studied by Hashin, Eshelby, Hill, Budiansky and others especially with respect to moduli. Bazant has developed similar procedures for concrete. His endochronic model has proved descriptive of gross properties and his much simpler plastic-fracturing model [7] is effective for time independent constitutive relations. Presumably the second, simpler model is more attractive in the sense of Occam's Razor and may be a useful bridge between micromechanics and gross behavior. Be that as it may, real questions remain. First, increasing the concentration of inclusions increases the strength and stiffness up to a critical concentration value. Then any increase in the concentration produces lower gross parameters [8,9]. To explain this phenomonon and to predict the optimum inclusion concentration should be within the province of micromechanics. Second, the decreasing of the coefficient of variation of geometric features, such as inclusion concentration or micro cracks, does not have the same effect on the gross parameters [10,11]. The relationship between the geometric statistics and the gross statistics could receive some help from micromechanics. Third, the use of proof loading, even though the introduction of dead loads, may truncate the lower extreme of the strength probability distribution but may also introduce microcracking and damage. If the concrete is not fully cured these cracks may partially heal. Correct micro modelling and mechanics could reveal the best time for the introduction of dead or proof loading.

DEFINITION

Professor Wittmann's paper indicates that high strength concrete may display markedly different phenomenological features compared to normal concrete. From an engineering view these differences may be critical and lead to separate paradigms for decision making. The strength level may not be the measure that allows the easy separation of these phenomona. From a micromechanics vantage high strength concrete may be defined as

a) having direct non-discrimatory fracture surfaces through matrix and aggregate

b) having an invariant matrix description until 90% of the strength

c) having little ductility

These three features have practical consequences which separate high strength from normal and light-weight concrete. For this reason, the existence of these three observable features must be included in a definition of high-strength concrete.

REFERENCES

1. Brown, C. B., AIDJEX Results on Arctic Ice Mechanics, Journal of the Structural Division, ASCE, 104, ST2, 1978.

2. Mohaghegh, M. M., Fracture of Sea Ice Sheets, Offshore Technology Conference, Paper No. OTC 2046, 1974.

3. Mukherji, B., Crack Propagation in Sea Ice, Arctic Ice Dynamics Joint Experiment Bulletin, 18, 1973.

4. Hawkins, N. M., A. N. Wyss and A. H. Mattock, Fracture Analysis of Cracking in Beams, Journal of the Structural Division, ASCE, 103, ST5, 1977.

5. Brown, C. B., The Use of Maximum Entropy in the Characterization of Granular Media, Proceedings, U.S.-Japan Seminar on Continuum, Mechanical and Statistical Approaches in the Mechanics of Granular Materials, Sendai, Japan, 1978.

6. Jowitt, P. W. and J. Munro, The Influence of Void Distribution and Entropy on the Engineering Properties of Granular Media, Proceedings, Second International Conference; Applications of Statistics and Probability in Soil and Structural Engineering, Aachen, 1975.

7. Bazant, Z. P., and S. S. Kim, Plastic-Fracturing Theory of Concrete, Inelastic Response of Normal, Lightweight and High Strength Concrete, ASCE Preprint 3431, 1978.

8. Ishai, O., Influence of Sand Concentration on the Deformations of Mortar Beams under Low Stresses, Journal of the American Concrete Institute, 58, 5, 1961.

9. Brown, C. B. and M. Mostaghel, Modulus and Strength of Reinforced Matrices, Critical Values of Inclusion Concentration, Journal of Materials, 2:1, 1967.

10. Brown, C. B., Minimum Volumes to Ensure Homogeneity in Certain Conglomerates, Journal of the Franklin Institute, 279:3, 1965.

11. Brown, C. B., Models for Concrete Stiffness with Full and Zero Contiguity, Proceedings, International Conference on the Structures of Concrete, London, 1965.

SESSION I - SUMMARY OF FLOOR DISCUSSION

MICROMECHANICS OF ACHIEVING HIGH STRENGTH CONCRETE

By

Sidney Diamond
School of Civil Engineering
Purdue University
West Lafayette, IN 47907

ABSTRACT

This contribution consists of a brief summary of the micromechanical model for high strength concrete which formed the substance of the presentation of the invited speaker of Session I of this Workshop, followed by a summary and interpretation of the open discussion that followed.

INTRODUCTION

In reporting discussions of the present type one needs to always keep in mind the distinction between the real material and any particular model of it that may be adopted for purposes of analysis or discussion. I have attempted to do this consistently, but in a few cases the distinction was not quite clear and distortions may result. The writer apologizes in advance for any misinterpretations of the comments made, especially any that are attributed to specific individuals.

Concrete is a real material comprising a matrix of hardened portland cement paste with its associated pores, cast around sand grains, and in turn spaced out with coarse aggregate particles varying in size from some designated maximum down to the size of the sand grains themselves. The concept of the existence of a homogeneous mortar into which single-sized coarse aggregate pieces are embedded is a frequent simplification that is helpful in modelling but is not a very close approximation to the truth. High strength concrete (HSC) is a real material differing from normal concrete (NC) because somebody has taken the trouble to (1) select good, i.e. strong and stiff, aggregate (2) procure a portland cement of superior strength-producing capability (3) design an appropriately rich mix (4) adopt some means of insuring than an appropriately low water:cement ratio is used, and (5) see to it that this potentially high strength concrete mix is properly placed, consolidated, and cured so that the potentially high strength available is actually achieved. There are other possible ways of getting at what one would call HSC, but this is the route that has been taken in practice so far. Conventional HSC is different from NC only in certain details reflecting the quality of its components and of the mix proportions.

While a number of models exist for description of the micro-mechanical behaviour of concrete, almost the sole basis for the present discussion was that presented at this conference by Wittmann (1). It seems appropriate to summarize the development of this model and its application to HSC in qualitative terms below.

The model may be said to represent a combination of a special development of linear elastic fracture mechanics with a Monte-Carlo computer simulation technique specially adapted so as to provide a running picture of crack initiation and propagation in the hypothetical concrete under discussion, which is assumed to be loaded in monatonically-increasing uniaxial compression. The development is, roughly speaking, along the following lines:

1) The initial step is consideration of the response to one-dimensional compressive loading of a single spherical pore in a homogeneous infinite matrix, the response being that a crack is induced from the pore oriented along the compressive axis and increasing in length in a stable fashion with increasing load.

2) The single pore is then replaced by a string of pores all of the same size but with randomly-assigned spacing, and it is shown that for such an assemblage cracks emanating from the individual pores coalesce: when the combined length reaches a critical value further unstable propagation occurs and failure is produced.

3) Next, consideration is given to an arbitrary inclined crack in a homogeneous plate still under uniaxial compression, and it is shown that extensions ("branches") develop from either end and propagate in a stable manner along the compressive axis in response to increasing load. At some critical load, which depends on the angle of inclination and on the stress intensity factor K_{IC} for the matrix, the critical length will be exceeded and the crack will propagate in an unstable manner along the compressive axis and cause failure.

4) At this point aggregate is introduced for the first time. A single aggregate particle-in-mortar is examined, and it is assumed that a partial linear "bond" crack exists along one portion of the aggregate mortar interface inclined at an arbitrary angle to the compression axis. It is shown that under load such a crack will grow in an unstable manner until it exceeds the projected length of the aggregate. It will then behave as an inclined crack in a homogeneous matrix as described under 3) above; that is, it will develop axially-aligned "branches" and propagate in a stable manner under increasing load. Presumably it will coalesce with any matrix crack that it approaches in the manner indicated in 2) above. The crack

will continue to grow until it gets large enough to become unstable, and failure will occur.

5) Finally, a more complete model is expounded in which many aggregates are randomly spaced in the "mortar" matrix. The cracking process is as previously considered up to the point where any stably-propagating crack encounters a second aggregate. It is shown that depending on fracture mechanics criteria and the angle of inclination of the second interface with respect to the compressive axis and also to the first aggregate's inclination, the crack may stop, go around the second aggregate, or penetrate through it without deviation. The equations that govern which of the possibilities are followed in a particular case are appropriately developed, and the whole scheme is simulated on a computer run in which the aggregates of arbitrary geometry are placed initially in random positions and size and positioning of initial bond cracks specified.

The distinction between HSC and NC in the model rests on whether the "strength" (K_{IC}) of the matrix is assumed to be substantially the same as, or substantially less than, the corresponding value for the aggregate. If less, the cracks go around most aggregates and final failure is developed as a result of inclined cracks. If the two stress intensity factors are of the same order some cracks propagate directly through the aggregates without deviating from the compressive axis and failure is through development of cracks aligned with this axis. These distinctions are in accord with those experienced in practice with real concrete.

Wittmann remarked on implications with respect to response in terms of expected lifetimes at high loadings and of rate of loading effects drawn from the model. However, he did not show numerical illustrations in which specific geometries and K_{IC} values were inputted into the model and the resulting estimates of failure strengths provided; nor were expected deflections and Poisson's ratios discussed.

Some discussion of alternate models based on other extensions of fracture mechanics were provided by Wittmann, and by Brown in his prepared discussion (2). These considerations of other models were relatively pro forma, and the other models were not described in detail nor compared with the Wittmann model in any meaningful way. This is understandable in terms of space and time restrictions, but in the writer's view is regrettable.

SUBSTANCE OF THE DISCUSSIONS

There was comparatively little floor discussion of the specific features of the model presented in detail by Wittmann.

One of the few significant points raised in response to the specific features presented was the questioning by Ingraffea of the assumption that cracks running in a stable mode in the same direction would coalesce; he suggested that cracks propagating in a stable manner in a compression field do not tend to coalesce, but in fact repel each other. In his view the unstable cracks that determine the fate of the system are those developed originally as secondary cracks within the matrix, rather than cracks projecting from originally-existing bond cracks. Bazant subsequently concurred that there may be a tendency for parallel cracks to go past each other in the homogeneous stress field assumed.

The relative importance of the initially occurring bond cracks assumed by Wittmann was minimized by Darwin, who reported results of experimental work in which all coarse aggregate grains had been coated with a thin but mechanically competent "bond breaker", which would have provided the equivalent of essentially complete bond cracks existing in the initial state. The effect on uniaxial compressive strength was only on the order of a 10 percent reduction. It was indicated by others that the influence of bond in other than uniaxial compressive loading may be more significant.

Some details of other models of concrete were discussed, stemming from mention by Darwin of finite element models being used to describe biaxial as well as uniaxial loading responses. The feature discussed most extensively was the large apparent Poisson effect at high strains, and the variation of Poisson's ratio with percentage of ultimate load for HSC and NC. There was some discussion as to how stress criteria could suitably be introduced into such models, and some concern with the effect of mesh size.

In further discussions of the Poisson effect it was indicated that the relative stiffness of matrix and mortar seemed to exercise the controlling influence. Some of the discussion ensued from brief mention of a model by Nilson which involved simulation of a specific concrete in which the modulus of the aggregate was assumed to be twice that of the paste matrix. It was indicated that in such circumstances shear compression cracking at portions of the aggregate-matrix interface inclined at approximately 30° to the compression axis developed early; some of these cracks were eventually transformed to vertical tensile cracks, in a manner analogous to that of Wittmann's model. Ward raised the question of how Poisson's ratio would behave in such a model, but it was indicated that the specific model concerned was too idealized to examine in this context. Darwin indicated that in some models with which he had been concerned, separate input values of Poisson's ratio of 0.20 and 0.26 for the mortar and aggregate respectively had yielded gross output values of as high as 0.5 for the stress levels just preceding failure.

A parallel and intertwined theme of discussion involved consideration of the physical basis for the achievement of HSC, especially the influence of porosity and pore size distribution. These considerations were originally raised by Wittmann in his primary report (1), and reinforced in the beginning of the discussion period by the present writer. Slides were shown illustrating the influence of water:cement ratio and the lesser influence of degree of hydration on intruded porosity and on pore size distributions in portland cement pastes. The water:cement ratio was seen to largely determine the pore volume that would be left after reasonable hydration, and also the maximum pore size and the trend of the size distribution. It was indicated that attainment of HSC depends on limiting the water:cement ratio to as low a value as possible. In current versions of HSC this is often done with the aid of a super water-reducer, which has the effect of overcoming the flocculating effect of gypsum present in the portland cement; the resulting dispersed fresh paste structure exhibits a much reduced water demand. Alternative "low porosity" systems were described which rapidly attain high strength by substituting lignosulfonate/alkali carbonate set control and dispersing admixtures for gypsum. Development work on such systems has been done by a number of organizations, although commercial acceptance has not yet been forthcoming.

In further comments along these lines Klemm indicated that the durability of HSC made with superplasticizers has been questioned because of difficulties with air entrained bubble systems, but the importance of freeze-thaw durability in most current applications of HSC was questioned by Clifton. Ward commented on the difficulty of introducing products based on new concepts into the conservative building materials industry.

Some discussion ensued concerning the sensitivity of HSC to mix proportioning and job-site variations, Parrott suggesting that in his experience HSC is actually less sensitive to such variations than is NC. Bertero demurred, suggesting that maintenance of quality is easier at low strength levels.

The question of pore structure was returned to in a discussion by Young, who suggested that at the low water:cement ratios necessary for the production of HSC factors other than the pore structure may limit strength. The importance of matrix type in cement paste strengths was illustrated by a close relationship between compressive strength and mean pore radius, the latter being a function of the type of matrix material in the specific comparisons depicted. Young also called attention to the possible importance of the relative amounts of crystalline and amorphous material in the paste at low water:cement ratios for both intrinsic strength and bond strength characteristics, and suggested that the ideal paste might be a system in which the (crystalline) cement grains were only partly hydrated, with the resulting (amorphous) hydration products completely filling the available space.

Further indication of the importance of porosity was provided by Pomeroy, who suggested that porosity-strength relationships form a fairly consistent pattern, and indicated that at least some of the improvement in strength resulting from polymer impregnation must be ascribed to pore filling. Tensile and flexural strength improvements are also observed and attributed to this feature. Pomeroy also discussed the probability aspects of failure in terms of the now conventional strength-critical flaw size concepts. In subsequent discussion Hope reiterated the importance of total porosity in limiting the strength that could be attained, and suggested that strength is favoured by keeping the volume of C-S-H to the minimum required.

Another interrelated theme discussed extensively in several different contexts was the relative importance and prevalence of microcracking within the matrix. It was suggested by Darwin that microcracking at comparatively low strains could produce extensive damage, resulting in non-linear "softening" of the matrix at subsequent high stress levels; he remarked that in a number of cases the expected increases for mortar "strength" under biaxial compression were not obtained, perhaps reflecting such effects. Bazant suggested that creep effects may be important in this context. Pomeroy indicated that microcracking could be detected by special techniques at stress levels very much lower than the high levels required for visible cracks to appear. Parrott also commented on the formation of extensive microcracking at low levels of stress and particularly on the apparent inability of such microcracks to grow under repeated loading cycles. He suggested that the occurrence of such non-propagating microcracks might even constitute something of a safety factor for the concrete.

The results of an extensive program of experimental work in high strength concrete being carried out at Cornell were discussed in turn by Slate, Ingraffea, and Nilson. Slate stressed the late onset of observable matrix cracking (to X-ray and microscopy at $\sim$20 X) in uniaxially-loaded HSC as compared with NC, and the comparatively limited number and volume of cracks developed before failure. Ingraffea discussed related differences in the ability of the same aggregate to arrest unstably-propagating matrix cracks in HSC and NC, most such cracks being arrested in the latter case but few in the former; however, in the former case of HSC such cracks are observed to develop only at high stress levels both absolutely and in terms of percentage of ultimate strength. Nilson, completing the presentations by the Cornell group, described the general similarity between the computer simulation model indications of Wittmann and the results of both experimental and model studies being done at Cornell. For HSC the relatively high percentage of ultimate strength that is reached before significant cracking occurs, the almost constant Poisson's ratio up to that point, the vertical crack pattern at failure all relate to the similarity in modulus that is present between the matrix and the aggregates in HSC. Naturally, to attain this characteristic HSC response the aggregate must be capable of matching the superior properties of the high strength and high modulus matrix that is being produced. A similar effect at a much lower strength level would be expected for "ordinary" matrices and weaker lightweight aggregates.

Bazant concluded his contributions to the discussion with brief mention of several disparate points. He called attention to a possible wedging action that might occur in local areas in placing "simulated" aggregates randomly; to the fact that elongated aggregates placed with long axes transverse to the compressive axis may increase uniaxial compressive strength significantly; and to the fact that the character of the aggregate gradation, especially gap grading, may influence strength. Attention was called to the abruptness of the changes produced in HSC on attaining the general level of the order of 90% of ultimate strength, in contrast to the more gradual changes taking place in NC as the load increases. Finally, he indicated that in his opinion far too much attention was being paid to the uniaxial behaviour of HSC; in the normal applications of such concrete there is much use of steel to restrain lateral expansion and a more complex stress state exists. This last comment was echoed by Nilson, who also indicated that in addition to the familiar use of HSC in lower columns of tall buildings, important potential uses in long-span bridges mandated much more attention to flexural behaviour of HSC than has been provided.

In final generalizations, Brown called attention to the somewhat paradoxical fact that the mechanics of HSC more nearly resemble those of an ideal brittle material and in consequence HSC should be somewhat easier to handle theoretically than NC; Wittmann expressed his agreement with many of the points previously raised, but indicated that while in some cases propagating cracks avoid coalescence, in the idealized situation they will coalesce and thus produce unstable crack propagation at lower stresses than would otherwise obtain. Frohnsdorff, in his summarizing remarks as Chairman, called attention to the fact that there are a number of different possible ways to "engineer" high strength matrices for HSC, portland cement being only one of the possibilities, and suggested that the others not be overlooked.

In the writer's opinion, the discussions were most useful in bringing out a considerable variety of aspects relating to both known and unknown features that characterize the behaviour of HSC as a material. They seemed to be at least partly successful in terms of opening up a dialogue among materials scientists and engineers, mechanics-oriented researchers, and to a lesser extent, the structural engineers present. It would have been interesting to have had the opportunity for further discussion on the materials aspects of HSC behaviour <u>after</u> the greater depth of information and analysis provided by the three subsequent sessions of the Conference had been absorbed by those present.

REFERENCES

1. F.H. Wittmann, "Micromechaniss of Achieving High Strength and Other Superior Properties", Proc. Workshop on High Strength Concrete, U. of Illinois at Chicago Circle, Dec. 1979, pp.1-22.

2. C.B. Brown, "A Discussion on the Micromechanics of Achieving High Strength and Other Superior Properties", Ibid. pp.23-27.

SESSION II - REPORT

MATERIAL BEHAVIOR UNDER VARIOUS TYPES OF LOADING

by

Kurt H. Gerstle, Professor of
Civil Engineering
University of Colorado at Boulder
Boulder, Colorado, 80309

ABSTRACT

The present state of knowledge of the mechanical behavior of high-strength concrete is summarized, and it is concluded that, with the exception of its response to monotonically-applied uniaxial loadings, little is known.

Response and strength of high-strength concrete under uniaxial compressive and tensile loadings are concisely described by reference to previous studies. Comparison of multiaxial tests of different strength concretes indicates no qualitative differences between the behavior of normal and high-strength concretes.

Areas of deficient knowledge are delineated, and some suggestions are made for future research.

The Present State of the Art

Our knowledge of the mechanical behavior of high-strength concrete (HSC) should be seen in the light of our understanding of the behavior of normal-strength concrete (NC). It is interesting to observe that after a hundred years of testing there are still vast "blanks on the map" for any type of concrete, particularly under multiaxial stress states. In view of this state of affairs, it is not surprising to find many unanswered questions about the response of HSC to loads.

To summarize, the behavior of concrete has been split up into different fields in Table 1, in each case with a subheading for uniaxial and multiaxial response. Three symbols denote the current state of knowledge for both NC and HSC, as a result of a thorough literature search which was greatly aided by a recent bibliography on HSC (1).

Most of the information about the properties of HSC was gained from test programs comprising a range of concretes from low or medium strength to higher strengths up to 80 N/mm^2 (12 ksi). For purposes of this review, concrete of cylinder strengths above 60 N/mm^2 (9 ksi) was considered HSC; besides, a number of test series with a range of strengths was carefully scrutinized for trends which seemed to be a function of strength.

Since about 1960, coincident with use of higher strength concretes in prestressing and precasting, some studies were specifically concerned with behavior and properties of HSC.

In the next section, some comments regarding HSC as a material will be made. Following this, response to uniaxial loads will be covered, after which behavior under multiaxial stress states will be discussed. Some suggestions for needed research, and a bibliography, will conclude this review.

Some Material Considerations

HSC can be achieved by a variety of different means; in this review, only properties of HSC in which high strength was obtained by decreasing the water-cement ratio will be considered. Different mix compositions will result in different material behavior, and generalizations of results obtained from tests of one particular concrete should be viewed with suspicion.

To illustrate this point, we cite the work of Hobbs (2), who studied the properties of concretes made of one type of aggregate but ranging from low to high strength. To explore just two variables, the water-cement ratio and the aggregate content, Figs. 1 (2) show the multiaxial behavior of these concretes of uniaxial strengths ranging from 20 to 80 N/mm^2 (3.0 to 12.0 ksi). From these curves we can extract information about the effect of aggregate content: a change of behavior occurs at aggregate content of between 40 and 70 percent for high water-cement ratios; this becomes less pronounced at lower water-cement ratios likely for HSC. The more common aggregate proportions shown in Fig. 2 (2) indicate uniform behavior which suggests that commercially feasible HSC may have behavior which is similar to that of NC.

High strength concrete made with lightweight aggregate seems to behave somewhat differently from that made with normal-weight aggregate, as discussed later. Conclusions drawn for one type may therefore not necessarily be transferred to another type of concrete without further study.

Failure surfaces of HSC are more likely to pass through aggregate particles because of the proportionately greater matrix strength. For this reason, aggregate strength and stiffness should be a more important parameter in HSC than in NC. However, no systematic investigations of this factor were found in the literature.

Bond cracks at mortar-aggregate interfaces are still likely to occur, but the gradually progressing microcracking through the mortar, and attendant energy absorption, will be retarded due to high mortar strength so that explosive, brittle failure is more likely in HSC than in NC.

Because of the reduced energy absorption capacity all those aspects associated with brittle failure - high strain rates, fatigue loadings, stress concentrations - need special attention; they will be addressed in the following.

Because of its brittleness, fracture mechanics may be applicable to explain the formation and propagation of cracks in HSC. The use of this method in the field of concrete is in its infancy, and its usefulness to any type of concrete is yet to be demonstrated. No use of fracture mechanics specifically addressed to HSC has been found in the literature.

Lastly, since creep of concrete is intimately related to its water and aggregate content, it might be expected that HSC with low water-cement ratio will have somewhat different creep properties than NC. This will also be demonstrated later.

Uniaxial Behavior

Attention to uniaxial response of HSC is important for at least two basic reasons: it is much easier to obtain experimentally than the more general response to combined stress states, and it provides a needed tool for the design of members of framed structures which are the "bread-and-butter" of most professional design offices. It will therefore be covered first.

Response to monotonically-increasing strains--The classical Hognestad (3) concrete stress-strain curves which show increasing stiffness, a greater linearity, and increasingly brittle behavior with increasing compression strength were extended recently in the course of an ongoing test program at Cornell University (4). Fig. 3 (4) shows stress-strain curves for concretes up to f'_c = 13.5 ksi (90 N/mm^2). The trends indicated by Hognestad are seen to continue for very high concrete strengths. The lack of descending branch for the more brittle high-strength concretes might be more a function of the testing machine than of the material: Fig. 4 (5), for instance, shows descending branches for concretes up to 80 N/mm^2 (12 ksi). Similar behavior was observed for concretes up to 13 ksi (91 N/mm^2) by Wang, Shah, and Naaman (6).

The area under the stress-strain curves indicates the energy-absorption capacity per unit volume, or toughness, of the material. Fig. 5 (5) shows this quantity for concrete strengths from 15 to 80 N/mm^2 (2.25 to 12.0 ksi). The increase of toughness lags well behind that of strength, and becomes negligible for HSC.

The increasing brittleness with higher strength is also demonstrated by the decrease of ultimate strains with increase of compressive strength shown in Fig. 6 (4). These strains were obtained from prism or cylinder tests and may be too conservative for use in beams where strain gradients may lead to larger strain capacity.

The increasing stiffness as well as the increasingly linear response with higher strength is seen from Fig. 7 (7) which shows the variation of modulus with stress for concretes of strengths from 19.1 to 61.9 N/mm^2 (2.8 to 9.3 ksi). Similar findings are shown in Fig. 8 (4), which extends these trends to concretes up to 14 ksi (90 N/mm^2). Moduli in this figure are shown to be proportional to $f_c'^{0.30}$, rather than $f_c'^{\ 0.50}$ as per A.C.I. 318-71.

Fig. 7 (7) also shows Poisson's ratios for three different concretes under increasing stress: a slight increase is noted here for higher strength; constant values of Poisson's ratio prevail over a great portion of the stress range for all concretes.

<u>Compressive behavior of lightweight concrete</u>--Wang, Shah, and Naaman (6) performed extensive tests and analysis of both normal- and lightweight concrete of strengths up to 13 ksi (91 N/mm^2) and 8 ksi (56 N/mm^2), respectively. A stiff testing arrangement allowed determination and close study of the descending portion of the stress-strain curve. Fig. 9 (6) shows a comparison of normal-and lightweight concretes of two identical strengths; the lesser stiffness and greater brittleness of the lightweight concrete is obvious. More generally, lightweight concrete seems to develop its peak strength at progressively higher strains, and shows progressively steeper-descending post-peak curves, with increasing strength.

<u>Tensile behavior</u>--All studies indicated a decrease of the ratio of tensile strength to compressive strength for higher compression strength; this is, for instance, shown in Fig. 10 (8), which shows the ratio of the compressive prism strength to the modulus of rupture for concretes of strengths from 1.5 to 10.5 ksi (10 to 70 N/mm^2). These findings are corroborated and extended to

higher strengths in Fig. 11 (4); here, the modulus of rupture is proportional to $f_c'^{0.50}$ for all strengths. Results from split cylinder tests follow similar trends (4).

The near-linear behavior of concrete under tension is shown for three compressive concrete strengths from 16.9 to 55 N/mm^2 (2.5 to 8.3 ksi) in Fig. 12 (7). Over this range, tensile and compressive elastic constants obey similar trends.

Formulation for flexural strength--Ref. (4) suggests use of a trapezoidal compressive stress-strain curve to replace the more restrictive rectangular Whitney stress block. Following this suggestion, and incorporating the findings of Figs. 3 to 8, we might consider the variable trapezoidal approximation in which the yield strain increases linearly from 2×10^{-3} to 3×10^{-3} and the ultimate strain decreases linearly from 4×10^{-3} to 3×10^{-3} as the concrete strength increases from zero to 15 ksi (100 N/mm^2). According to this approximation, which is shown graphically in Fig. 13 (4), the compression block coefficient k_1k_3 takes on values as shown by the dash-dot line of Fig. 13. It is seen that the correlation with A.C.I. 318-71 as well as with test results is good. While the continuous dash-dot curve is more rationally founded, no doubt the current specifications are somewhat easier to use in practice.

It is of interest in this context that the German specifications (9) represent the compressive behavior of all concretes without restrictions as to strength by the identical parabolic-constant stress-strain curve.

Response to load histories--Early investigations of the behavior of concrete, and concrete members, to load histories such as cyclic and variable repeated loading were usually carried out on only one strength of usually normal concrete.

Several series of fatigue tests, however, were performed on concretes of different strengths. Mehmel and Kern (10) tested four different concretes of cube strengths between 16.2 and 42.4 N/mm^2 (2.4 and 6.4 ksi), and Bennett and Muir (11) tested four concretes, of two maximum aggregate sizes and cube strengths ranging from 7.8 to 11.2 ksi (52 to 75 N/mm^2). Both investigations found minimal effect of concrete strength. Fig. 14 (11) shows the S-N curve for all concretes tested by Bennett and Muir; the non-dimensionalized fatigue strengths seem remarkably similar, but the authors note more explosive failure in the higher-strength concrete. In Ref. (10) the statement appears "concrete rich in cement is more resistant to fatigue than concrete poor in cement," but the evidence shows this effect to be relatively minor.

No studies of the effects of low-cycle fatigue or other load histories on HSC were found, but it could be expected that the increased brittleness would have to be considered in the material response.

Behavior under dynamic conditions - strain rate effects--The effect of these conditions on HSC does not seem to have been studied.

Time-dependent behavior--Little data have been found on the creep response of HSC, but a general study contains some relevant observations (12):

1. Creep increases with increasing water-cement ratio; since HSC will in general be achieved by lowering this ratio, it might be expected that it creeps less. Fig. 15 (12) and Table 2 (12) document these facts.
2. High aggregate content will tend to minimize creep at working stress levels, as shown in Fig. 16 (12).
3. Effects of aggregate properties are prominently mentioned; it might seem fair to conclude that generalizations relating creep only to compressive strength might be fraught with danger.

The decreased creep of HSC predicted by this report has been observed in preliminary tests carried out at Cornell (4). Comparison with Meyers' tests (13) of NC at a stress level about half of ultimate shows a specific creep of HSC about one fifth that of NC.

Multiaxial Behavior

Knowledge of the multiaxial behavior of HSC seems particularly necessary because its use appears specifically indicated in situations subject to complex stress states - beam-column intersections, anchorage and perforation zones of prestressed structures, reactor vessels, shell-type offshore structures, and the like. To be useful, results must be presented in such form that they can be used in analysis of such structures.

The usual engineering approach to multi-axial material response is to aim at prediction of the behavior under general stress states from experimental data obtained from uniaxial tests. For HSC, two relevant questions are: is it possible to carry out this procedure, and, if so, are the multiaxial properties basically different from those of NC? The following aims to answer these questions in the light of available information.

Response to Monotonically Increasing Loads

Strength--Experimental results of multiaxial tests must be interpreted with care. Recent studies (14) have shown that boundary constraints can lead to fictitious strength increases, and that therefore only results from tests in which friction between loading platens and specimen surfaces is minimized might be valid. Be that as it may, only results of tests with similar surface conditions should be compared.

The biaxial failure points, for instance, of Fig. 17 (15) compiled for concretes ranging from 16 to 68 N/mm^2 (2.4 to 10.2 ksi) compressive strength, cannot therefore serve to explore the effect of strength without further scrutiny because they were compiled from different test series with widely differing boundary conditions.

Another problem in multiaxial compression testing are the extreme strengths attained under triaxial compression which may exceed the capacity of the testing apparatus. For this reason, many of the tests in the literature were purposely carried out on lower strength concretes. However, in spite of this problem, considerable information is available.

Kupfer (7) carried out comprehensive tests of the multiaxial behavior and strength on concretes of three uniaxial compression strengths from 20 to 60 N/mm^2 (3.0 to 9.0 ksi), so these results can serve to compare the response of NC and HSC. The non-dimensional failure envelope of Fig. 18 (7) gives information for all three concretes on three ranges of biaxial stresses: Compression-Compression (C-C), Tension-Tension (T-T), and Compression-Tension (C-T):

C-C: All three concretes appear to obey the same failure laws.

T-T: A decrease of the ratio tensile strength to compressive strength is observed for higher-strength concrete, as already noted for uniaxial cases. Similar shapes imply that a principal tensile stress criterion seems valid for all concretes.

C-T: Percentagewise, a large difference is noted. A small amount of tension will decrease the compressive capacity more radically for HSC than for NC. Simplified strength envelopes in this range might have to vary in shape as shown in Fig. 19 for NC and HSC.

Newman, Hobbs, and Pomeroy (16) have advocated a simplified engineering approach to the prediction of concrete strength which neglects the effect of the intermediate principal stress; this permits the multiaxial strength to be represented in terms of only the major and minor principal stresses, σ_1 and σ_3. Fig. 20 (16) shows such a representation, presumed valid for concretes of cylinder strengths between 20 and 70 N/mm^2 (3.0 to 10.5 ksi).

For conditions of practice, these curves have been linearized further:

for failure under C-C, with appropriate material safety factor (16),

$$\sigma_{1u} = .67\ f'_c + 3\sigma_3$$

and for failure under C-T (σ_3 is negative),

$$\sigma_{1u} = .67\ f'_c + 20\sigma_3\ .$$

In Ref. 16, validity of these strengths is implied for concretes of strengths up to 80 N/mm^2 (12.0 ksi).

Stress-strain behavior--Kupfer's biaxial stress-strain curves (7) for concretes from 20 to 60 N/mm^2 (3.0 to 9.0 ksi) indicate uniform behavior of all concretes, as already shown in Figs. 7 and 12. These modulus curves indicate a longer elastic compressive range, but a shorter elastic tensile range for the higher-strength concrete.

The multiaxial behavior can be expressed by the variable shear and bulk moduli G and K; their variation under increasing load for the three concretes of Ref. 7 is shown in Fig. 21. The lesser decrease of these moduli for the higher-strength concrete indicates the more linearly-elastic behavior of HSC.

Kotsovos and Newman (17) performed triaxial tests of four concretes ranging from 15.3 to 62.1 N/mm^2 (2.3 to 9.3 ksi). Octahedral stress-strain curves for all concretes are shown in Fig. 22 (17), and indicate the uniformity of behavior over all strengths. The linearity of volumetric behavior up to failure appears similar for all concretes.

Further results of Kotsovos and Newman's work indicate their capacity to predict response to load histories for all concretes in a unified fashion.

Behavior under multiaxial load histories, dynamic conditions, and time-dependent behavior--No information on these topics specifically applicable to HSC was found in the literature.

Summary

Sufficient information about uniaxial response to statically applied, monotonically increasing loads appears available to formulate the behavior of beams and columns of HSC under static conditions. With this information, code provisions should be extended to cover use of HSC.

Insufficient information is available for accurate assessment of response of structures of HSC to seismic motions. Shear strength and confining effects of reinforcement should be explored further. While creep does not appear as critical for HSC as for NC, its effects should be explored specially in view of the expected increased slenderness of members of HSC.

The response of HSC to statically, monotonically applied multiaxial stresses may be similar to that of NC. For all other situations involving combined stresses, the cupboards of knowledge are equally bare for NC and HSC.

Some Suggestions for Future Research

The current deficiencies of our knowledge of the properties of HSC are clear from the foregoing. Similarly clear is our lack of knowledge about many items of interest to the user of NC. In any case, it appears appropriate to verify the applicability of current knowledge and rules about use and response of NC to HSC. Any new findings requiring changes should be in a form to cause a minimum of disruption to presently-used procedures.

Because of the ease with which uniaxial test results can be obtained in the laboratory, and formulated for the design office, it appears reasonable to emphasize uniaxial properties and behavior of HSC first.

Research should first of all consider the needs of the engineer designing with HSC. Results should be cast in a form which is meaningful to and can be used by professionals.

The following specific topics warrant study, in rough order of their practical importance:

1. The single most distinctive feature of HSC appears its brittleness; accordingly, those aspects in which energy absorption capacity is important, including cyclic and dynamic effects associated with seismic resistance, should be investigated first.

2. Because high material strength leads to slender members and structures in which deflections and instability become increasingly important, the time-dependent strains should be investigated.

3. High strength can be achieved by a variety of different methods-- increase of cement-water ratio, polymerization, various admixtures, fiber-reinforcement; with increasing mortar strength the aggregate properties become more important. For these reasons a test program to compare the properties of various high-strength concretes of equal strength achieved in different ways appears appropriate. Its goal should be to determine whether a broad-gage approach to "HSC" is appropriate, or whether this family of materials might have to be broken down into different members.

4. In view of the scant knowledge of multiaxial behavior of both NC and HSC, programs involving multiaxial testing of a range of concretes of all strengths should be initiated, including load history and time effects. Results should in all cases be formulated with a view to their use in rational analysis of concrete structures.

5. Fracture mechanics approaches to understanding of concrete behavior appear particularly appropriate to HSC because of its brittleness, and test programs should therefore focus attention on HSC.

References

1. Carasquillo, R.L., "Very High-Strength Concrete - An Annotated Bibliography," Rept. No. 367, Dept. of Struct. Engin., Cornell Univ., April 1977.

2. Hobbs, D.W., "Strength and Deformation Properties of Plain Concrete Subject to Combined Stress, Part 3," TR 42.497, Cement and Concrete Association, July 1974.

3. Hognestad, E., N.W. Hanson, D. McHenry, "Concrete Stress Distribution in Ultimate Strength Design," A.C.I., Vol. 52, No. 4, Dec. 1955.

4. Nilson, A.H., and F.O. Slate, "Structural Properties of Very High-Strength Concrete," Second Progress Report, Dept. of Struct. Engin., Cornell Univ., Jan. 1979.

5. Wischers, G., "Application and Effects of Compressive Loads on Concrete," Betontechnische Berichte, No. 2 and 3, Duesseldorf, Germany, 1978.

6. Wang, P.T., S.P. Shah, and A.E. Naaman, "Stress-Strain Curves of Normal and Lightweight Concrete in Compression," Jnl. A.C.I., Vol. 75, Nov. 1978.

7. Kupfer, H., "The Behavior of Concrete under Multi-Axial Short-Term Loading," Deutscher Ausschuss fuer Stahlbeton, Berlin, Heft 229, 1973.

8. Graf, O., "Strength and Elasticity of High-Strength Concrete," Deutscher Ausschuss fuer Stahlbeton, Berlin, Heft 113, 1954.

9. DIN 1045, Sec. 17.2.1, Beton-Kalender, Berlin, 1978.

10. Mehmel, A., and E. Kern, "Elastic and Plastic Strains of Concrete Caused by Compressive Load Cycling and Sustained Loading," Deutscher Ausschuss fuer Stahlbeton, Berlin, Heft 153, 1962.

11. Bennett, E.W., and S.E. St. J. Muir, "Some Fatigue Tests of High-Strength Concrete in Axial Compression," Mag. of Concr. Research, Vol. 19, No. 59, June 1967.

12. A.C.I. Committee 209, "Effects of Concrete Constituents, Environment, and Stress on Creep and Shrinkage of Concrete," in "Designing for Creep, Shrinkage, and Temperature," A.C.I. SP-27.

13. Meyers, B.L., "Time-Dependent Strains and Microcracking of Plain Concrete," Ph.D. Thesis, Cornell Univ., 1967.

14. Gerstle, K.H., et al., "Strength of Concrete under Multiaxial Stress States," Proc. McHenry Symp., A.C.I., SP-55, Oct. 1976.

15. Hannant, D.J., "Nomographs for the Failure of Plain Concrete Subjected to Short-Term Multiaxial Stresses," The Structural Engineer, Vol. 52, No. 5, May 1974.

16. Hobbs, D.W., C.D. Pomeroy, J.B. Newman, "Design Stresses for Concrete Structures Subject to Multiaxial Stresses," The Structural Engineer, Vol. 55, No. 4, April 1977.

17. Kotsovos, M.D., and J.B. Newman, "A Mathematical Description of the Deformational Behavior of Concrete," Mag. of Conc. Research, March 1979.

Acknowledgement

This report was written under a U.S. Senior Scientist Award of the Humboldt-Foundation, Federal Republic of Germany, for which hearty thanks are extended, as well as to the Technical University at Darmstadt, Germany, for hospitality and help.

Figures

Table 1. Current State of Knowledge

	Behavior under							
	Monotonic Loading		Load Histories		Dynamic Conditions		Time Dependency	
	Uni- Axial	Multi-	Uni- Axial	Multi-	Uni- Axial	Multi-	Uni- Axial	Multi-
Normal Conc.	M	M	M	N	L	N	M	L
High-Strength Concrete	M	L	L	N	N	N	L	N

M - Much; L - Little; N - Nothing

Table 2. Creep of Concrete of Different Strength (12)

Compressive strength at time of application of load, psi	Ultimate specific creep, 10^{-6} per psi
2000	1.40
4000	0.80
6000	0.55
8000	0.40

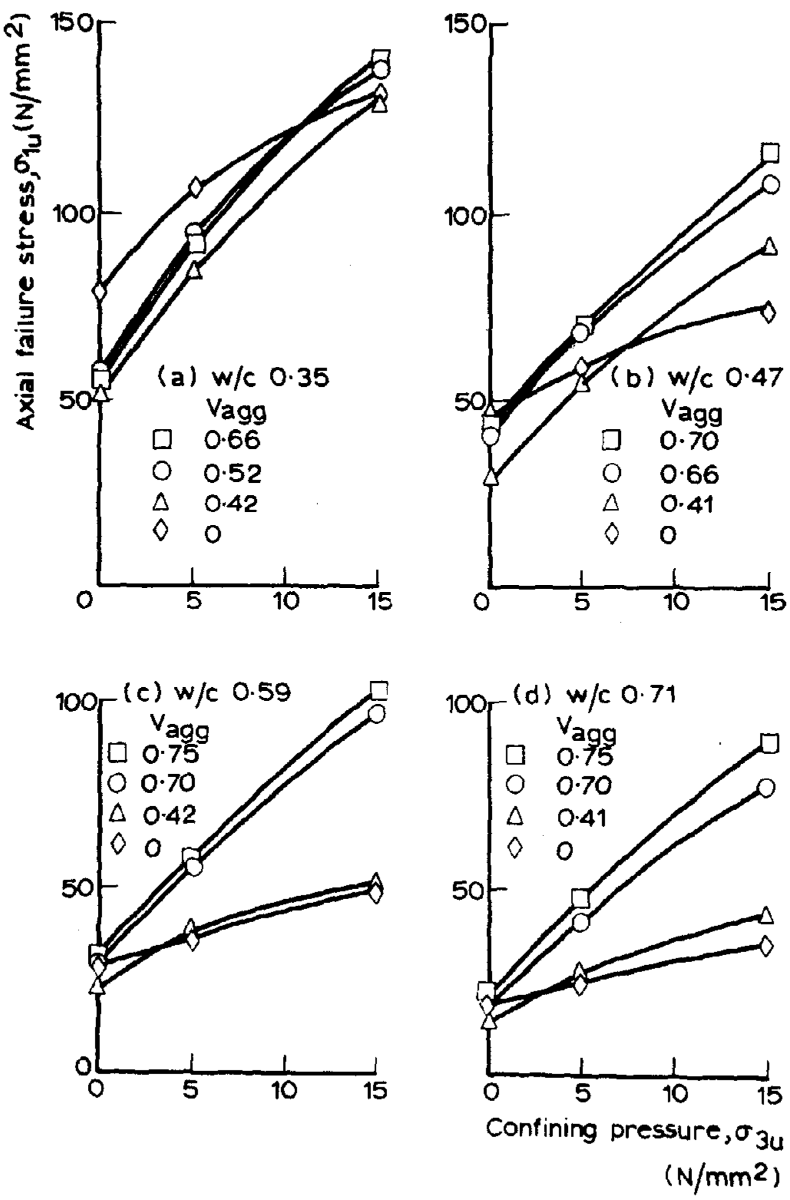


Figure 1: Influence of mix proportions on the strength of pastes and concretes in triaxial compression. (2)

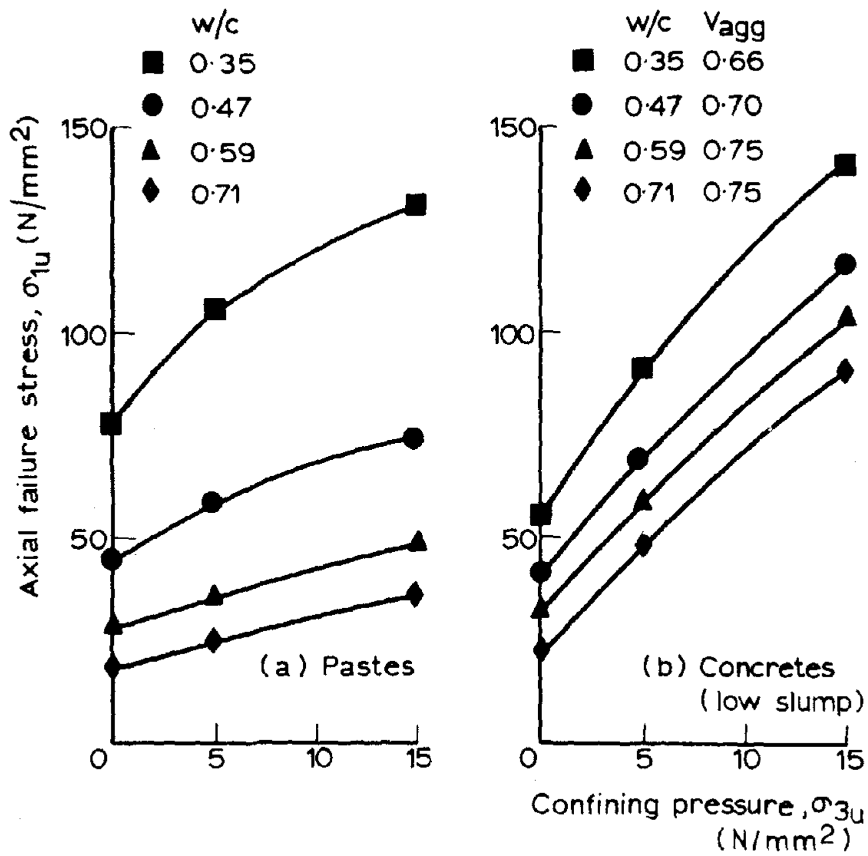


Figure 2: Influence of water/cement ration on strength in triaxial compression. (2)

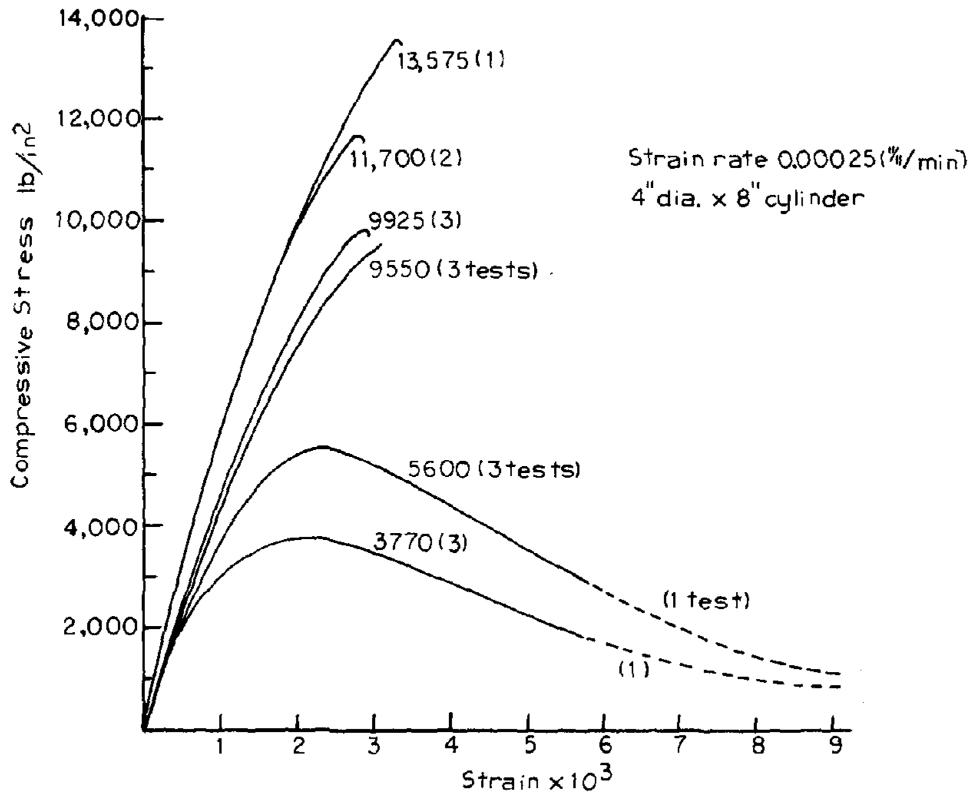


Figure 3: Compressive stress-strain curves (4)

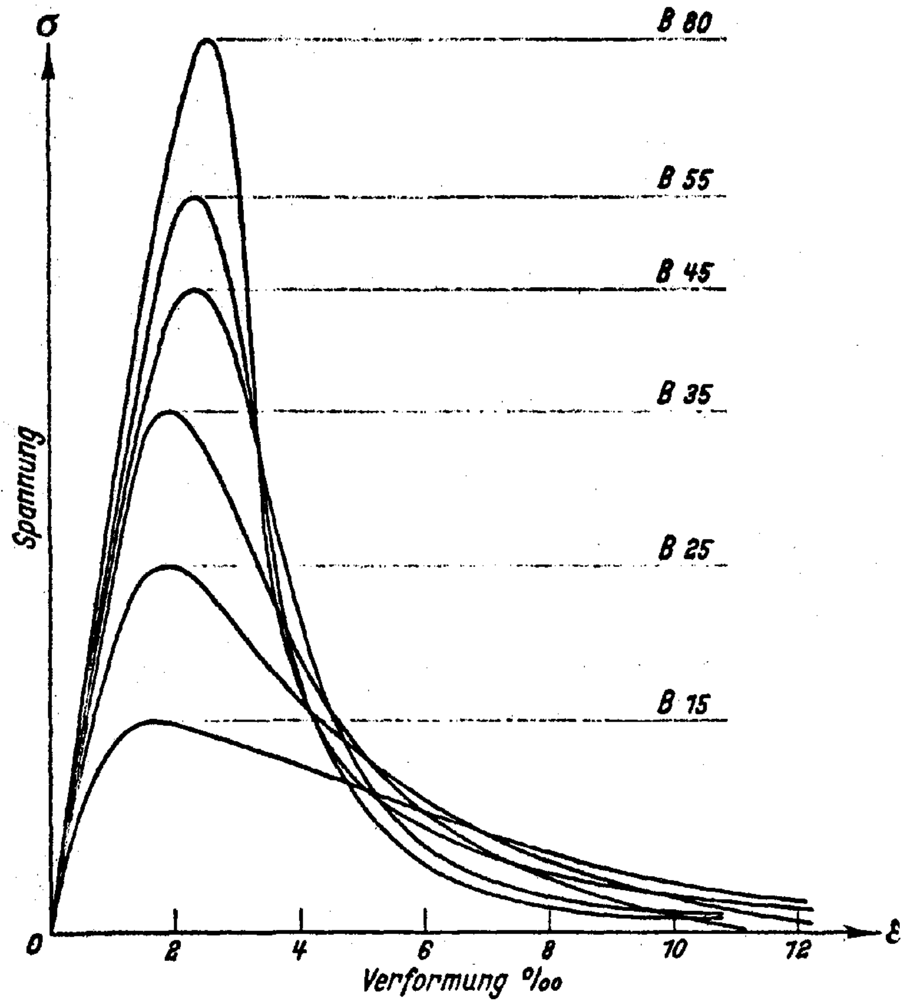


Figure 4: Complete Compressive Stess-Strain Curves (5)

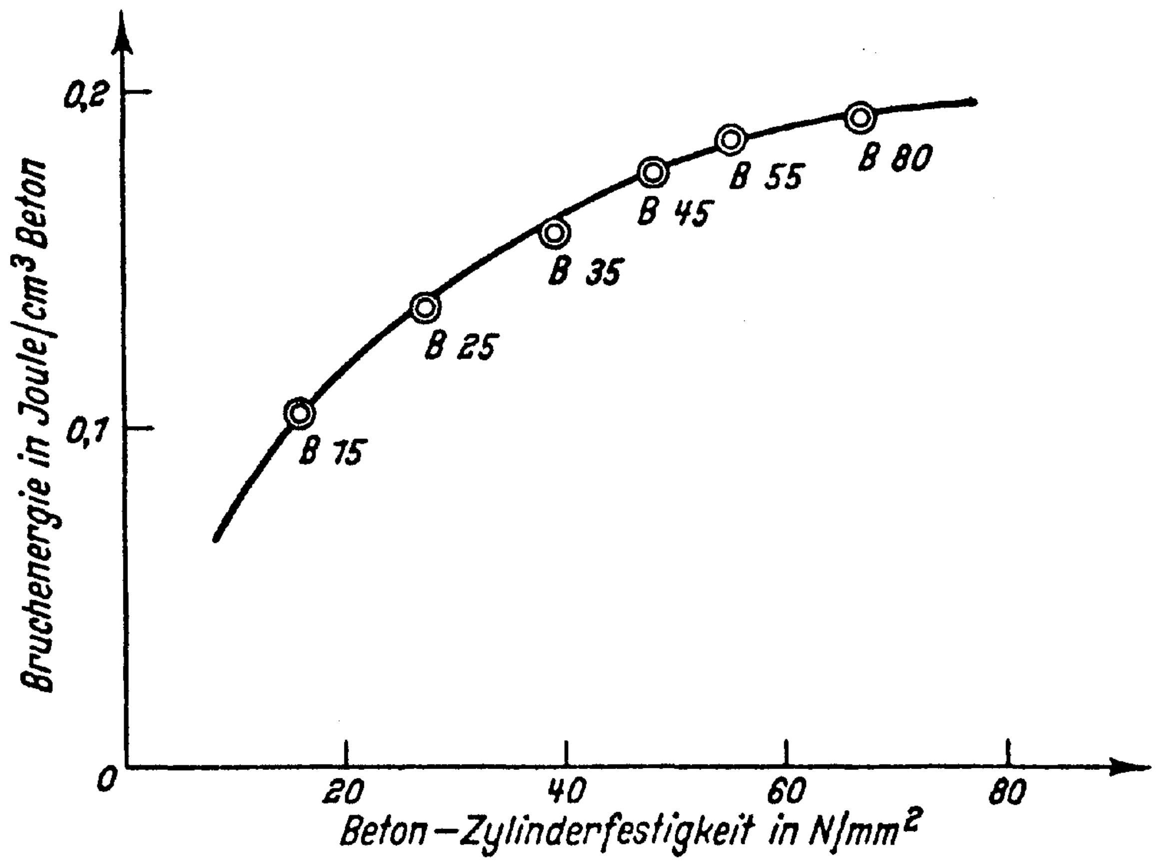


Figure 5: Toughness versus Strength (5)

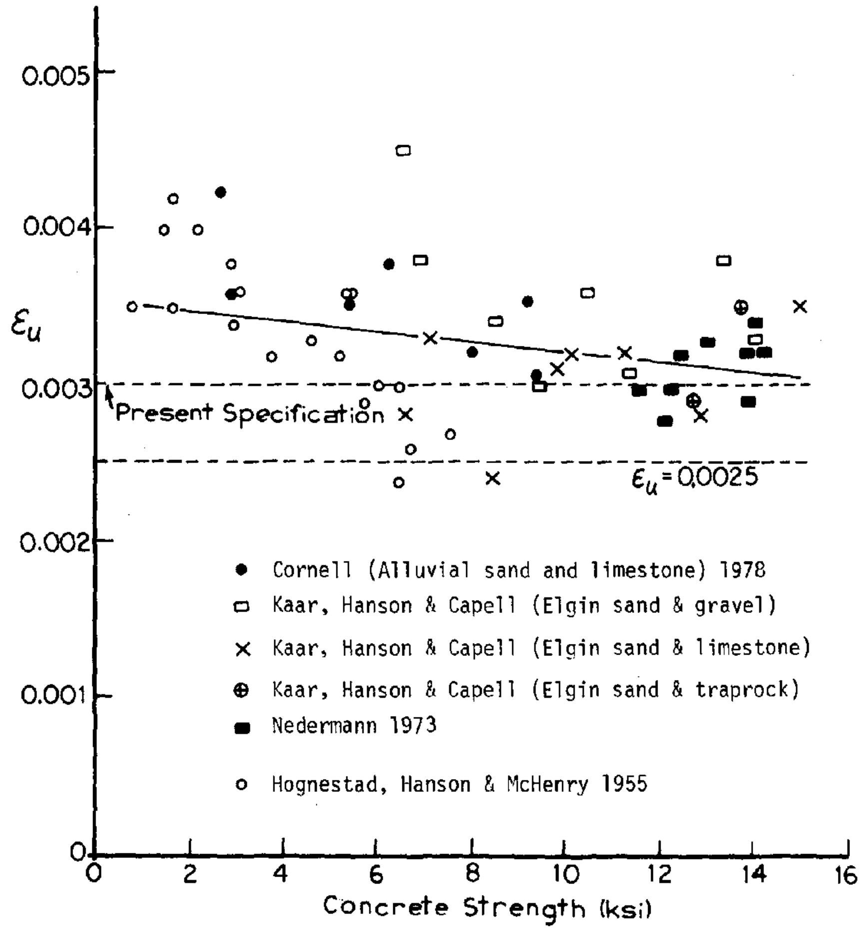


Figure 6: Ultimate Concrete Strain versus Concrete Strength (4)

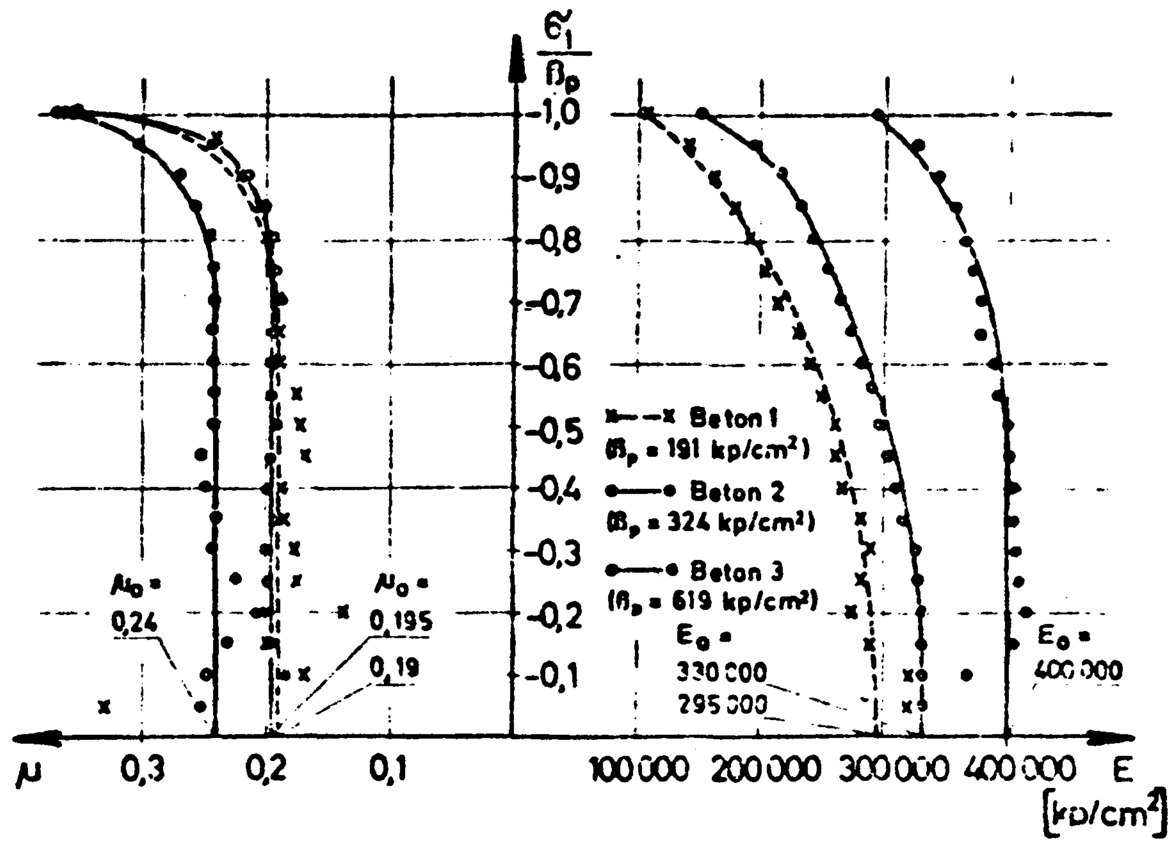


Figure 7: Modulus and Poisson's Ratio for Three Concretes (7)

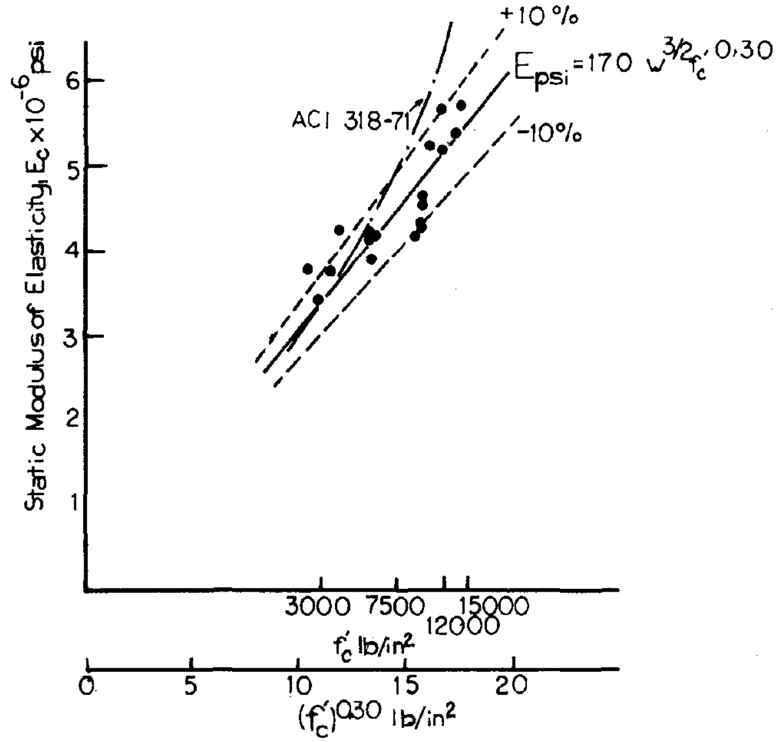


Figure 8: Static Modulus of Elasticity versus $(f'_c)^{0.30}$ (4)

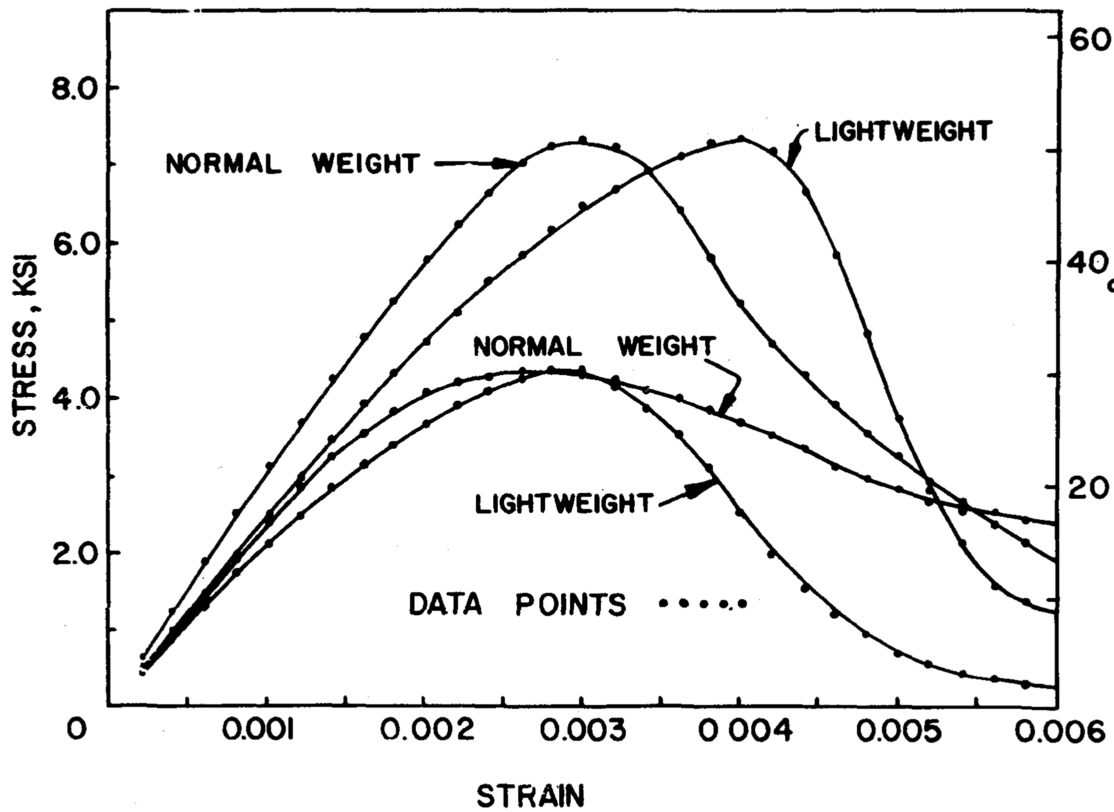


Figure 9: Comparison of normal weight and lightweight concrete stress-strain curves (6)

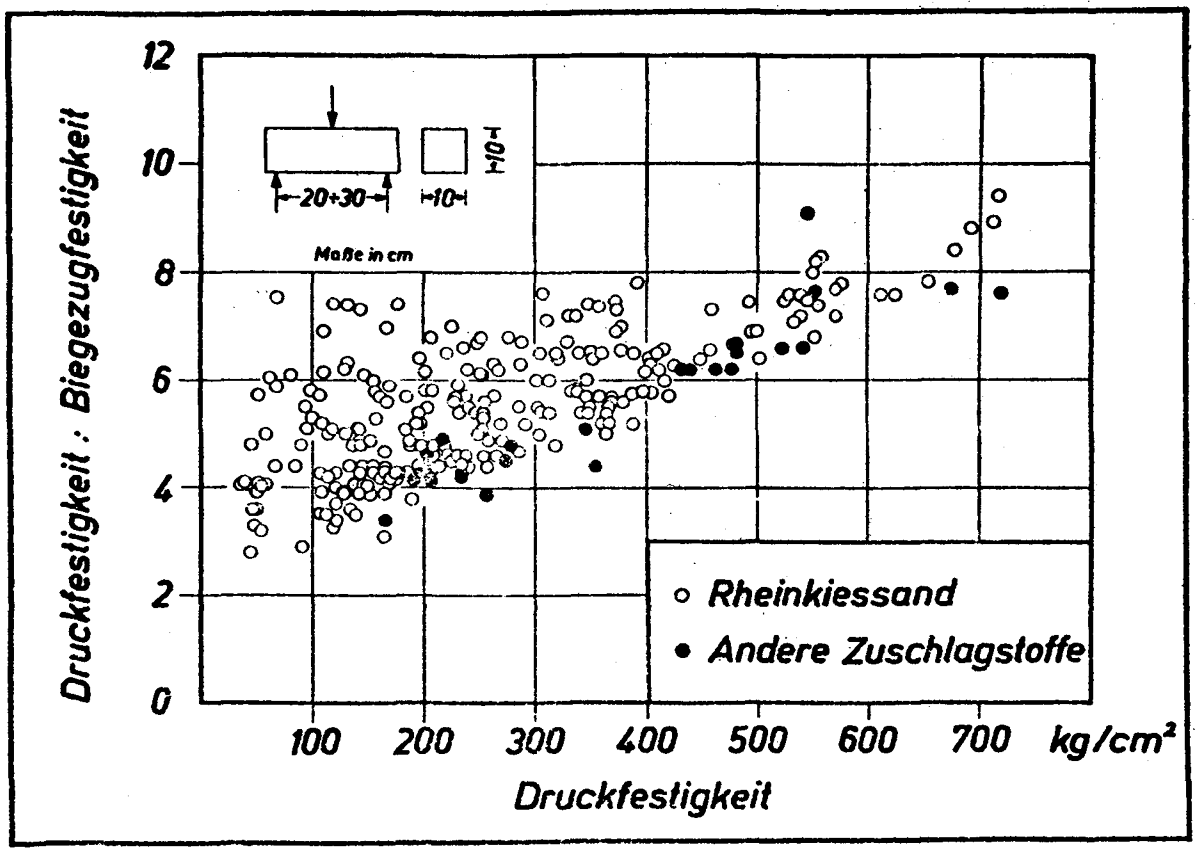


Figure 10: Ratio of Compressive Strength to Modulus of Rupture versus Compressive Strength (8)

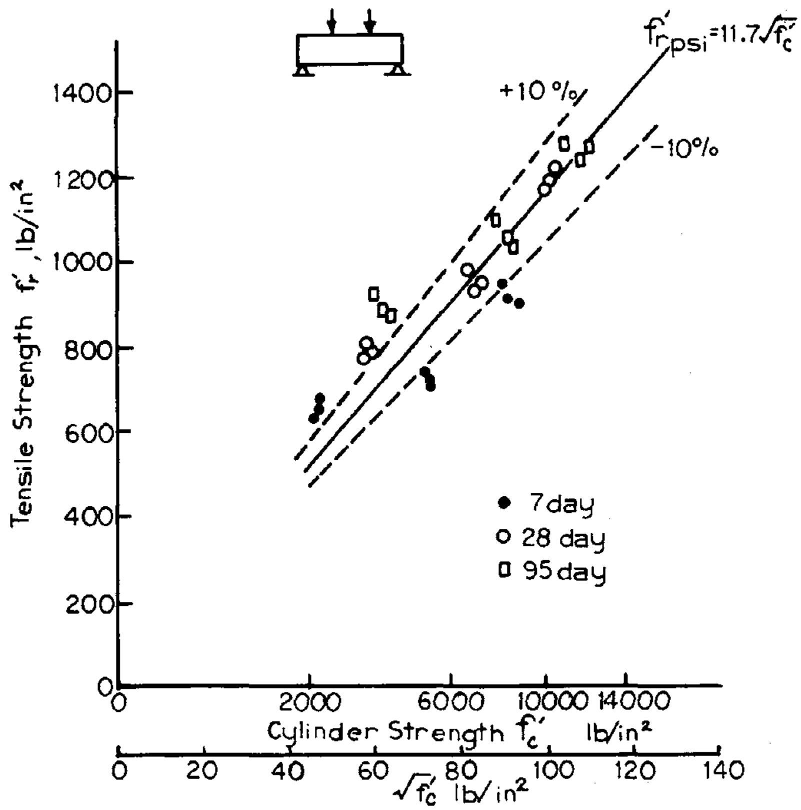


Figure 11: Tensile Strength based on Modulus of Rupture Test (4)

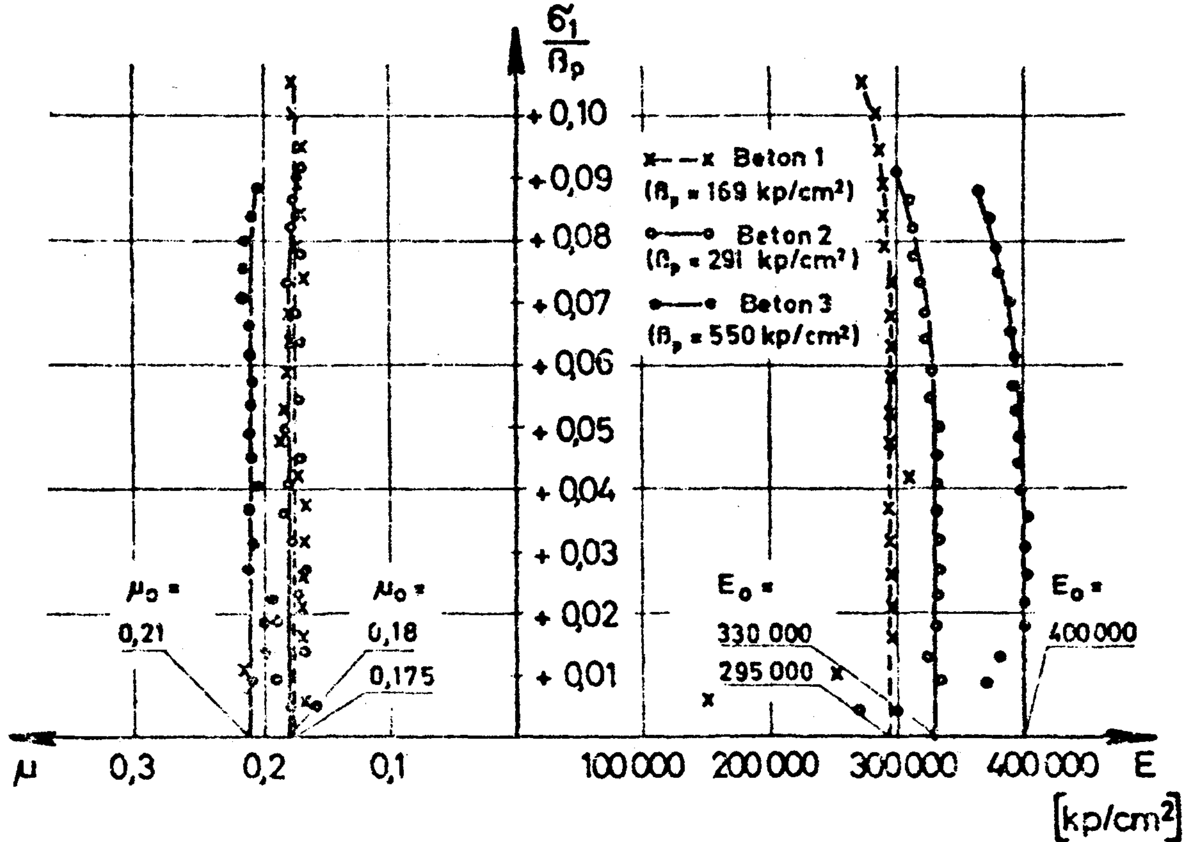


Figure 12: Tensile Properties for Three Concretes (7)

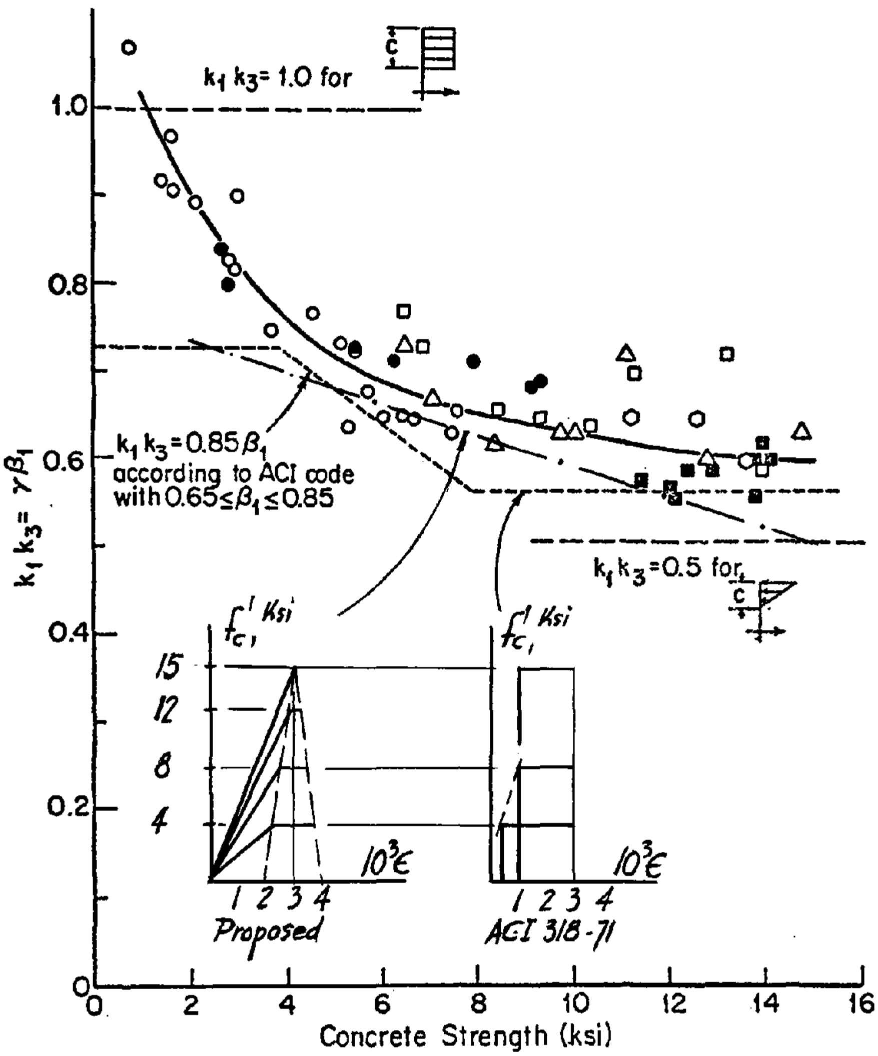


Figure 13: k_1k_3 versus concrete strength (4)

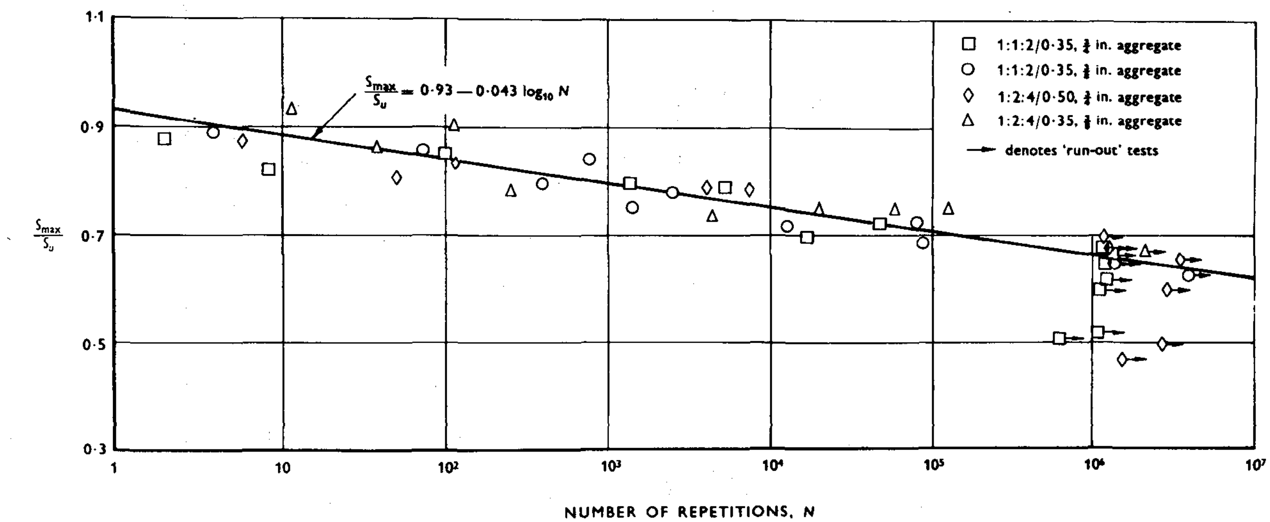


Figure 14: Fatigue Strength for Four Concretes (11)

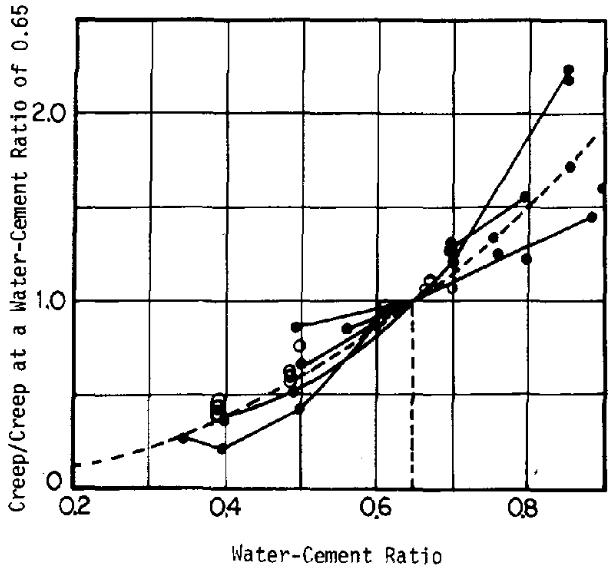


Figure 15: Effect of Water-Cement Ratio on Creep (12)

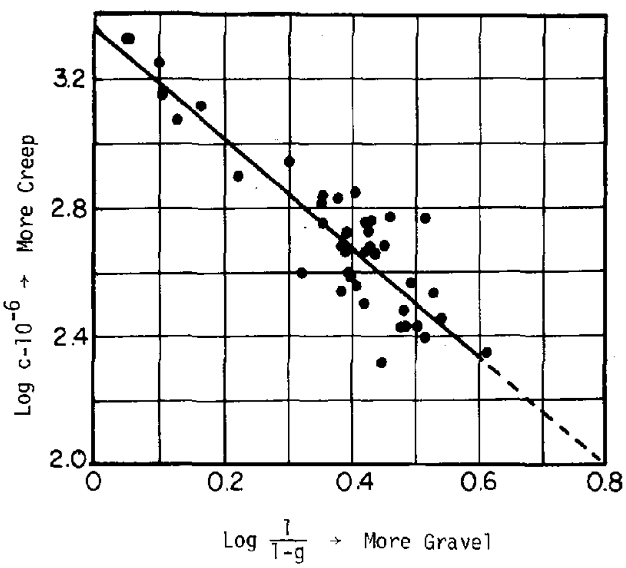


Figure 16: Effect of Aggregate Content on Creep (12)

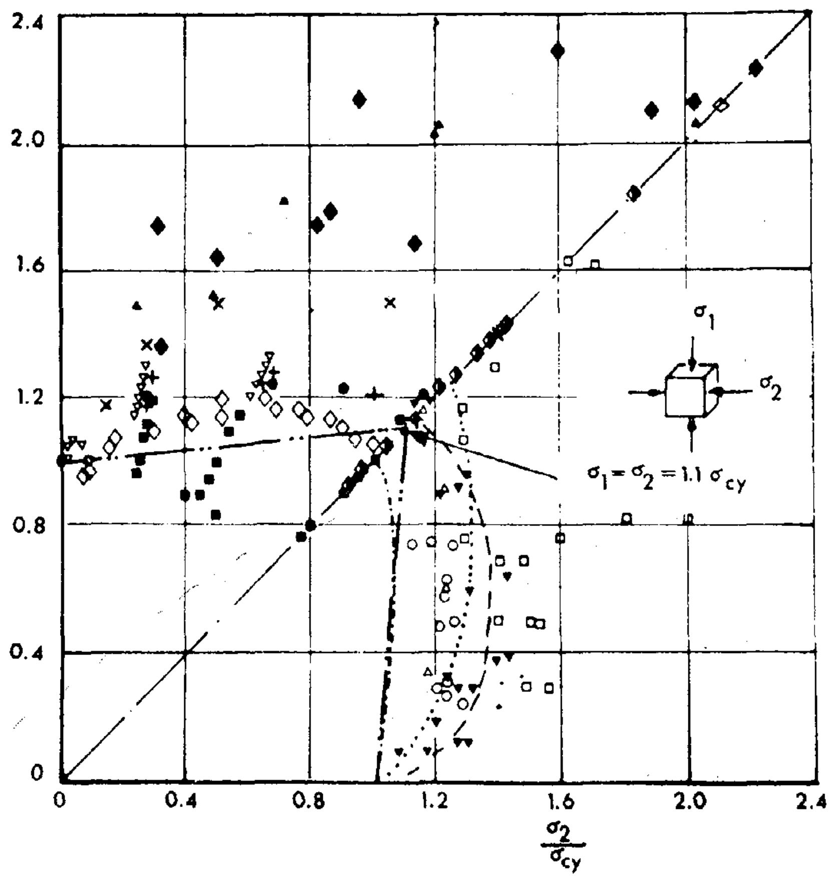


Figure 17: Failure under compression-compression-zero stress systems (15)

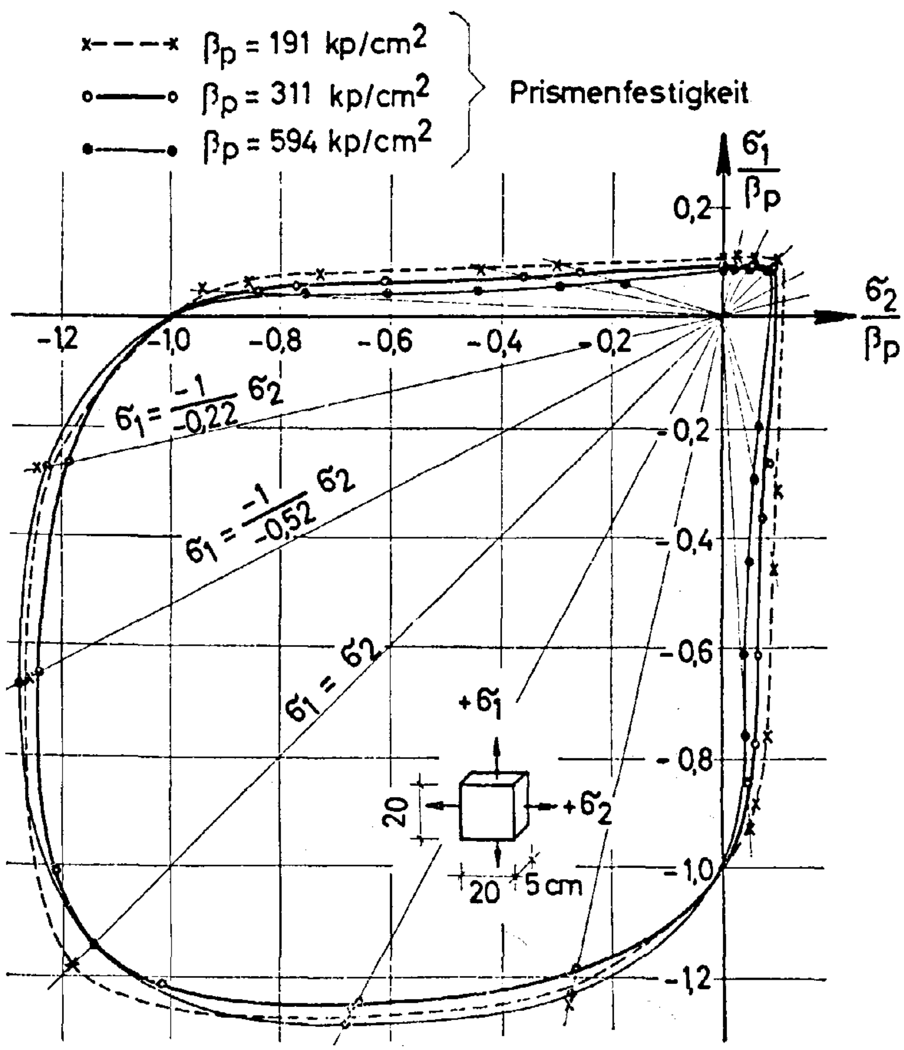


Figure 18: Biaxial Strength of Three Concretes (7)

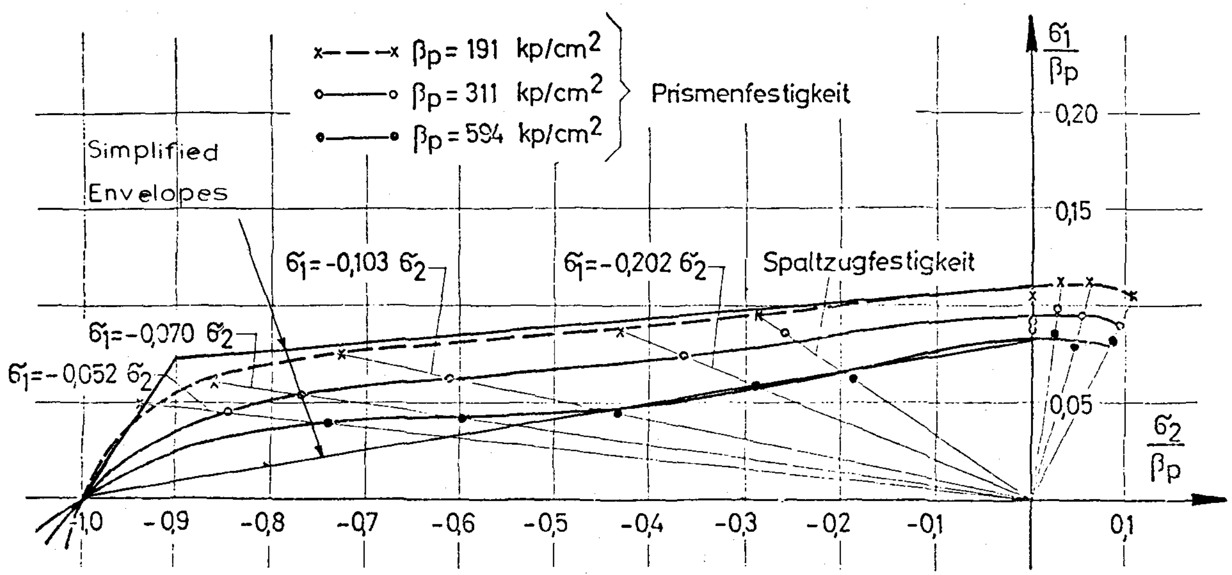


Fig. 19 Biaxial Strength in Tension-Compression Range for Three Concretes (7)

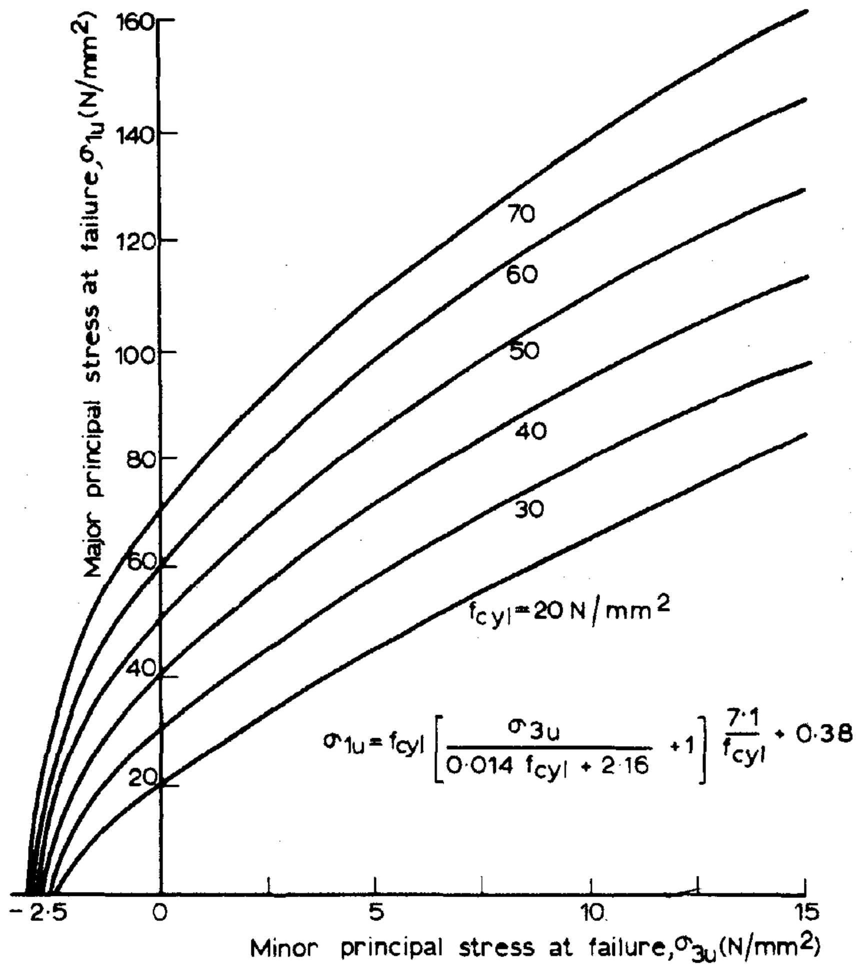


Figure 20: Failure stresses for various values of f_{cyl} (16)

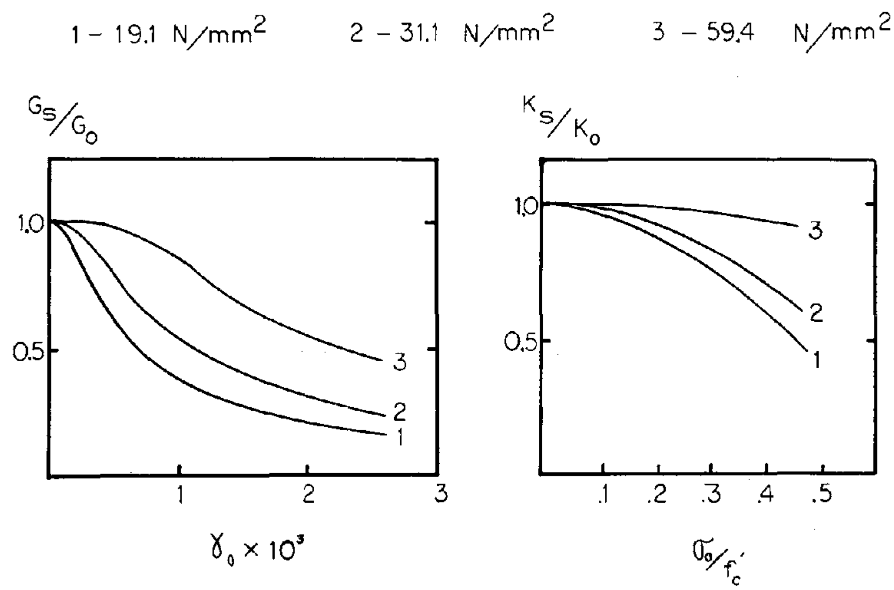


Figure 21: Secant Shear and Bulk Moduli for Three Concretes (7)

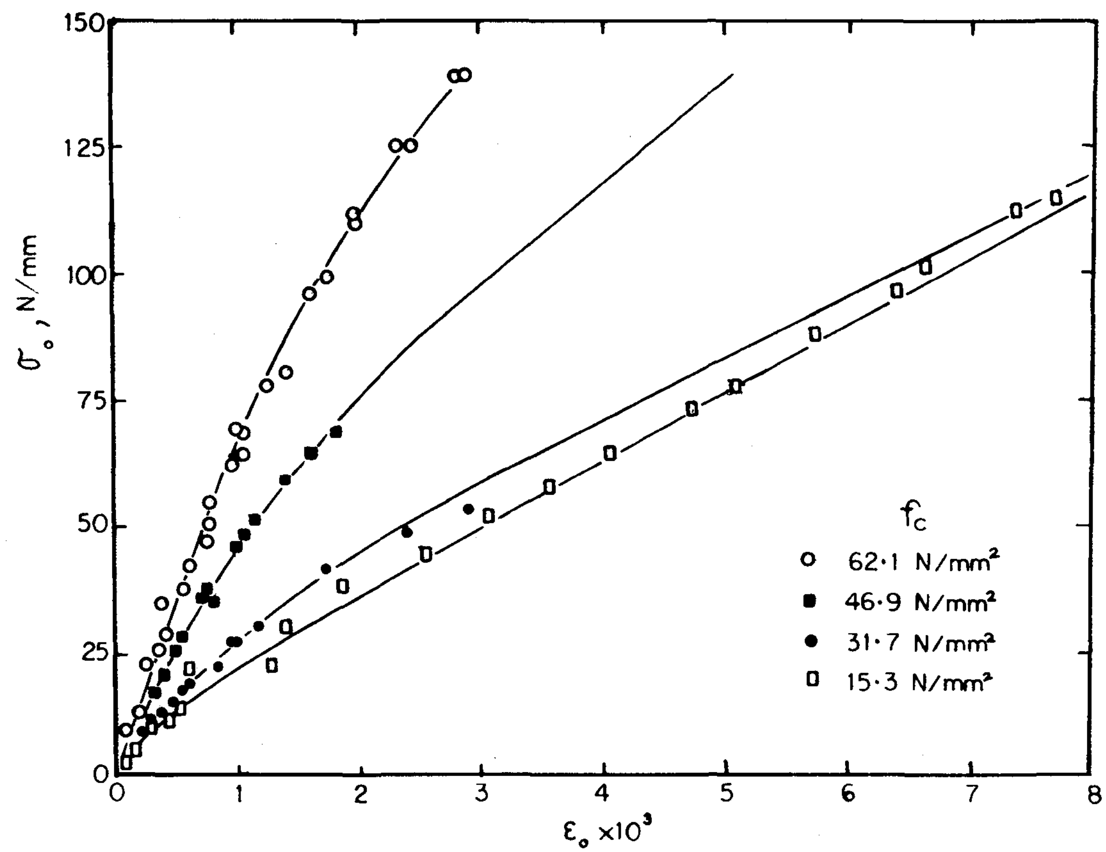


Figure 22a: Octahedral Normal Stress-Strain Relations for Four Concretes (17)

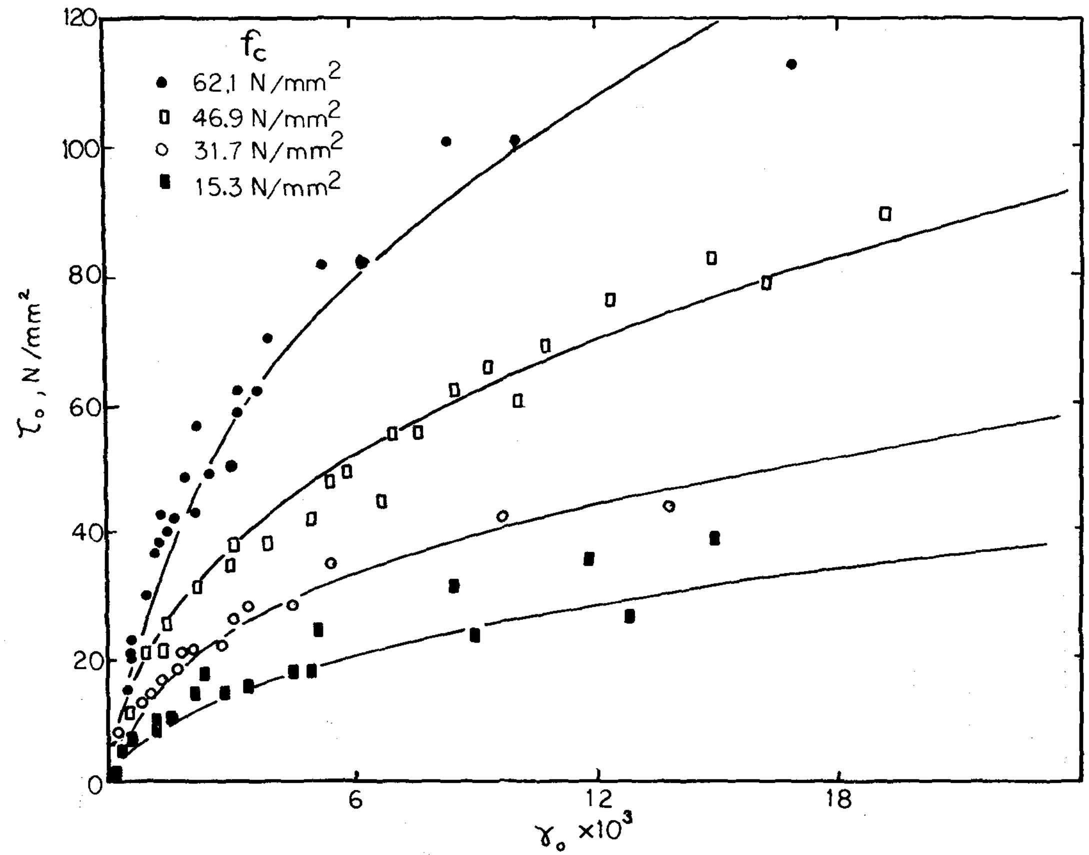


Figure 22b: Octahedral Shear Stress-Strain Relations for Four Concretes

SESSION II - DISCUSSION

MATERIAL BEHAVIOR UNDER VARIOUS TYPES OF LOADING

by

Zdeněk P. Bažant
Professor of Civil Engineering
Northwestern University
Evanston, Illinois 60201

ABSTRACT

The report on this theme presented by K. Gerstle is discussed principally from the viewpoints of continuum mechanics and micromechanics. First attention is found on the universal behavior, in which the effects of confinement on transverse reinforcement in normal and high-strength concretes are compared and the questions of ductility and failure strain are analyzed. This is followed by discussion of triaxial behavior, centering attention on differences in inelastic dilatancy, volume compaction and pressure effect on stiffness and ductility between normal and high-strength concretes. Tensile cracking is examined from the viewpoint of fracture mechanics of crack bands in heterogeneous materials and strain-localization instability, pointing out differences in fracture energy variation and the size and stiffness effects between high-strength and normal concretes. Significant differences in stress transmission across rough interlocked cracks and shear dilatancy of such cracks are also pointed out. Subsequently, the differences between creep properties of high strength concrete are considered. Finally, attention is called to some particular aspects of moisture transfer, shrinkage, drying creep and temperature effect. A study of the differences between high strength concrete and normal concretes enhances our knowledge of concrete mechanics and micromechanics in general.

INTRODUCTION

Although concretes of relatively high strength coming close to what we now classify as high strength concrete were used on a large scale already during the 1930's for military fortification belts in Czechoslovakia (Appendix I) and were also applied in Europe during the 1940's and 1950's for some early prestressed concrete bridges, the use of concretes exceeding cylindrical compression strength 60 MN/m^2 is a recent development. Successful large scale applications have been made during the last decade in spite of the lack of thorough understanding of the differences in the behavior of high strength and normal concretes.

The reporter on this subject presented an excellent and a rather complete review of the current knowledge from the viewpoint of engineering mechanics (4). We will now attempt to examine various important aspects from the viewpoint of continuum mechanics.

UNIAXIAL BEHAVIOR

For predicting limit loads and large deformation dynamic response, the shape of the stress-strain diagram, especially the post-peak behavior is of greatest importance. In discussing these effects it must be understood that the uniaxial stress-strain diagrams to be used in design of beams and frames are not unique. They are considerably influenced by the degree of confinement provided by lateral reinforcement, i.e., stirrups or ties. With heavier stirrups, the response is stiffer and, especially, more ductile in the post peak range (Fig. 1a). This is particularly marked in case of spiral reinforcement.

a) Normal Concrete b) High Strength Concrete

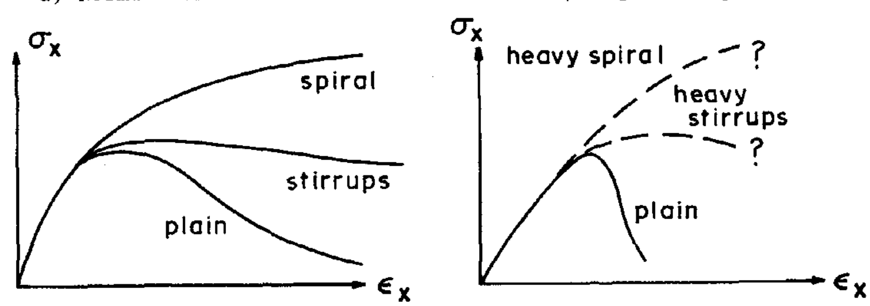


Fig. 1 Effect of Confining Reinforcement

The question now is how the effects of confinement by lateral reinforcement differ in case of high strength concrete. Test data seem to be lacking. The problem is tied to that of inelastic dilatancy due to deviatoric strains and to pressure sensitivity. Because of the more sudden appearance of microcracking, the beneficial effects of confinement are probably less pronounced in the pre-peak range. In the post-peak range, however, they are likely to be even more important than for normal concretes, because microcracking is responsible for a greater portion of inelastic behavior than plastic deformation (See Fig. 1b).

Another important parameter is the value of the strain ε_f at failure or its ratio to the strain ε_p at peak stress (ductility ratio). According to the tests on normal concretes, ε_f is not unique, and according to the concept of failure as strain-localization instability, the value of ε_f actually cannot be unique. The same must be expected for high strength concrete. It is known that for normal concretes, ε_f is much higher if the support of the specimen (testing frame) is stiffer, or if the specimen is shorter, or if the unloading slope E_u is less, or if the restraint is provided by reinforcement or by adjacent concrete as in presence of transverse stress gradient (bending). The fact that the descending slope E_t is steeper for high strength concrete decreases the strain ε_f at failure. On the other hand, the fact that for high strength concrete the unloading slope E_u shoots closer to the origin (Fig. 2b)(because a greater portion of inelastic strain is due to microcracking rather than plastic slip) causes the failure strain to increase.

From this viewpoint it also appears that ε_f must be a function of the cross section size, beam slenderness and type of reinforcement. This last fact was demonstrated for normal as well as high strength concretes by Wang, Shah and Naaman (1).

As for the effect of confinement, we may expect that to achieve the same relative effect on the ductility ratio, the lateral reinforcement (stirrups) must be heavier for high strength concrete than for normal concrete. A study of the effect of confinement by ties and spirals is of utmost importance because columns of high-rise buildings are the major application of high-strength concrete. These questions, intimately related to inelastic dilatancy and pressure sensitivity, are considered later.

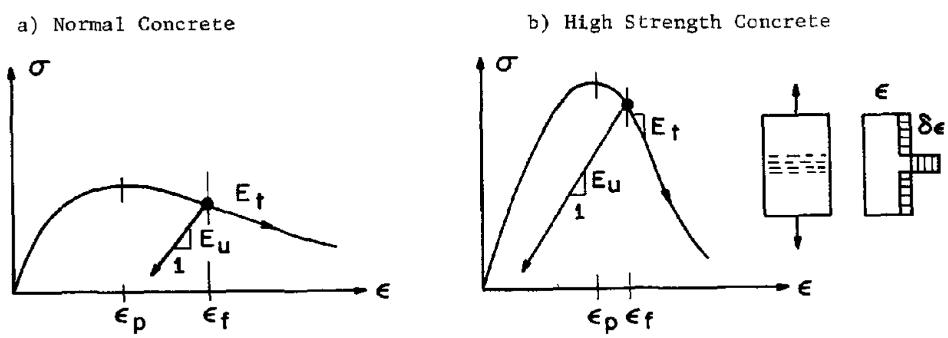


Fig. 2 Failure Strain ε_f in Relation to Peak Strain ε_p, and Bifurcation Giving Rise to Unstable Strain Localizations

With regard to the ultimate bending moment calculation, the only necessary information on the compression stress block in beams and columns is the magnitude and the location of the compression resultant. The shape of the stress block is irrelevant as long as it is chosen to give these two values correctly. Thus, for a rectangular cross section it makes no sense to introduce complicated stress distributions and it is sufficient to make the simplest choice, i.e., the Whitney's rectangular stress block (as concluded by Wang, Shah and Naaman, Ref. 1).

Arguments regarding the shape of the compression stress block are, however, of interest for finding simple rules for the magnitude and the location of the compression resultant in cross sections of arbitrary shape. The rectangular compression block cannot serve this purpose well for both rectangular and non-rectangular compression regions of the cross sections of beams and columns. Moreover, the stress distribution makes a difference, of course, in calculating the strains at failure and determining the condition of balanced reinforcement.

TRIAXIAL BEHAVIOR

In the elastic range, the situation is simple -- it suffices to know the Poisson ratio; it is higher than for normal concretes, which is a consequence of a tighter microstructure. For the inelastic behavior, as explained by the reporter, microcracking plays a larger role in high-strength concrete. Plastic deformation, on the other hand, plays a lesser role, and the confining pressures at which the high-strength concrete becomes essentially plastic are no doubt distinctly higher (although tests are lacking at present).

Based on this observation, the applicability of the constitutive relations in the form of incremental plasticity, which give a reasonable albeit limited description of normal concretes, would be more limited in case of high-strength concrete. We may expect that it would be even more beneficial than for normal concretes to enhance a plasticity model by the fracturing stress decrements (relaxations), which are derived from loading surfaces in the strain (rather than stress) space and are related to degradation of the elastic moduli. On the other hand, the inelastic phenomena due to microcracking may be also well described by the endochronic theory, and therefore, this theory may be also expected to be effective for high-strength concretes.

Another popular incremental approach to triaxial modeling of concrete has been the so-called "orthotropic" (variable moduli, hypoelastic) models. We will not consider these models, however, because they do not satisfy the basic invariance requirements of continuum mechanics. Namely, if we calculate the response of an "orthotropic" model to a nonproportional stress path in one coordinate system and also in a rotated coordinate system, we get different (in fact very different) states of strain. Moreover, the facts that in the orthotropic models the principal stresses and principal strains are always parallel and that shear strain increments cause no volume changes, are at variance with experimental reality.

A very simple and practically useful approach is the total strain theory (also called the deformation theory), in which, due to isotropy, the material is fully characterized by two nonlinear relations, one between the tangential components of octahedral stress and strain, and another between the normal components. This may be alternatively described as a variation of the secant bulk modulus K and secant shear modulus G, which was considered by the reporter.

It must be kept in mind, however, that this theory can apply only to proportional or near-proportional loading. Moreover, due to the restriction that the secant stiffness matrix must be of isotropic form, the theory cannot model coupling between shear strains and normal stresses, such as shear compaction and, especially, the inelastic dilatancy due to shear, which represents a salient feature of the inelastic behavior of concrete and is a manifestation of the opening of microcracks. When the ratio of shear stress magnitude (stress intensity) to hydrostatic pressure is large, the inelastic dilatancy becomes very large (Fig. 3a), so large that the apparent bulk modulus K would be negative (volume expansion in the presence of hydrostatic pressure). The available total strain theories do not give negative K, and so they *cannot* model this phenomenon; this means that they may be applied only to initial inelastic phenomena but not to those at large shear strain. (It is, however, possible to develop a total strain theory which gives negative K).

a) Normal Concrete

b) High Strength Concrete

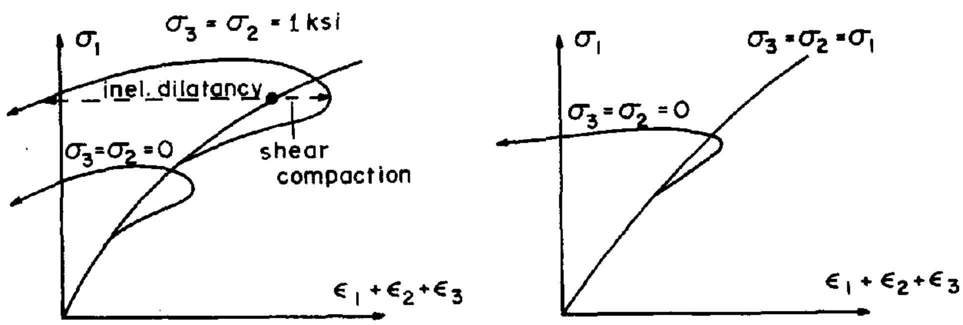


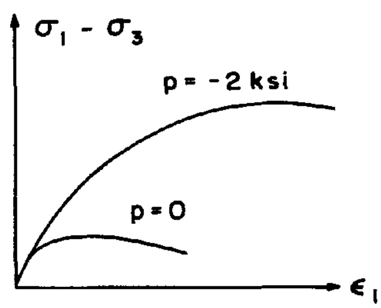
Fig. 3 Inelastic Dilatancy due to Shear and Shear Compaction

For high strength concretes, the inelastic dilatancy may be expected to exhibit a more abrupt development (Fig. 3b), because of the greater brittleness and the more sudden development of microcracks. Moreover, because of greater cohesion within the microstructure, a larger shear strain magnitude is probably needed to produce the same dilatancy ratio, compared to normal concretes.

The shear compaction (Fig. 3a,b) which precedes the dilatancy and represents an effect of lesser magnitude, is likely to be less pronounced in case of high strength concrete. This is because the microstructure is more compact to begin with, leaving less room for further compaction, such as that due to closing of pores.

Related to dilatancy, the effect of hydrostatic pressure on ductility and plasticity of response is of great importance, especially for achieving safer failure behavior of the columns in high-rise buildings. Pertinent triaxial tests seem to be again lacking, but from the known effect of strength within the normal strength range it may be inferred that a considerably higher hydrostatic pressure is needed to achieve the same ductility of response and to eliminate the dilatancy associated with strain - softening behavior. This is evidenced, e.g., by comparing the standard triaxial tests carried out at the Bureau of Reclamation (Balmer, 1949) on low strength concrete and those carried out at Tera-Tek (Green, Swanson, 1973) on higher strength concretes; cf. Ref. 3 or 5. In the latter tests, the same increase of hydrostatic pressure causes a much lesser stiffening of response curves. E.g., for $f_c' = 3570$ psi (Balmer), hydrostatic pressure $p = -1000$ psi increases the peak axial stress (superimposed) about 4-times compared to uniaxial loading ($p = 0$), whereas for $f_c' = 7020$ psi (Green and Swanson), the increase is only about twice (cf. Refs. 3 and 5); see Fig. 4. But the same ratio p/f_c' seems to give about the same relative increase of stiffness and of peak stress.

a) Normal Concrete



b) High Strength Concrete

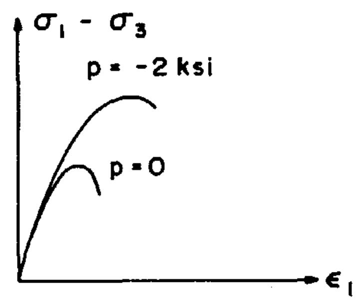


Fig. 4 Effect of Confining Hydrostatic Pressure

With regard to tensile cracking, an important effect, indicated by the reporter, is the reduction of tensile strength by the simultaneous transverse compressive stress. This effect appears to be much stronger in high strength concrete than in normal concretes. E.g., while at $\sigma_2 = f_c'/2$ the tensile strength of normal concrete is about $0.8\ f_c'$, the tensile strength of high strength concrete seems to be only about $0.4\ f_c'$, i.e. 50% less. This effect might have to do with higher Poisson ratio of high strength concrete, causing that the transverse compressive stress produces a larger axial extension and a greater weakening of bonds in the axial direction. A proper explanation would have to involve fracture mechanics, in particular the facts that the microcracked zone at the crack front is smaller for high strength concretes and that the microcracks are more aligned in the transverse direction. In normal concrete where the cracks cannot cross the aggregate they tend to be more randomly oriented, and so the transverse compression tends to close the non-aligned microcracks and thus stabilize the fracture. This stabilizing effect is weak in high strength concretes. Moreover, the microcracked zone at crack front, undergoing tensile strain softening (descending segment) exhibits in high strength concretes a sharper volume dilatancy due to transverse compressive stress; this promotes fracture, as has already been indicated.

FRACTURE

Development of fracture mechanics of concrete is complicated by its heterogeneity, which causes that at the front of a crack there is a large microcracked zone. As a consequence, the linear fracture mechanics cannot be applied except for structures whose cross sections are at least 100-times larger than the size D_m of the microcracked zone. Typically perhaps D_m = 5 maximum aggregate sizes, which means that for aggregate of maximum size 1 inch, the cross section would have to measure at least 500 inches (2.5 m) for the linear fracture mechanics to apply perfectly. Most structures as well as test specimens are well below this size limit. This causes that the values of stress intensity factor K_I (or critical energy release rate G_{cr}) are not unique, and considerably different values are obtained from different experiements.

These difficulties do not mean, however, that we could determine propagation of cracks or crack bands on the basis of failure criteria in terms of stresses (i.e., strength). Such criteria are unobjective and give greatly different results depending on the method of calculation, e.g., the size of the finite elements. The problem we face is one that lies between the range of fracture mechanics and the fine-scale limit of the applicability of a continuum model.

In view of the fact that there is no sharp crack front but a rather diffuse front consisting of a band of microcracks, it appears to be more appropriate to treat fracture propagation in terms of crack bands of finite width that is a material property (6). Fracture extension may then be viewed as unstable localization of strain into the crack band. Global energy treatment of such instability (analogous to boundary layer methods in fluid mechanics) becomes asymptotically equivalent to linear fracture mechanics as the width of the band approaches zero (6). For the actual finite width of the band the energy release rate G_{cr} associated with fracture extension Δa is not a constant but rather appears to be a function of the band width w_c and of the stiffness of the rest of the structure relative to the opening δ at the crack band front (Fig. 5). Note that the smaller the stiffness C, the larger is the stored energy in the structure as well as the rate at which this energy unloads. Evidently, G_{cr} decreases as w_c decreases, and as C increases. The value of G_{cr} may be related to the area under the tensile stress-strain diagram (Fig. 6). For small w_c and small C, and if there is an abrupt transition from elastic to inelastic behavior, G_{cr} is approximately equal to the total area under the curve (Fig. 6a). For a large w_c and a large C, the value of G_{cr} can be much less, as shown in Fig. 6b.

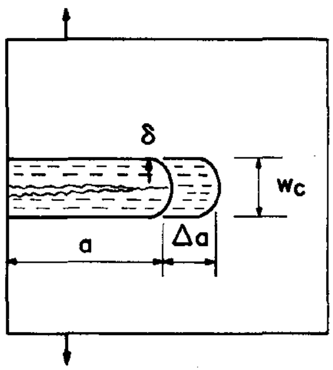


Fig. 5 Crack Band Propagation

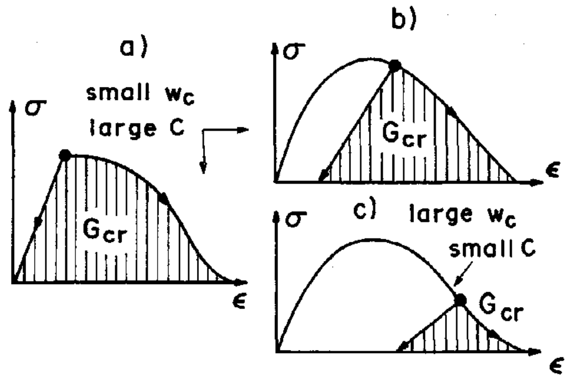


Fig. 6 Fracture Energy of Crack Band and Strain Localization

The differences between normal and high strength concretes may now be analyzed from the viewpoint just explained. Because of the greater stiffness of the structure and the steeper descending portion of the stress-strain diagram, G_{cr} in high strength concrete should be closer to the total area under the curve, especially for medium w_c and medium C (Fig. 7b and 7). Thus, we may infer that the range of applicability of linear fracture mechanics should be broader for high-strength concretes, i.e., the size of the structure does not have to be as large.

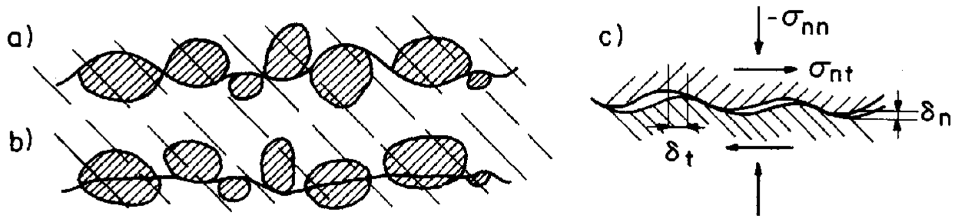


Fig. 7 Rough Crack in (a) Normal Concrete, (b) High Strength Concrete, and (c) Stress Transmission

We can arrive at the same conclusion from another point of view. Due to the high strength of mortar between large pieces of aggregate, the microcracking will be less extensive, i.e., the microcracked zone will be smaller for high strength concretes, the crack band will be narrower. Accordingly, the structure size, for which linear fracture mechanics applies is smaller.

A salient property of cracks in concrete which is of great practical importance is the capability of transfer of shear as well as normal stresses across the cracks. This property results from interlock of opposite surfaces due to their roughness. Obviously, the greater the roughness, the stronger is the stress transfer capability. As the reporter points out, cracks in high strength concrete do not always tend to follow the aggregate-mortar interface, but often pass across the pieces of aggregate. This is due to the fact that the strength of mortar differs less from that of aggregate, i.e., the microstructure is less heterogeneous (or more homogeneous) in terms of its elastic properties.

Therefore, the cracks in high strength concrete are likely to be less rough (and it is a general experience that in more homogeneous materials the crack surfaces are smoother); Fig. 7b. Thus, the capability of transfer of shear and normal stresses across the cracks and the effective friction coefficient must be expected to be smaller for high strength concretes. This may have some adverse effect on structural performance near the ultimate load.

On the other hand, however, smoother crack surfaces will lead to a smaller dilatancy of cracks due to relative tangential displacement of their surfaces. This would cause, for example, that the reinforcement crossing the cracks would receive a lesser extension (lesser tensile force) due to relative slip on the cracks.

CREEP AND TIME DEPENDENCE

Although the general trends expounded by the reporter are correct, the detailed effect of various factors on creep is more complicated. The effect of composition parameters on creep of concretes of various strength was carefully studied in Ref. 7. In that work, 80 different published data sets, involving different concretes of various strengths, from different laboratories, but not including high strength concretes, were analyzed with the help of a computer, and formulas giving the creep parameters were established. Creep was represented as a sum of the basic creep (i.e., creep at no moisture exchange) and the drying creep which is similar to shrinkage. Composition and strength were found to have considerably different effects on various parameters.

As the reporter states, it is true that the lower the water-cement ratio, the lower is creep. Furthermore, the creep coefficient, i.e., the ratio of the long-time deformation to the short-time deformation, normally also decreases as the water-cement ratio decreases. This seems logical if we regard creep as the result of migrations of solid particles from loaded weakly bonded areas in cement gel to load-free areas. For a lower water-cement ratio the porosity of the hardened cement paste (as well as cement gel) is less and so the microstructure is more extensively bonded, which means that there are fewer potential sites for particle migrations due to load.

The direct effect of strength f_c', other than that associated with water-cement ratio, is more subtle. Higher f_c' means a higher exponent n of the double power law* for basic creep (7). So does a higher aggregate-to-cement ratio, a/c, but an increase in f_c' is usually accompanied by a lower a/c; thus, the total effect of strength increase on n may go either way. As for w/c, a lower w/c means a lower n.

The effect of f_c' on the rate of aging, i.e., decrease of creep with the age at loading t', is according to Ref. 7 quite simple: the exponent m in function t'^{-m} giving the age effect is less for higher f_c', i.e., the aging effect is more pronounced for concretes of higher strength. This is certainly not illogical if we note that, in order to achieve a more tightly bonded microstructure, the hydration reactions must advance more intensely and perhaps also for a longer time period.

The nonlinear components of creep also depend on the strength. The creep increase beyond proportionality (flow), observed at high stress, will no doubt exhibit a sharper change, similar to time-independent stress-strain diagram, i.e., the increase will be less at medium stress but suddenly become very large near the long-time strength value (Fig. 8). This is because the nonlinearity of the stress-strain diagram is largely a manifestation of rapid nonlinear creep(viscoplasticity). Whether the decrease of the long-time strength with the period of loading (Δ in Fig. 8) is less pronounced for higher strengths is not known.

a) Normal Concrete

b) High Strength Concrete

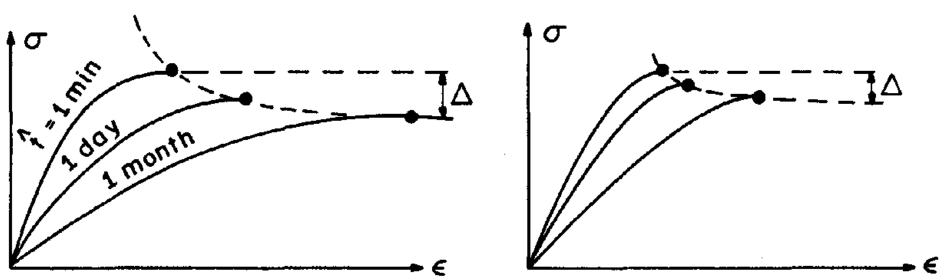


Fig. 8 Creep Isochrones and Long-Time Strength

Another significant nonlinearity of creep, which occurs at low stress (service stress range), is the adaptation. It is due to accelerated bonding or hydration under compression, and causes that after a longer period of low sustained compressive stress the creep (as well as the instantaneous deformation) due to the subsequent superimposed

---

*$J(t, t') = \frac{1}{E_0} + \frac{\varphi_1}{E_0}(t'^{-m} + \alpha)(t - t')^n$ = strain at time t caused by a unit stress acting since time t'; E_0, φ_1, n, m, α = material parameters.

stress is considerable less than this stress alone would produce in virgin concrete of that age. Although measurements are unavailable, the adaptation is likely to be more intense in high-strength concrete because it essentially constitutes accelerated bonding or hydration, and the hydration (or aging) effect on creep is stronger for higher strengths(7), as already remarked.

The acceleration of creep due to a cyclic load superimposed on a static load can be modeled as a relatively simple extension of the double power law (7). So it seems that the strength effect in cyclic creep would be similar to that in static creep.

The dependence of short-time deformations on the loading rate, usually regarded as the strain-rate effect, is nothing but a consequence of creep, the rapid initial creep. Since this creep is described by the double power law, the same law that applies to long-time creep, quite well, our previous observations on the strength effect in creep probably also apply to the rapid initial creep, i.e., to the strain-rate effect. But these results are limited to the low (service stress) range. For higher stresses, one can perhaps extrapolate the preceeding observations on long-time nonlinear creep into the short-time range.

Since the relative creep in high-strength concrete is less, the strain-rate effect on the initial elastic modulus as well as the strength may be expected to be weaker. This agrees with observations.

Of great interest is the strain-rate effect on the descending (strain softening) portion of the stress-strain diagram. It was recently observed by W. Dilger that at high strain rates characteristic of blast loading the descending slope for normal concretes becomes much less. The same no doubt occurs for high strength concrete, and a reasonable conjecture is that this effect is even stronger since the descending slope for normal static tests is larger.

The triaxial properties of nonlinear creep and the strain-rate effect are insufficiently known even for normal concrete.

MOISTURE TRANSFER, SHRINKAGE, DRYING CREEP AND TEMPERATURE

The permeability of concrete is a property which can vary by six orders of magnitude. Permeability is related, albeit only quite weakly, to porosity, and porosity is essentially determined by the water-cement ratio. A high strength is achieved by a low water-cement ratio, and according to Ref. 7 this leads to a lower diffusivity as well as permeability. So, high strength concretes are distinguished by low permeability, as well as diffusivity. Therefore, high strength concretes dry and shrink slower. Moreover, it is found in Ref. 7 that a higher strength and a lower water-cement ratio both lead to a smaller final shrinkage strain. A complicating factor is, however, the aggregate-cement ratio, the decrease of which (often characteristic of high strength concrete) increases the final shrinkage strain.

The increase of creep due to drying, or drying creep, may approximately be treated as shrinkage. Therefore, the foregoing observations on the effect of strength on shrinkage may be readily extended to the drying creep.

The temperature effect on basic creep is known to be twofold: (1) heating reduces the apparent viscosities, which increases creep; (2) and it also intensifies hydration, i.e., aging, which decreases creep. Competition of these two influences makes the temperature effect rather complicated. Influence 1 is an activation energy effect. This phenomenon is characteristic of molecular structure, and it cannot be affected by strength. Influence 2 is another activation energy effect, which is itself again unaffected by strength; however, since this effect controls the rate of hydration or aging, and because the effect of age at loading upon creep is larger in a stronger concrete (7), as already mentioned, influence 2 is likely to be greater in a concrete of higher strength. This would mean that in a not too old concrete, the overall relative increase of creep in high strength concrete would be less than it is in normal concretes. But tests are needed.

The two competing influences are also present in permeability (or diffusivity), in shrinkage and in drying creep, and a similar conclusion may be drawn.

Nothing appears to be experimentally known on the differences of high strength concrete in moisture transfer and pore pressure under high temperature response (over 100°C).

CONCLUSION

Compared to normal concretes, the experimental information as well as the theoretical understanding of the mechanics of high strength concrete is much more limited at present. Apart from the obviously advantageous property of high uniaxial compression strength, accompanied by side advantages in lower creep shrinkage and drying rates, there exist some disadvantageous properties, essentially various manifestations of increased brittleness. Although experimentation and simple direct interpretation of observed behavior will be indispensable for further development, it is clear that continuum mechanics, including fracture mechanics, will be needed to gain adequate understanding. Moreover, a study of the differences between high strength and normal concretes will improve our knowledge of concrete mechanics and micromechanics in general.

REFERENCES

1. Wang, P.T., Shah, S.P.,Naaman, A.E., "High-Strength Concrete in Ultimate Strength Design," J. of the Structural Div., Proc. Am. Soc. of Civil Engrs., Vol. 104, Nov. 1978, pp. 1761-1773.

2. Wang, P.T., Shah, S.P., Naaman, A.E., "Stress-Strain Curves of Normal and Lightweight Concrete in Compression," J. of the Am. Concrete Institute, Vol. 75, Nov. 1978, pp. 603-611.

3. Bažant, Z.P., Kim, S.-S., "Plastic-Fracturing Theory for Concrete," J. of the Engineering Mechanics Div., Proc. Am. Soc. of Civil Engrs. Vol. 105, June 1979, pp. 407-428.

4. Gerstle, K. "Material Behavior under Various Types of Loading," Preprint of Report on Theme II, NSF Workshop on High Strength Concrete, held at University of Illinois at Chicago Circle, Dec. 1979.

5. Bažant, Z.P., "Endochronic Theory for Inelasticity and Failure of Concrete, J. of the Engng. Mech. Div., Proc. ASCE, Vol. 102, August 1976, pp. 701-721 ; for refinements see Nuclear Engng. and Design, Vol. 47, 1978, pp. 305-315, and ASCE Meeting Preprint No. 3611, Atlanta, October 1978.

6. Bažant, Z.P., "Fracture Mechanics of Crack Bands in Heterogeneous Materials," work in preparation.

7. Bažant, Z.P., Panula, L. "Practical Prediction of Time Dependent Deformation of Concrete," Materials and Structures, Vol.11, 1978, Part I and II, pp. 307-328, Part III and IV, pp. 415-434, Vol. 12, 1979, Parts V and VI, pp. 169-183.

8. Bažant, Z.J.,"L'emploi du béton armé pour fortifications et abris contre avions"(Use of Reinforced Concrete for Fortifications and Air Raid Shelters), Travaux-Science et Industrie (Paris), April-May 1937.

APPENDIX I. 60 MPa CONCRETE IN PRE-WORLD WAR II FORTRESSES

In the numerous reinforced concrete fortresses built in Czechoslovakia during 1937-38 (see Ref. 8), the design called for minimum cubic compressive strength of 60 MPa. This strength was achieved by the use of:

1) Gap-graded aggregates—Elbe River sand up to 7 mm size and crushed Litice gravel. (On the site the aggregates were stored on plank floors to prevent their contamination by local soil.)

2) Water-cement ratio 0.44, low for those times; this required tamping of 15 cm thick layers by pneumatic hammers and pneumatic vibrators attached to the shuttering (all of which was prepared before concreting even for two-story fortresses). Each layer was placed not more than one hour after the lower layer was tamped.

3) Special, low-heat cement for walls more than 60 cm thick; this prevented cracks due to volume changes.

The design strength was always achieved easily and was usually substantially exceeded. Some test cubes even attained strength 100 MPa. The high strength resulted mainly from the fact that, due to gap-grading, the large gravel pieces were bearing one upon the other throughout the whole thickness of the wall. It was noticed that the compression strength was especially high when the crushed gravel pieces were carefully placed so that no sharp points protruded from the layers in the direction of compression; however, in that case, the strength in the transverse direction was no doubt less. The gap-grading also substantially reduced shrinkage and creep.

Today, after 41 years, there is no appreciable deterioration of concrete in these fortresses, in spite of the low water-tightness due to gap-grading.

SESSION II - SUMMARY OF FLOOR DISCUSSION

MATERIAL BEHAVIOR UNDER VARIOUS TYPES OF LOADING

by

W. F. Chen
Professor of Structural Engineering
School of Civil Engineering
Purdue University
West Lafayette, Indiana 47907

GENERAL

The behavior of concrete materials under various types of loading conditions has been the subject of research for many years. Despite this, the problem is still not fully understood even for regular concrete, and design methods for reinforced concrete structure are, for the most part, based mainly upon accumulated experience, as laid down in empirical formulae and rules of thumb. However, the recent advances in the following three areas have accentuated the need for a more rational design basis:

1. The development of high strength concrete with possible reinforcing systems from reinforcing bars, fibers, polymers, and other additives has resulted in a drastic modification of concrete properties from either a material with high strength and little ductility to one with a somewhat lower strength and large ductility. Thus, we have reached a stage at which potential material properties can be tailored to particular structural service requirements.

2. The adoption of the limit state approach to design has focussed particular attention on two requirements: accurate information regarding the behavior of structures throughout the entire range of loading up to ultimate load, and simple procedures to enable designers to assess this behavior.

3. The introduction of non-classical constructions such as the offshore field and reactor vessels has rendered this empirical approach unviable.

CONSTITUTIVE MODELS

A rational theory means that the same fundamental principles are used for the analysis of different structures - beams, walls, slabs, shells - subjected to different loadings - bending, torsion, shear, punching. The classical theory of continuum mechanics such as elasticity and plasticity constitutes such a general framework. The discussers of this Session have agreed that the following three levels of scale of stress-strain relationships of concrete material are of equal importance.

1. Micromechanics level of formulation in terms of aggregate size, cement paste, pore size distribution etc. - this is the subject of Session I.

2. Continuum mechanics level of formulation in terms of stress and strain tensors - this is the subject of the present Session.

3. Structural level of formulation in terms of generalized stresses such as moment and axial force, and generalized strains such as curvature and axial elongations - this is the subject of Session III.

The basic information required in the present continuum mechanics formulation is the material behavior of a small element under various types of loading conditions. Once the stress-strain relation of the composite material is available, generalized stress-strain relationships for structural elements or subassemblages may be developed. This has generally been achieved with success in the flexural analysis of beams and frames where the state of stress is essentially one-dimensional, but it only has limited success in slabs. Although the continuum mechanics approach has been used in the analysis and design of off-shore structures, concrete dams, reactor vessels, and tunneling problems, it has not widely accepted as a general design tool for concrete structure. Professor Pister emphasizes the need for such an extension from continuum level to structural level. Hopefully, this approach will eventually yield useful results which, if and when they are incorporated into the national codes, will lead to sabstantial savings of materials, in addition to the fact that it will put the design of reinforced concrete structures on a more rational basis.

MECHANICAL PROPERTIES

The key to this progress is hinged on the availability of sufficient experimental data on material behavior under various types of loading conditions. The present state of knowledge of the mechanical behavior of high-strength concrete is summarized by Professor Gerstle. He found that, with the exception of its response to monotonically - applied uniaxial loadings, little is known. The current state of knowledge is summarized in the following Table

	Behavior Under							
	Monotonic Loading		Load Histories		Dynamic Conditions		Time Dependency	
	Uni- Axial	Multi- Axial	Uni- Axial	Multi- Axial	Uni- Axial	Multi- Axial	Uni- Axial	Multi- Axial
Normal Concrete	M	M	M	N	L	N	M	L
High Strength Concrete	M	L	L	N	N	N	L	N

M - Much; L - Little; N - Nothing

To supply additional data on time-dependent behavior of concrete, Professor D. J. Parrott presented a set of supplementary data on creep and shrinkage under uniaxial stress condition. Further, Mr. Russell reported a set of creep and shrinkage data measured from actual full size reinforced concrete structures. Simplified expressions were also described.

Additional information on the behavior of confined concrete was reported by Professor S. P. Shah, and results of tests on tied columns (12" x 12" square) along with proposed stress vs. strain curve for confined concrete was given by Professor S. M. Uzumeri. A proper confinement by lateral reinforcements is seen possible and efficient in producing large ductility for structural needs.

The stress-strain plot depends on the definition of the so-called "stress" and "strain". Professor A. Hillerborg discussed the inter-relationship between the descending branch of an uni-axial tensile stress-elongation curve and the corresponding tensile stress-strain curve and proposed a method for plotting this data properly. Professor J. Dougill emphasized the importance of the effects of size, and the unavoidable strain localization phenomenon in the concrete, and pointed out the difficulties in relating test results to actual situation.

DEFINITIONS OF HIGH-STRENGTH CONCRETE

High-strength is a relative matter. In general, concrete of cylinder strength above 8 ksi is considered HSC, but the strength of concrete has a continuous variation and cannot be defined in specific numbers. This Recorder proposed that the proper definition lies not in the specific values, but rather in whether or not the design or construction are influenced by the quality of "High-Strength" and require special considerations in design and construction when compared with the design of ordinary concrete. To a structural engineer, for example, if he can use the present ACI Building Code for Design without taking special measures to insure ductility, then, as far as he is concerned the concrete is a normal strength concrete, although the strength may be as high as 10 ksi. Hence, depending entirely on the user's viewpoint judging from the significance of the impact of high strength on his environment, structural engineer may define it as a normal strength concrete, while the material engineer may call it a high-strength concrete.

SUMMARY

The starting point of a more rational analysis and design of reinforced concrete structures in general, and high-strength concrete in particular, is a constitutive model for the concrete. This model must be sufficiently elaborate to give a reasonable description of concrete stressed into the ultimate range, and sufficient simple to permit an easy experimental identification of the material parameters. Based on the description of continuum mechanics and experimental stress-strain data, generalized stress-strain relationships for reinforced concrete structural elements can be developed from which structural behavior of entire reinforced concrete systems may be analyzed. At present, an encouraging start has been made, but much more remains to be done.

SESSION III - REPORT

INELASTIC BEHAVIOR OF STRUCTURAL ELEMENTS AND STRUCTURES

by

Vitelmo V. Bertero
Professor of Civil Engineering
University of California, Berkeley, California, 94720

ABSTRACT

To design and construct sound structures, it is necessary to predict their inelastic behavior under different types of environmental excitations. For structures subjected only to normal types of excitations, the prediction of inelastic behavior is needed to estimate the structure's safety against collapse. However, for structures that can also be subjected to severe abnormal excitations, the visualization of inelastic behavior is necessary even to design the structure. Inelastic response of a structure is sensitive to the type and history of the excitation, thus it is necessary to predict such response under all possible excitation histories. This paper reviews the present state-of-the-art in making such predictions for normal-strength concrete. Next, the advantages and disadvantages of using high-strength concrete in high rise buildings are discussed. This discussion is of a speculative nature because of the lack of available data about structures and elements built with high-strength concrete. Although the inelastic behavior of all structural elements is reviewed, the discussion emphasizes problems in predicting and attaining good hysteretic behavior in columns and shear walls of tall buildings, because it is in the construction of these elements that the application of high-strength concrete appears to be advantageous. Finally, recommendations are made for future research and development.

INTRODUCTION

General Remarks

A question continuously raised in literature, conferences, and personal discussions is: Why do we need to predict the nonlinear (inelastic) behavior of R/C structures? Sometimes the question is formulated in more specific terms such as: Why inelastic concrete analysis? Why inelastic concrete design? Why nonlinear analysis and design? The first question actually covers all the others.

In the past, many academicians, researchers, and professionals have discussed these questions (1-7). Today, it is generally accepted that the ideal design is that which results in minimum total cost, including possible losses, for all limit states. This comprehensive design philosophy, discussed by Sawyer (8), has however, not yet been totally applied in actual design. No practical design method has yet been developed that simultaneously satisfies all the requirements imposed by the different limit states. In practice, the most critical limit state is used as the basis for proportioning members in the preliminary design; all other main limit states should then be checked through a comprehensive analysis. The advantages of developing a design method based on two failure stages have been discussed by Sawyer, and a design method has been developed based on two behavior criteria (collapse and loss of serviceability) and four optimizing criteria (9). Application of this method to the seismic resistant design of ductile moment-resisting frames seems feasible and practical (10).

The writer believes that structures should be designed according to the limit state which controls their design, and therefore finds it convenient to classify the design in two broad groups, according to the type of excitations they will be subjected to. The first group includes structures that will be only exposed to normal type of excitations; the second group includes structures that may also be subjected to severe abnormal loading such as earthquake ground motions, blasts, hurricanes, etc.

For structures subjected to normal loading, the limit state controlling design is serviceability and the preliminary design can usually be based on linear elastic analysis of the structure. However, even in this case it is necessary to predict the inelastic behavior of the structure, both to determine the realistic safety factor against collapse in case of possible overloads, and to have a more economical design. This is presently recognized in building codes. ACI 318-77 (11), for example, allows moment redistribution in continuous flexural members (Sections 8.4 and 18.10.4) as well as in the design of slab (Chapter 13). However, this recognition comes through empirical equations whose general applicability and soundness has been continuously questioned. In order to properly design concrete structures that would be subjected to only normal loadings, taking advantages of their possible inelastic behavior, it is necessary to predict the load-deformation relationship under monotonically increasing loads and/or deformation beyond the service load up to collapse. The prediction of strength at collapse seems quite accurate for planar moment-resisting frames as well as for planar wall-frame structural systems (12). However, the prediction of the deformation capacity at collapse still presents problems because there is insufficient data to predict the deformation capacities of the critical regions of different structural members (12).

For structures subjected to abnormal loading, it is sometimes necessary to predict not only the load-deformation relationship up to collapse under monotonically increasing loading, but also the hysteretic behavior under different types of cyclic loading. This is a considerably more complex problem. Although some authors claim that by knowing the load-deformation relationship under monotonically increasing loading it is possible to predict the hysteretic behavior, it has been shown that in most cases this is not so. Under cyclic loading, particularly loading requiring deformation reversals, the mechanisms of shear resistance and deformation in the flexural critical regions are quite different from the same mechanisms under monotonically increasing loads. Similarly, the mechanisms of bond resistance and deformation are also quite different under cyclic loading than under monotonically increasing loads.

Advances have been made in understanding hysteretic behavior of members of moment-resisting frame and wall-frame structural systems in R/C structures which are subjected to severe seismic excitations and in which normal strength concrete is used. (See Refs. 12-15). Although no general analytical method has been developed that can predict the three-dimensional hysteretic behavior of actual R/C buildings under all possible seismic ground motions, significant advances have been made in understanding the mechanisms that affect the planar behavior of such buildings. The question then arises, what happens when it is necessary to use high-strength concrete?

Objectives and Scope

The main objectives of this report follow. 1) Discuss the advantages and disadvantages of using high-strength concrete in the construction of R/C buildings subjected to either normal loadings or to both normal and severe abnormal loadings, especially severe seismic ground motions. 2) Review the data available on the inelastic behavior of reinforced concrete material, structural elements, and whole buildings. 3) Analyze the potential problems that the use of high-strength concrete can create in predicting inelastic behavior of structures, and therefore in structural performance, especially under severe seismic motion. 4) Give possible solutions to these problems. 5) Formulate recommendations for research and development.

To achieve these objectives, a definition of high-strength concrete is given, followed by a review of pertinent literature on the use of high-strength concrete in structures and members and the mechanical characteristics of these structures. Special emphasis has been placed on reviewing data about the inelastic behavior of the material itself and of simple structural elements made of such material. Because very little data is available, the author has analyzed in a speculative way, what might be the problems created by the use of high-strength concrete. This analysis is based on present knowledge of the behavior of R/C structural elements made with normal-strength concrete and the problems that have been observed using higher and higher strength concrete. Some solutions to potential problems are offered, based on the solutions that have been found to be effective for similar problems that have been encountered using normal-strength concrete. The paper ends with recommendations for research and development, to assure the proper and beneficial use of high-strength concrete in buildings, particularly buildings which may be subjected to severe seismic ground motions demanding significant hysteretic behavior.

HIGH-STRENGTH CONCRETE

Definition of High-Strength Concrete

There is, at present, no unique definition of what constitutes high-strength concrete. It is hoped that this workshop will make some recommendations regarding the ranges of strength for normal and high-strength concrete. Perhaps it will be necessary to define some other grade of strength, such as intermediate, very high, etc.

In this report, the following definition, put forward by the task force report (16) is adopted:

"High-strength concrete is considered to be concrete with compressive strengths higher than 6000 psi."

However, it is necessary to distinguish between the use of normal and lightweight aggregate. While the above definition applies to normal aggregate concrete, in the case of lightweight aggregate, high-strength concrete is considered to be concrete with compressive strengths higher than 4000 psi.

The above definitions are relative and have been influenced by some present code provisions. For example, it has been claimed that the present ACI Building Code Method (11) of strength design is based on beam tests with concrete strength in the range of 3000 psi to 6000 psi and that the code rectangular stress block does not predict beam behavior with f_c' above 8000 psi (55 MN/m^2) (17). Furthermore, present UBC seismic code provisions (18) and ATC tentative seismic provisions (19) require that the specified 28-day compressive, f_c', shall not exceed 4000 psi for concrete with lightweight aggregate, when this concrete is used in structural components of special moment frames and walls proportioned to resist seismic forces. This limitation has been imposed because of the lack of data on the behavior of such components under load reversals into the nonlinear range of response (19).

Specimens in which the concrete was made specifically for certain engineering uses have developed compressive strengths as high as 18,000 psi. There have also been concrete pastes which have developed compressive strengths of 40,000 psi (20). These facts make it clear that a new category of very high-strength concrete, having a compressive strength above 15,000 psi, could be defined.

Available Data

There have been numerous articles on the general subject of high-strength concrete (21). However, there is very little data available on the mechanical characteristics of high-strength concrete and most of them are from tests conducted on plain unconfined concrete. The data available will be discussed in more detail later on in this report.

Advantages of Using High-Strength Concrete

As pointed out by Hollister (22) and the Chicago Committee on High-Rise Buildings (16), there are several factors which place increasing pressure on designers and builders to use high-strength concrete. Perhaps the most important of these factors is the economic one.

The following discussion of reasons to use this type of concrete is divided into two parts. First, there is a summary of the situation of buildings located where they will be exposed to what can be considered normal loading. Next, there is a discussion of buildings in which the probability of being subjected to severe abnormal loadings (particularly due to seismic ground motions) during their service life is significant enough to control their design.

Concrete buildings whose design is controlled by normal loadings -- The need for tall buildings has been thoroughly discussed in the monograph on the Planning and Design of Tall Buildings (23). For concrete buildings of ordinary low strength concrete, the potential number of stories is limited by the large columns and shear walls required. The number of stories can be increased by using high-strength concrete in the design and construction of these columns and shear walls. Concrete with compressive strengths of 9000 psi has already been used economically in several buildings, the tallest one having 79 stories (16).

The most economical columns and shear walls are the ones with the smallest cross-sectional area and the minimum percentage of steel (16). Thus, the use of high-strength concrete, together with high-yielding-strength steel, seems to be very attractive from the economic point of view.

Concrete buildings whose design is controlled by severe abnormal loading -- For buildings which can be subjected to severe seismic excitations, it is highly desirable that the structural material have high energy absorption and energy dissipation capacities per unit weight under the different types of time history excitations that the building could be subjected to (24). To achieve this the material should possess:

(1) High strength (tension, compression and shear) per unit weight.

(2) High stiffness per unit weight.

(3) High internal damping per unit weight.

(4) High toughness per unit weight.

(5) High resistance (tensile, compressive and shear) to low cyclic fatigue.

(6) Stable hysteretic behavior under repeated strain reversals.

Furthermore the structural material should be homogeneous isotropic, easily adaptable and conducive to forming full-strength connections having the same characteristics as the materials itself. In the case of R/C this last requirement demands good bond.

In selecting the best structural material for earthquake-resistant construction, a simple plot of the ratio of stress per unit weight versus strain for the different available structural materials can be of use (Fig. 1). From this plot it is clear that plain normal weight concrete is not a desirable structural material for this type of construction. Its weakness in tension requires that it be reinforced. The usually small ductility of ordinary reinforced concrete dictates the use of confined reinforced concrete. The relatively low value of strength per unit weight of normal-strength concrete made of normal weight aggregate concrete suggests the desirability of using high-strength lightweight concrete. The advantages of using confined lightweight aggregate concrete can be seen from the results shown in Fig. 2. The results shown in this figure are for a lightweight aggregate concrete whose cylinder strength was 5.3 ksi. Note the need for confinement, not only to improve the strength but also the deformation capacity. Increasing the strength alone is not enough. The increase in deformation capacity is essential, so that when the concrete is properly reinforced there is compatibility of deformation up to a sufficiently high strain so the steel reinforcement, when working in compression, can be efficiently strained in its strain hardening range. If it is practically and economically possible to confine high-strength lightweight aggregate concrete of 12000 psi and obtain the ideal curve shown in Fig. 2, we would have a very desirable material, when properly reinforced, for seismic-resistant design of flexural members. The problem remaining to be answered follows:

(1) Is it practically and economically feasible to improve the mechanical characteristics of reinforced high-strength concrete by proper confinement, as has been done with reinforced normal-strength concrete? If the answer is yes, then:

(2) What are the consequences of using an unconfined cover shell to protect the reinforcement in confined high-strength concrete, particularly for columns subjected to high axial forces?

(3) How will the behavior of structural elements, whose critical regions can be subjected to reversal of high shear and bond strains, be affected when high-strength concrete is used?

These questions and their possible answers are discussed below.

Disadvantages and Possible Problems in the Use of High-Strength Concrete

In practically all reinforced concrete members, there are areas which can be considered laterally unconfined (the cover shell), and others which can be considered laterally confined (usually the central part). Initially (i.e., under low stresses--smaller than 0.70 f'_c), the behavior of confined reinforced concrete is similar to that of the unconfined reinforced concrete.

Unconfined concrete -- References 16, 20 and 22 discuss the factors requiring special attention in attaining high-strength concrete. Among these factors are the selection of materials and mixture proportions; and procedures of mixing, placing, consolidating, and curing. It appears that it is presently possible to attain a compressive strength up to 11,000 psi at 28 or 56 days without using extraordinary materials by using good aggregate (particularly stone aggregate). However, it appears it is possible to attain such strengths with very few of the current commercially available lightweight aggregates.

What is the inelastic behavior of such unconfined high-strength concrete? Although very little reliable data exist regarding such behavior, all the reported experimental results indicate that the higher the strength the more brittle the failure. To be more specific, the higher the f'_c the larger the rate of decrease in strength after reaching maximum strength. This is shown in the stress-strain curves in Fig. 3.

In analyzing the implications of the stress-strain curves for high-strength concrete usually reported in literature, it is necessary to consider the following two factors.

(1) The higher the strength of the concrete the more sensitve the stress-strain curve becomes, particularly in the inelastic range, to the testing procedure.

(2) In practice there is interest in the behavior of longitudinally reinforced concrete and not just plain concrete.

> Because of the stabilizing effect of the reinforcing steel, the inelastic behavior of concrete in a reinforced specimen is better than that of the same concrete in an unreinforced cylinder.

Wang, et al (27) have carried out experiments on high-strength concrete using a testing technique that permits reliable data to be obtained on the inelastic behavior of concrete cylinders. Some typical curves obtained in these experiments are shown in Fig. 3 (c). These curves confirm the first of the above factors, and show that for the same strength, lightweight concrete is more brittle than normal weight concrete.

There are still many who claim that the descending branch of the concrete stress-strain curve is not of particular interest, except to some researchers. Perhaps these people are interested only in estimating the axial and/or flexural strength of the cross section of a member. However, it is the descending branch of this curve that can control the inelastic behavior of real structural members. Therefore, the study of its real shape is of great importance.

Other important factors brought out by the analysis of the curve shown in Fig. 3 are:

(3) The higher the strength of the concrete the larger the strain ε_o at which the maximum strength i.e., f'_c, is reached. Values up to and even larger than 0.003 have been recorded for normal weight concrete of high f'_c.

(4) The value of the strain at which the maximum strength of lightweight aggregate is reached is higher than that for normal weight concrete of similar strength. Values of 0.004 and even larger have been recorded.

These last two observations are important because they indicate that high strength concrete cannot be used effectively with low yielding strength steel, particularly in columns subjected to very high service axial forces. From the above observations and the following discussion, it should be clear that there is an urgent need for research to obtain, as accurately as possible, the complete stress-strain relationship of high-strength concrete (normal and lightweight). This has been already noted in Refs. 22 and 27, and needs to be reemphasized. This stress-strain relationship should be obtained both from plain high-strength concrete specimens and, even more importantly, from longitudinally reinforced specimens, since this is how this concrete will be used in real structures.

Confined concrete -- It is beneficial to confine concrete in order to increase its maximum strength, and particularly to increase the deformation capacity of plain concrete. Although increasing the deformation capacity has great significance for the safety of concrete structures subjected to normal loading, it is of paramount importance for concrete structures than can be subjected to severe abnormal loading, particularly to severe seismic excitations.

Unfortunately, exploration of the pertinent literature resulted in no significant information concerning the effects of confinement on high-strength concrete. Therefore, what follows is a speculative discussion of what can happen to high-strength concrete when confined by lateral and longitudinal reinforcement in the amount and detailing used in real concrete structures. These speculations are based on results from studies on the effects of confining normal and lightweight concrete having strengths up to 6,000 psi (28-33).

Reference 28 presents a detailed discussion of the results obtained from concrete whose compressive strengths, obtained from plain concrete cylinders, are given in Table 1. The corresponding stress-strain curves are shown in Fig. 4. The confined concrete specimens consisted of special 6 x 18 in. cylinders. Confinement was provided by steel wire spirals 1/8 to 3/16 in., spaced at 0.5 in. to 0.75 in., with a yielding strength of 40 to 100 ksi. The spirals were so proportioned that at their yielding strength, confinement pressures, f_r, were produced varying from 0.11 to 0.34 of the compressive strength, f_c'. The range of confinement pressures corresponding to practical design conditions can be established from the ACI (Sec. 10.9.3 of Ref. 11) spiral requirement as follows:

$$\rho_s = \frac{4A_{sp}}{D_c s} = 0.425[(A_g/A_c) - 1]\frac{f_c'}{f_s} \qquad (1)$$

where

ρ_s - ratio of volume of spiral reinforcement to total volume of core

A_{sp} - cross-sectional area of spiral wire

D_c - diameter of core, out-to-out of spiral

s - spacing (pitch) of spiral

A_g - gross cross-sectional area of concrete

A_c - cross-sectional area of concrete core

f_c' - specified compressive strength of concrete

f_s - hoop stress in the spiral wire

The confinement pressure, f_r, defined below can be expressed in terms of Eq. 1 as follows:

$$f_r = \frac{2A_{sp}f_s}{D_c s} = 0.2125[(A_g/A_c) - 1]\,f_c' \qquad (2)$$

For values of (A_c/A_g) ranging from 0.4 to 0.7, the confinement pressure f_r falls in the range of 0.09 to $0.32f'_c$.

The effect of confinement pressure on the stress-strain characteristics of the five concretes, under monotonic compression and at low strain rate, is shown in Fig. 5. Deformation characteristics and gain in strength of confined concrete are sensitive to the type of aggregate used and to the relative amount of confining pressure. The yielding strength of the confining steel wire is also a very important parameter. The higher the yielding strength the more effective the confinement. This observation has been confirmed in a series of tests reported in Ref. 32, and more recently by results reported in Ref. 34.

Confinement of concrete - with all types of aggregate - is effective in developing large deformability, the ultimate strains in all cases being greater than 0.02 in/in. However, increase in compressive strength due to confinement is much greater for normal weight concrete than for lightweight concrete. After yielding of spiral steel, the strength of confined lightweight aggregate concrete may decrease to values lower than that of unconfined concrete. Thus, the higher the yielding strength of the lateral confining steel the better the resulting confinement.

It was found that maximum confinement, which can be obtained within practical limits of spiral spacing and yield strength of spiral steel wire, is not sufficient to achieve the expected increase in strength (of about 4 times the lateral pressure) for the lightweight aggregate concrete used in the study of Refs. 28-33.

Use of confined concrete in buildings -- The effectiveness of concrete confinement in producing extra strength and beneficial ductility in highly redundant structures (such as tall buildings) and particularly in earthquake resistant reinforced concrete structures is based on two conditions. These are: (1) that confinement increases compressive strength so that it is possible to offset the loss of strength from the loss of load-carrying capacity due to crushing and spalling of the unconfined concrete cover; (2) that confinement increases the capacity of concrete to sustain large deformation without loss of strength, thus transforming concrete from a relatively brittle material (when unconfined) to a relatively ductile material (when confined).

Results presented in Fig. 5 show that, for different concretes, these conditions are satisfied to a varying extent, and that the effectiveness of confinement is highly sensitive to the type of aggregate used and the strength of the unconfined concrete. The effectiveness of confinement can be characterized by two material constants, k_o and k_u, which are defined by relating the increased compressive strength, f^*_c, to the confinement pressure f_r.

The compressive strength of confined concrete, $f^*_{c\,max}$, occurs at some strain, ε^*_o, and can be defined as follows:

$$f^*_{c\,max} = f_c + k_o f_r \qquad (3)$$

where f_c is the compressive strength of the same concrete, but unconfined.

With very large deformations, $\varepsilon^*_u >> \varepsilon^*_o$, the compressive strength usually decreases to a value of f^*_{cu} and can be defined as follows:

$$f^*_{cu} = f_c + k_u f_r \qquad (4)$$

The confinement pressure, f_r, depends on the geometric and material characteristics of the spiral and can be expressed as follows (see Eq. 2):

$$f_r = \frac{2A_{sp} f_s}{D_c s}$$

Assuming that the ductile spiral wire yields when the longitudinal strain in the concrete is in the range ε^*_o to ε^*_u, and that strain-hardening of the spiral is negligibly small in the range of these strains, f_s is equal to f_y, and f_r can be calculated for given values of A_{sp}, D_c, and s from Eq. 2. Then, values of k_o and k_u can be calculated from Eq. 3 and 4 and test results. These values for the five different concretes used in the study of Ref. 28 are shown in Table 2.

Early investigators have shown that the confinement effectiveness coefficient k varies with lateral pressure intensity and with longitudinal strain. However, in developing ACI criterion for spiral requirement (Sec. 10.9.3 of Ref.11) and other similar criteria based on the confinement of concrete, a constant value of k, usually taken as 4.0 or 4.1, has been assumed.

As shown in Table 2, the values of k for normal weight aggregate concrete vary in range from 0 to 7.0. For the two lateral pressures ($0.13f_c$ and $0.32f_c$), values of k_o at maximum compression are 7.0 and 5.0, respectively, and values of k_u at ultimate strength are 0 and 3.1, respectively, Based on these values, and noting from Fig. 5 that concrete behaves in a relatively ductile manner throughout a significant range of strains, a constant value of k = 4.0 may be justified for concretes such as E-5.

For concretes B-3, B-5, R-3, and R-5, the values of k vary in range from -1.0 to 4.4. Negative values of k_u indicate that compressive failure in the confined concrete may occur at values below the compressive strength of unconfined concrete. For the two lateral pressures ($f_r \approx 0.1\ f_c$ and $f_r \approx 0.3\ f_c$), values for k_o, at maximum compression, range from 1.0 to 4.4 and values for k_u, at ultimate, range from -1.0 to 2.1.

Based on these results, a value for k in the range from 1.0 to 2.0 should be taken in developing design criteria based on the confinement of lightweight concrete when aggregates and steel wire spirals similar to those used in the investigation reported in Ref. 28 are used. In such cases, the amount of spiral steel required in a column of lightweight aggregate concrete will be 2 to 4 times as great as that currently prescribed by the ACI Code. Because of the geometric limitations introduced by the size of spiral wire and the minimum spacing, it would be virtually impossible to produce a spiral which would also allow normal placing of concrete.

The effect of the variable coefficient k is illustrated in Fig. 6. Loss of the capacity to carry load by spirally reinforced concrete columns due to spalling is plotted against k, assuming that the spiral reinforcement was designed in accordance with the ACI criterion. This loss of capacity is expressed as a ratio and derived as follows:

$$\begin{aligned}\text{Loss} &= 0.85f'_c(A_g - A_c) - kf_rA_c \\ &= 0.85f'_c(A_g - A_c) - 0.5k\rho_sf_sA_c \qquad (5)\end{aligned}$$

By substituting $\rho_s = 0.425\ [(A_g/A_c)-1](f'_c/f_s)$ (Eq. 1) into the above, and dividing by $0.85f'_cA_g$, the following ratio is obtained:

$$\frac{\text{Loss}}{0.85f'_cA_g} = (1 - \frac{A_c}{A_g}) - 0.25k(1 - \frac{A_c}{A_g}) \qquad (6)$$

For spirally reinforced square columns, (A_c/A_g) varies from approximately 0.4 to 0.6 and for round columns this ratio varies from approximately 0.5 to 0.7. The loss ratio for typical values of (A_c/A_g) is plotted in Fig. 6.

From above results it becomes clear that the low confinement effectiveness in some concretes may lead to significant losses in compression capacity when spalling occurs in reinforced concrete elements whose confinement is designed according to present ACI Code provisions (11). This is important when estimating the overall factor of safety against collapse for structures subjected to normal types of excitations. It is even more important for structures that can be subjected to severe abnormal excitations, as in the case of seismic resistance design of columns. These elements (columns) should be able, at all times, to resist the combined effects of the gravity loads and of the lateral forces (overturning moments).

What are the implications of the above results and observations for the use of high-strength concrete? Analysis of the little data available on behavior (stress-strain relationship) of high strength normal weight concrete indicates that the higher the strength of the concrete the more brittle the failure appears to be (Fig 3). It appears that the stress-strain curves of unconfined high-strength concrete (f'_c>7,000 psi) become quite similar in shape to the curves obtained for lightweight concrete having $f'_c \geq$ 5,000 psi. Since it has been shown that the higher the f'_c of the lightweight concrete the lower the confinement effectiveness, one can speculate that, as the f'_c of the unconfined normal weight concrete is increased the confinement effectiveness would decrease. However, it is believed that it will be possible to somewhat improve the confinement effectiveness, by: (1) using lateral reinforcement with very high yielding strength, no plastic plateau, and a high rate of strain hardening characteristics; (2) selecting better and better aggregate as the demanded f'_c is increased.

Another speculative implication can be formulated from analysis of Fig. 6. Use of high-strength concrete will result in relatively smaller column cross section. Thus, the A_c/A_g will be relatively smaller. Therefore, the losses when the cover spalls can be higher than when normal-strength concrete is used, if no special precautions are taken to increase the confinement effectiveness coefficient, k.

The beneficial effect of assuring large and stable inelastic deformation (i.e. without loss of strength) of high-strength concrete in members subjected to flexure and axial loads can also be appreciated from the following results (for more detail, see Ref. 31).

<u>Axial load-moment (P-M) interaction diagrams</u> -- Two types of reinforced concrete members (columns) were chosen to analyze the effects of confinement on the behavior of lightweight concrete structural members subjected to flexure and axial loads:

(a) a square 30 in. x 30 in. (762 mm x 762 mm) cross section column; and

(b) a circular column, 33 in. (838.2 mm) in diameter.

(See Figs. 7(a) and 7(b) respectively).

Both sections contained eight #18 reinforcing bars. The hoops in the square section had a 4 in. (101.6 mm) spacing whereas the spiral's pitch in the circular section was 2.1 in. (53.34 mm).

The volumetric ratio of the lateral reinforcement, as well as the ratio A_g/A_c, were similar on both columns. The volumetric ratio of spiral reinforcement ρ_s used for the circular cross section was 0.013, about 25% larger than that required by section A.6.5.2 of the ACI Code (11).

The analysis of the P-M diagrams for the section of these columns was performed by using a computer program (35). The computer program allows modeling of progressive spalling of the concrete cover as this reaches its maximum strain. Also, different f_c - ε_c relationships for concrete core and cover may be used.

Figures 8.(a) and 8.(b) show the f_c - ε_c relationships for the confined concrete in the core of the column section, which were used in the analysis of the square and circular cross sections respectively. Figure 9.(a) shows the f_c - ε_c relationship that was used for the unconfined concrete in the cover of both types of sections. The reinforcing steel f_s - ε_s relationship used is shown in Fig. 9.(b). The strain hardening portion of this relationship was idealized as a cubic polynomial.

The same sections of the columns were analyzed using a fictitious f_c - ε_c relationship for the concrete, which is implied when assuming the equivalent stress-block distribution, and a more realistic f_c - ε_c relationship which was proposed by Hognestad and widely used in practical application. The longitudinal steel was idealized as an elastic-perfectly plastic relationship. The relationships are depicted in Figs. 10(a), 10(b) and 10(c).

For each of the two sections shown in Fig. 7, the following interaction curves have been plotted in Figs. 11 and 12.

(a) An interaction P-M curve based on the f_c - ε_c implied when assuming the equivalent rectangular stress-block distribution, an elastic-perfectly plastic relationship for the steel, and the maximum concrete strain, to be $\varepsilon_c = 0.003$. (Fig. 10).

(b) An interaction P-M curve based on the more realistic f-ε relationships and for the case when the concrete strain at the maximum compressive fiber of the cross-section reached a value of 0.003 (i.e., $f_{c(cover)} = f'_c$). From Figs. 11 and 12 it can be observed that, at this concrete strain, the axial load strengths of the two columns are at maximum.

(c) An interaction P-M curve for the case when the extreme fiber of the concrete cover reached $\varepsilon_c = 0.0045$.

(d) An interaction P-M curve for the case when the confined concrete reached a strain of $\varepsilon_c = 0.05$, which is close to the experimentally obtained buckling strain of the longitudinal reinforcement for a reinforced column where concrete is laterally confined with circular spirals.

From a comparison of the above plots for the rectangular cross section, Fig. 11, the following observations can be made. At a concrete strain of $\varepsilon_c = 0.0045$ (although, under pure axial load, the cover has completely spalled), the axial-load strength of the column's concrete core differs very little from that calculated using the ACI equivalent stress-block distribution over the whole cross-section. It is also clear that the P-M interaction curve--as determined by the fictitious f_c-ε_c -- is unconservative for loads above the balanced point. When the experimentally obtained confined concrete f_c - ε_c relationship is used, it can be seen that, at very large deformations, the flexural capacity is increased. This is mainly because of the steel's strain hardening. For loads below the balanced point, if buckling is delayed, the lateral confinement will allow for increases in moment capacity from 23267 k-in. (fictitious f_c - ε_c curve) to 34059 k-in. (confined concrete curve) or an increase of 46% when P = 0. Also, due to the strain hardening of the longitudinal steel, the axial-tension capacity of the column is actually about 50% above the one predicted according to the elasto-perfectly plastic assumption of Fig. 10c. These considerations are important in seismic-resistant design because a column may undergo tension under a strong seismic ground motion due to overturning moment effects. Similar observations can be made from analysis of the plots obtained for the circular cross section.

As can be observed from a comparison of results presented in Figs. 11 and 12 the performance of the circular column section at very large strains is better than the behavior of the square cross-section column at similar strains. This is due to the more beneficial effect of circular spiral confinement on strength of the concrete at these large strains. For the ratio, A_g/A_c, used in this section, the axial load strength is maintained up to the strain when the cover begins to spall off. However, the envelope curve based on the stress block and $\varepsilon_c = 0.003$ overestimates the flexural capacity of the section when strains in the confined core and concrete cover reach $\varepsilon_c = 0.003$ or $\varepsilon_c = 0.0045$. It is only when large concrete strains have developed ($\varepsilon_c = 0.05$) that the capacities based on the two different f_c - ε_c (equivalent stress block vs. confined concrete) are similar for loads above the balanced point and as observed for the square cross-section, both the moment and the tension capacity of the column, as predicted by the relationship for the confined concrete, are considerably larger than those predicted by the fictitious f_c - ε_c derived from the equivalent rectangular stress block and the elastic-perfectly plastic f_s - ε_s of Fig. 10c.

Figure 13 compares the effect of increasing the lateral confining pressure for normal weight and lightweight concrete on columns with circular cross-section. In this case, f_c - ε_c relationships for the concrete are shown in Fig. 14 for normal and lightweight concrete, respectively. These relationships were found experimentally from tests carried out at the University of California in trying to determine the effects of lateral reinforcement on the behavior of normal and lightweight concrete columns (32). The confinement characteristics of these sections are shown in Fig. 14. As can be observed from Fig. 13, for an increase of about three times in the lateral reinforcement (from 0.013 to 0.043), the increase in axial and flexural capacity is considerable. The effect on normal weight concrete is even more remarkable. In Fig. 15 the interaction curves based on ACI's relationship are compared with curves obtained by considering $\rho_s = 0.043$. It is observed that it is only at this very high level of confinement that the ACI based curves are on the conservative side.*

From the above discussion it needs to be noted:

(a) There is a significant difference in axial and flexural capacity exhibited by similar reinforced concrete sections, one fabricated with normal weight concrete and the other with lightweight concrete. This indicates that present ACI code minimum requirements for lateral reinforcement may not be satisfactory when lightweight concrete is used.

(b) A design philosophy based on the criteria of strength only (a fictitious $f_c - \varepsilon_c$ implied by the equivalent stress block or a Hognestad's type relationship) may lead to an unconservative design. This is because spalling of the cover (especially in cases where the ratio A_g/A_c is large) signifies loss in capacity. It is only when a high volumetric ratio of lateral reinforcement is provided and large concrete strains are developed (which requires special precautions in designing and detailing the lateral reinforcement) that the original combined axial and flexural strength will be recovered once the cover has spalled, particularly for cases of high axial forces.

From the above results and discussion it becomes clear that there is an urgent need for obtaining reliable information regarding the complete stress-strain relationship of confined high-strength concrete as it is affected by: the compressive strength f'_c; the type of aggregate and other materials; mixture proportions and procedures of mixing, placing, consolidating, and curing; the confinement pressure f_r; the mechanical characteristics of the reinforcement; the detailing of both the lateral and the longitudinal reinforcement--diameter, spacing--; and the variation of the f_c vs ε_c according to distribution of strain along section.

After the above information is available, study should be conducted on the soundness of present ACI code methods to evaluate the P-M interaction curves.

---

*ACI relationships for concrete imply either a Hognestad's relationship or a fictitious one based on the equivalent rectangular stress-block distribution.

Other possible disadvantages or problems -- The fact that the use of high-strength concrete leads to a reduction in the cross-section, can result in an increase in severity of the following problems.

(1) Decrease in stiffness. An analysis of the rate of increase of the modulus of elasticity of the concrete, E_c, with the strength of the concrete shows that this rate is considerably lower than 1. Thus, the use of high-strength concrete will lead to members with relatively greater slenderness and smaller stiffness. While it is true that this will demand more careful attention to the "stability problems" (buckling of individual members or stability of the structure as a whole) the author believes that these problems can be overcome (solved) by proper selection of the structural system, i.e., additional lateral stiffening systems such as shear walls. Thus, this problem will not be further discussed herein.

(2) Problems created by volumetric changes (shrinkage and creep) require scrutiny, particularly for columns of tall slender buildings which can undergo rapid and high intensity fluctuations in axial forces, as in the case of severe abnormal loadings. In this case the concrete can undergo *significant cracking* even *if* the whole member is not under a net tension. This cracking can significantly decrease the shear resistance of such columns.

(3) Problems created by the fact that the bond strength does not increase at the same rate that the compressive strength of the concrete. This can lead to serious problems, particularly at the beam-column joints, when the use of high-strength concrete is accompanied by the use of high-strength steel.

(4) Problems created by the fact that the contribution of the concrete to the shear resistance of members made of high-strength concrete can decrease when compared with members made of normal-strength concrete.

These last two types of problems will be discussed in more detail later.

INELASTIC BEHAVIOR OF STRUCTURAL ELEMENTS, OF THEIR CONNECTIONS, OF THEIR SUBASSEMBLAGES, AND OF WHOLE STRUCTURES

Need for Ductile Inelastic Behavior of Structural Elements

In the previous section the importance was shown of confining concrete to improve the $f_c - \varepsilon_c$ relationship for concrete with a high f'_c. Also shown was the need for obtaining reliable data on this relationship in order to predict the P-M interaction curves of members subjected to flexure and axial loads. Also discussed was the desirability of having a large deformation capacity (ductility) for concrete structures, particularly structures that may be subjected to severe abnormal excitations. The information presented in the previous sections causes certain questions to be raised. These questions are discussed below.

What are the requirements for achieving ductile inelastic behavior of a concrete structure? Why is it beneficial to have ductile inelastic behavior?

Obviously, to attain a ductile structure its elements should be ductile. A necessary but not sufficient prerequisite for achieving ductile inelastic behavior of structural elements is that the structural material be ductile. Then the question to answer is: can properly reinforced high-strength concrete offer sufficiently ductile behavior to permit the practical and economical design and construction of sufficiently ductile structural elements? According to the available data and speculative discussion presented in the previous sections it appears that the higher the strength of the concrete the more difficult it will be to achieve large inelastic deformations without loss of strength. However it seems that by careful confinement it would be possible to attain the ductility required by most of the buildings.

Why is it beneficial to have ductile inelastic behavior and how great should the ductility be? In answering this question it is convenient to distinguish the case of structures subjected only to normal excitations from those that can also be subjected to severe abnormal excitations.

<u>Concrete structures subjected only to normal excitations</u> -- In this case, ductility is a desirable feature that permits a larger safety factor against possible overloads to be obtained. It permits the structure to deform considerably before collapse, giving time for safe evacuation of the structure. Use of ductility permits economical redistribution of forces and moments in the design of structures. This is recognized in the present ACI 318-77 ultimate strength method of design through the use of empirical equations (see sections 8.4 and 18.10.4 of

Ref. 11). The empirical equations given in the code are based on results of experimental and analytical studies conducted on continuous beams made of normal-strength concrete. Thus there is need to investigate the validity of these equations in case that high-strength concrete is used. This would require determining the deformation capacity of the different structural members under monotonically increasing deformation.

Concrete structures subjected to severe abnormal loadings -- Let us consider the case of concrete buildings located in a site that may be subjected to severe earthquake ground motions. In this case the effect of these severe seismic excitations control the design. Economic reasons require that design be based on the energy dissipation capacity of the structure. In general, the larger the ductility the more economical the design. Therefore, the following problem needs to be solved: how much ductility can be developed economically, both under monotonically increasing excitations and particularly under the generalized variable repeated loadings that can be induced by abnormal excitations.

Reinforced concrete structures, located in zones of high seismic risk and designed according to present seismic code provisions, are expected to undergo several cycles of deformation excursions well into the inelastic range when subjected to the maximum credible earthquake ground motions at the site. Analytical methods for estimating the ductility demands of a structure have been developed [36-38]. Once the "required ductility" is assessed, the designer must have some way of predicting the available ductility for the selected structure in order to design and detail it for the required ductility. It should be emphasized that, because present methods used to estimate ductility demand are based on simplified mathematical models and procedures which might lead to unconservative estimations of the actual demand, the designer must ensure that the structure is supplied with a larger ductility than that required.

A large ductility requirement for lateral displacement can be achieved through localized inelastic deformations that occur at certain critical regions along the members of the structures. These critical regions are usually located around the sections where the stresses in the steel or concrete reach their "yielding" values. In reinforced concrete structures, this may occur at practically any section since the member can be designed with variable strength; it is possible to tailor the reinforcing steel and the sizes of cross-sections to any selected or estimated envelope of internal forces. Fortunately, because of economic considerations and uncertainties about loading conditions (i.e., selection of moment envelope), sizing of members and distribution of reinforcement are usually done in such a way that the critical regions are located around the points of peak internal forces. These regions are illustrated in Fig. 16 for just one floor where nine different regions are indicated. These regions will be denoted hereafter as "critical regions."

Depending on the type of structural system, the relative stiffness of members, and the detailing of these members and their connections, the ductility requirements for local deformations occurring at the critical regions may considerably exceed the ductility requirements for the lateral displacement. Therefore, the requirement of a lateral displacement ductility factor, although necessary, is not by itself sufficient to prevent failure under a severe earthquake. Results obtained in a study (39) have shown that analysis based on linear-elastic response generally overestimates the deformation ductility in columns and underestimates them in girders as predicted by an elasto-plastic analysis. For reinforced concrete structures, accurate values of the required ductility at critical regions are crucial since the available rotation capacity is sensitive to the type, amount, and detailing of the reinforcement.

Thus, to obtain the required ductility of a reinforced concrete structure, it is necessary to predict the ductility of their members which, in turn, depends on the ductility of their critical regions. This last is usually measured by the rotation capacity obtained from the curvature ductility of their sections using the principles of continuum mechanics.

To achieve large ductility under generalized dynamic excitations induced by severe ground shaking, the structure must be designed and detailed such that flexure dominates the behavior of the critical regions. The following reviews the available ductility of flexural critical regions.

The behavior of R/C critical regions is very sensitive to the history of actions the regions are subjected to. Therefore, in determining the ductility of such regions, at least two types of behavior should be distinguished: (1) monotonically increasing curvature or bending moment; and (2) generalized or variable curvature or bending moment.

Available Ductility of R/C Flexural Critical Regions Subjected to Monotonically Increasing Curvature

General expressions for estimating strength and ductility of doubly reinforced concrete sections -- The available section ductility of R/C flexural regions can be obtained by obtaining the moment-curvature (M-ϕ) diagram of a section for a curvature that increases monotonically from zero up to its maximum value, ϕ_{max}. Such a diagram is illustrated in Fig. 17 including the most significant points of the moment-average curvature ($M - \phi_{average}$) relationship. Using the notation illustrated in Fig. 18 the values of M and $\phi_{average}$, corresponding to these points in the cracked state, can be obtained using the following general Eq. (7) and (8). Note that, in this cracked state, strictly speaking, it is not possible to define the curvature at a cracked section. Furthermore, because it is not possible to measure curvature at a section, it is more realistic to work with M vs. $\phi_{average}$ ($\phi_{avg.}$).

General equations for predicting curvature and moment -- (See Fig. 18)

$$\phi = \frac{(\varepsilon_c)_y}{y} = \frac{(\varepsilon_c)_{ymax}}{kd} = \frac{\varepsilon_s}{(1-k)d} \qquad (7)$$

$$M = bd^2 f'_c \left[k\, k_1\, k_3\, (1-k_2k) + \rho' \frac{f'_s}{f'_c} (1- \frac{d'}{d})\right] \qquad (8a)$$

or

$$M = A_s f_s jd = bd^2 j\rho f_s \qquad (8b)$$

According to the general definition of ductility ratio or ductility factor, and the notation used in Fig. 18, curvature ductility ratio, μ_ϕ, can be defined as:

$$\mu_\phi = \frac{\text{curvature at failure}}{\text{curvature at yield}} = \frac{\phi_{max}}{\phi_y} \qquad (9)$$

Curvature at Yield $\qquad \phi_y = \frac{\varepsilon_{sy}}{(1-k_y)d} \qquad (10)$

The neutral axis depth factor k_y is given by*

$$k_y = \sqrt{\left[(\rho + \rho)^2 n^2 + 2(\rho + \frac{\rho' d'}{d})n\right]} - (\rho + \rho')n \qquad (11)$$

Where

$$n = \frac{E_s}{E_c} = \frac{\text{modulus of elasticity of steel}}{\text{modulus of elasticity of concrete}}$$

Curvature at Failure $\qquad \phi_{max} = \frac{\varepsilon_{cmax}}{k_{\varepsilon_{cmax}} d} \qquad (12)$

*In the derivation of this equation, effects of volumetric changes have been neglected. The main purpose for this equation is to give an idea of the relative importance of the different parameters affecting the ductility rather than to accurately estimate its value.

It can be shown that for

Unconfined section:

$$k_{\varepsilon_{cmax}} = \frac{(\rho f_s - \rho' f'_s)\varepsilon_{c\,max}}{(k_1 k_3)\varepsilon_{cmax} f'_c} \qquad (13)$$

Confined section after cover spalls:

$$k_{\varepsilon_{cmax}} \approx \frac{(\rho f_s - \rho' f'_s)\varepsilon_{c\,max}\,bd}{(k_1 k_3)\varepsilon_{c\,max} f'_c\, b_c d_c} \qquad (14)$$

Incorporating Eqs. 10-13 into Eq. 9, the available section ductility for a doubly reinforced unconfined section may be expressed as:

$$\mu_\phi = \frac{\varepsilon_{c_{max}}(1-k_y)}{\varepsilon_{sy}\, k_{\varepsilon_{c_{max}}}} = \frac{\varepsilon_{c_{max}}}{\varepsilon_{sy}} \; \frac{\left(1 - \left[\sqrt{(\rho+\rho')^2 n^2 + 2\,(\rho+\rho'\frac{d'}{d})n} - (\rho+\rho')n\right]\right)}{(\rho f_s - \rho' f'_s)\,\varepsilon_{c_{max}} \Big/ (k_1 k_3)\,\varepsilon_{c_{max}}\, f'_c} \qquad (15)$$

For a doubly reinforced confined section the available μ_ϕ after the cover has spalled may be estimated from an equation similar to 15 except that it should be multiplied by the ratio $b_c d_c/bd$ (see Fig. 18).

Analysis of the above equations for curvature ductility, and plots that have been made from equations similar to these, shows that with the other variables held constant (40):

1. An increase in the extreme fiber concrete strain at failure, $\varepsilon_{c_{max}}$, increases the μ_ϕ because it increases ϕ_{max}.
2. An increase in the steel yield strength decreases the μ_ϕ because both ε_{sy} and $k_{\varepsilon_{c_{max}}}$ increase; therefore ϕ_y is increased and ϕ_{max} is decreased.
3. An increase in the ratio of tension reinforcement ρ decreases the μ_ϕ because both the k_y and the $k_{\varepsilon_{c_{max}}}$ are increased; therefore ϕ_y is increased and ϕ_{max} is decreased.
4. An increase in the ratio of compression reinforcement ρ' increases the μ_ϕ because both the k_y and the $k_{\varepsilon_{c_{max}}}$ are decreased; therefore ϕ_y is decreased and ϕ_{max} is increased.
5. An increase in concrete strength, f'_c, increases μ_ϕ because both k_y and $k_{\varepsilon_{c_{max}}}$ are decreased; therefore ϕ_y is decreased and ϕ_{max} is increased.

Examining Eq. (15), which can be considered representative of the general case of a flexural section, we see that μ_ϕ is directly proportional to $\varepsilon_{c\,max}$. In the case of unconfined normal f'_c and normal-weight concrete, although no significant reduction in resistance occurs until $\varepsilon_{c\,max} = 0.004$, at which time some spalling of concrete can be detected, this value of $\varepsilon_{c\,max}$ is usually taken as 0.003, i.e., the value at which a cross section usually reaches its maximum flexural resistance. Thus, the curvature ductility ratio is limited by this low value of $\varepsilon_{c\,max}$. To overcome this low ductility ratio, it is necessary to increase $\varepsilon_{c\,max}$ by confining the concrete in the compression zone of the section with closely spaced transverse reinforcement.

In order to take advantage of the larger $\varepsilon_{c\,max}$ when using confined concrete and compression steel, ρ', it is necessary to properly restrain the compression reinforcing steel against inelastic buckling. This might require that the lateral reinforcement be spaced even more closely than required to confine the concrete (14, 31). The ideal would be to confine the concrete and to restrain the compression steel against buckling so that ϕ_{max} be controlled by the rupture of the tension steel. Then the following ideal maximum value for μ_ϕ could be obtained.

$$\mu_{\phi_{ideal}} = \frac{\varepsilon_{s_{f_s\,max}}}{\varepsilon_{sy}} \frac{(1-k_y)}{(1-k_{\varepsilon_{s_{f_s\,max}}})} \tag{16}$$

Experiments described in Ref. 41 have shown that it is possible in practice to obtain this upper bound of μ_ϕ.

Effects of using high-strength concrete on the M-ϕ relationship -- In discussing the possible effects of high-strength concrete on the M-ϕ relationship it is convenient to distinguish between unconfined and confined high-strength concrete.

Flexural strength of unconfined high f'_c -- The increase of concrete strength makes little difference to the flexural strength of members whose tension steel yields before the concrete crushes.

Flexural ductility of unconfined high f'_c -- The use of high f'_c can lead to an increase or decrease in the μ_ϕ depending on the relative weight in Eq. 15 of the three following factors, which have opposite effects on μ_ϕ.

1. Higher f'_c -- This will have a beneficial effect because both k_y and $k_{\varepsilon_{c_{max}}}$ will decrease.

2. More abrupt descending branch of the f_c vs ε_c curve -- This will have the effect of requiring higher $k_{\varepsilon_{c_{max}}}$, thus decreasing μ_ϕ.

3. Lower $\varepsilon_{c\,max}$ -- this will result in a direct decrease of μ_ϕ.

<u>Flexural strength of confined high f'_c</u> -- If the tension reinforcement yields before the concrete crushes, the increase of f'_c will make little difference to the flexural strength. However, if the deformation capacity of the confined concrete is sufficiently high, and buckling of the compression reinforcement is delayed to permit the tension steel to be strained well into its strain hardening range, then a significant increase in flexural strength can be achieved (see Figs. 13 and 15).

<u>Flexural ductility of confined high f'_c</u> -- From data on confining normal-strength concrete with higher and higher strength concrete, it appears that as f'_c is increased it will be more difficult to increase the deformation capacity without any significant loss in strength. Thus, the beneficial effect that a higher f'_c has on μ_ϕ can be counteracted by the more abrupt descending branch of the f'_c vs ε_c curve. A solution to this problem appears to be the use of confinement reinforcement with very high yielding strength and at the minimum practical spacing.

<u>Concluding remarks</u> -- From above discussion it would appear that in case of R/C structures whose inelastic behavior is controlled by monotonically increasing types of excitations (loading and/or deformations) there is little to gain by using concrete with high f'_c in the flexural members, i.e., members subjected to relatively low axial and shear forces, as the case of beams. This is because the increase in f'_c makes little difference to the flexural strength if the beams are underreinforced, as is usually recommended; and there can be a decrease rather than an increase in available ductility. If in spite of this, concrete with high f'_c is used in flexural members of structures that are subjected to only normal types of excitations, there are then several problems that need to be investigated.

(1) Can present code methods (11) for predicting the flexural strength be applicable when concrete with high f'_c is used? The little data available on this problem appears to be contradictory. While in Ref. 17 Leslie, et al., concluded that "the ACI rectangular stress block does not predict the behavior of beams with f'_c above 8000 psi (55 MN/m^2)" in Ref. 42 Wang et al. concluded that "Rectangular stress distribution gives sufficiently accurate predictions of the ultimate loads and moments of reinforced concrete beams and columns made with high-strength concrete".

(2) Can present code provisions (11) on redistribution of negative moment in continuous flexural members be applicable when concrete with high f'_c is used? To the best of the writer's knowledge no data is available on this problem.

For flexural members of structures whose design can be controlled by collapse against severe abnormal excitation inducing monotonically increasing deformation (such as the case of severe blasts), there is a need to investigate how to obtain satisfactory confinement effectiveness when concrete with high f'_c is used.

Effect of Generalized Variable Repeated Loading on the Hysteretic Behavior of Flexural Critical Regions

Results from experimental studies (13, 14, 43) indicate that the behavior of R/C structural members under generalized actions similar to those which occur in severe seismic ground motions, including reversals of bending moments, is characterized by a loss of stiffness that increases as the number of cycles of severe deformation reversals is increased. Although a loss of stiffness did not prevent properly reinforced critical regions from developing their ultimate strength, the initial stiffness at load reversals decreased and the deformation at which the carrying capacity was reached increased as the number of alternating load cycles increased. This reduction in stiffness was also observed in tests performed on actual multistory buildings (44-47).

For reinforced concrete structures designed on the basis of code requirements, the more serious problem, then, appears to be one of stiffness rather than strength deterioration.

Mechanics of stiffness deterioration -- The moment and curvature at critical regions of reinforced concrete members subjected to large inelastic deformations are sensitive to: (1) the inelastic behavior of steel reinforcement which often exhibits a pronounced Bauschinger effect; (2) the degree of cracking in the concrete; (3) the effectiveness of composite action (bond) between steel and concrete; (4) the possibility of slippage or loss of effective anchorage; and (5) the presence of shear deformations and diagonal shear cracking. These factors are all sensitive to the stress history of the structure during an earthquake and often lead to a decrease of stiffness in successive cycles of loading. This decrease is commonly referred to as "degradation" or "deterioration."

The role of some of the above factors in stiffness deterioration has been described in previous publications (13, 14, 43) and is summarized below with reference to a cantilever beam shown in Fig. 19 (43).

If a doubly reinforced concrete member is loaded well into the inelastic range, causing yielding in the tensile steel, the major flexural crack denoted as C_1^t in Fig. 19(a), will not close completely upon unloading, Fig. 19(b). The degree of opening will depend on how far the tensile steel was strained into the plastic range during first loading. If strained well beyond the initial yielding, a crack C_2^b may originate on the bottom side during unloading.

If the member is then loaded in the opposite direction (Fig. 19(c)), the critical section, which is already cracked, will offer considerably less resistance to rotation than it does during the first loading. This decrease in resistance may be due to the imperfect contact between the two faces of a prior crack, C_2^t. The crack at the top may or may not close depending on the peak value of the reversed load P_3 compared with P_1, the amount of top and bottom reinforcing steel, and other factors. Because the concrete in the two faces has undergone a process of disruption, a reduction in the stiffness of the critical region should occur even if the crack closes.

If the load P_3 in the reversed direction reaches the same peak value as P_1, the width of the crack C_3^b will be larger than that of C_1^t, which was obtained under P_1. If the members is now unloaded, Fig. 19(d), the critical cross section will be cracked throughout C_4^t - C_4^b and the width of the crack will depend mainly upon the amount of yielding of the steel, the effectiveness of bond between steel and concrete and, to a lesser extent, on the degree of concrete disruption.

At the start of a new cycle of alternating load, the original concrete section will behave as a steel cross section represented by tensile and compressive steel reinforcement. If the reinforcing steel exhibits a pronounced Bauschinger effect, this will result in a reduction of stiffness in the critical region. Furthermore, the presence of shear at this stage will tend to displace the faces on either side of the crack relative to one another, as illustrated in Fig. 19(e). This tendency is resisted by the dowel action of the main reinforcement and will cause the steel bars to be pressed against the concrete and may possibly lead to longitudinal splitting of the concrete. The degree of damage introduced by this shear effect will depend on the tie spacing but, most likely, it will affect the bond and, consequently, the overall stiffness of the member.

Bond deterioration, increased by the shear in thoroughly cracked sections, may cause local failure at any point of discontinuity in the main reinforcement and, particularly, in the beam-to-exterior column joint. For example, in the joint shown in Fig. 19(f), deterioration of bond along length BA due to alternating steel stress and the effect of any shear force acting in section C^t - C^b may lead to the development of high radial stresses at A and subsequent bearing failure. This could produce significant slippage, pull-out of the bar and an outward movement of the vertical leg of the hook, thereby inducing spalling of the concrete in the back column face. This spalling is even more likely if the concrete confinement in the joint is inadequate. It has been observed by other investigators (see section 13.8.3 of Ref. 40); and in field inspections of earthquake damages.

The mechanics of stiffness deterioration described above have been derived from observations of experiments conducted with normal-strength concrete flexural elements. However, the same behavior will be obtained if concrete with high f'_c is used.

Effect of Axial and Shear Forces on the M-ϕ Relationship of Flexural Critical Regions

The moment-curvature relationship of flexural critical regions can he significantly influenced, particularly in the inelastic range, by axial and shear forces. Although the presence of moderate compressive axial forces can lead to a significant increase in the flexural strength of R/C sections (Figs. 11-13), in general the presence of high shear and axial forces have detrimental effects on the inelastic behavior of R/C elements, particularly if they are subjected to generalized variable repeated loading. The main cause of these accentuated detrimental effects can be found by analyzing the stiffness deterioration mechanism illustrated in Fig. 19. The presence of an axial tension force in the flexural critical region will increase the opening of the crack. Thus the higher the tension force the larger the flexural stiffness deterioration. Similarly, the higher the shear force the larger the relative displacement between the two faces of the crack (Fig. 19(e)) and therefore the larger the damage to the stiffness of the member.

While the presence of certain amounts of axial compression force can lead to both an increase in flexural strength and a decrease in the stiffness deterioration caused by the mechanism of Fig. 19, in general the presence of compression leads to a reduction in the deformation capacity of a flexural member. It is in the construction of flexural elements with high axial compression forces (columns) where the use of high-strength concrete offers the greatest advantage. Therefore the effects of axial and shear forces in the flexural behavior of these members are of interest. Rather than doing this in a general manner, the writer has chosen to summarize the effects of these forces on each of the different basic elements of the different R/C structural systems, i.e., beams, columns, beam-column joint, and shear walls. The writer has emphasized columns and shear walls of tall buildings where use of concrete with high f'_c has been accepted by the building industry. A detailed discussion of the hysteretic behavior of all these elements except the shear wall, is presented in Ref. 14.

Beams

It is convenient to classify the effects of axial and shear forces in the hysteretic behavior of beams according to their use in moment-resisting frames or in coupled shear walls.

Moment-resisting frames -- The magnitude of axial forces in beams is usually small and its effect on the M-ϕ relationship can be neglected. The shear forces on the beams, may not be negligible, especially in lower stories of tall buildings having short-span bays, and could significantly affect the M-ϕ relationship. Inelastic behavior of beam critical regions is very sensitive to type of excitations. While under monotonically increasing loading and/or deformations the behavior of flexural critical regions is not very sensitive to the amount of shear, if this does not exceed the amount permitted by present codes, i.e., $v_u \leq 10\sqrt{f'_c}$ (psi). Under excitations, such as severe seismic excitations, producing loading and deformation reversals of the flexural critical regions, the behavior is very sensitive to the amount of shear induced in the critical regions. Thus, it is convenient to classify beams or flexural critical regions that can be subjected to significant loading and/or deformation reversals as follows.

<u>Beam critical regions subjected to low shear: $v_u < 3\sqrt{f'_c}$ (psi)</u> -- Present code requirements for the design of these beams results in satisfactory hysteretic behavior. Furthermore, this hysteretic behavior can be predicted quite accurately analytically (12, 48). The use of high strength concrete should neither offer large advantages nor create serious problems for this type of beams.

Present code requirements regarding design against shear forces (the use of nominal shear stress as the main design parameter, v_u, and its value limitation as a direct function of $\sqrt{f'_c}$), can lead to some problems even when the code computed, v_u, does not exceed $3\sqrt{f'_c}$ (psi), particularly when the f'_c used is very high. Research is needed to find out if it is necessary to establish new lower limits for v_u if high-strength concrete is used.

<u>Beam critical regions subjected to high shear: $v_u > 3\sqrt{f'_c}$ (psi)</u> --When beams using normal-strength concrete have been designed according to present U.S. seismic codes, and their flexural critical regions are subjected to nominal shear stresses exceeding $3\sqrt{f'_c}$ (psi), these regions are capable of developing maximum flexural strength and large flexural deformation capacity under monotonically increasing loads. However, this kind of critical region, under repeated moment reversals and particularly under full rotation reversals, will undergo a degradation in stiffness and energy absorption and dissipation capacities considerably larger than the degradation of a similar critical region with very low shear stresses. Although such regions are capable of developing flexural yielding strength, an early shear failure mechanism (sliding shear) starts to develop after one cycle of full bending reversals beyond the yielding strength level.

A detailed discussion of the effects of high shear in flexural critical regions is given in Ref. 48. Figure 20 illustrates the results obtained from two beams, R-5 and R-6, which were tested to examine the effects of a high shear force on flexural critical regions (48). These beams were identical except for their shear span. The shear span of R-5 was $\ell/d = 2.75$; the shear span of R-6 was $\ell/d = 4.46$. Comparison of Figs. 20(a) and 20(b) show the pinching effect induced by high shear on the load-displacement relationship. This pinching effect resulted in a reduction in the energy dissipation capacity of more than 66 percent - (349 k./in. for beams R-5 vs. 738 k./in. for beam R-6). There was also a reduction in the plastic hinge rotation capacity from 0.036 radians to 0.026 radians. This difference in hystetic behavior can be explained as follows.

The shear resistance in cracked R/C critical regions subjected to monotonically increasing load is developed through: (a) shear stress of uncracked concrete; (b) aggregate interlocking and frictional resistance along cracked faces; (c) web reinforcement resistance at inclined cracks; and (d) dowel action of the main steel reinforcement. As the beam is subjected to several loading reversals, flexural and/or flexure-shear cracks may develop across the entire beam section; therefore, the shear must be resisted by web reinforcement, dowel action, and aggregate interlocking and friction. The last two resistances become less effective as the crack width increases and concrete crushes in the compression zone. As a result, large shear distortion could occur and become an important source of beam deflection as well as a significant parameter in the overall behavior of the flexural member. It should be re-emphasized, however, that this degradation occurs because of the opening of the cracks induced by yielding of the main reinforcement and is therefore a combined flexure-shear type of degradation mechanism. Because bond slippage of the main reinforcing bars can contribute significantly to the opening of flexural cracks, the deterioration observed is the result of a combined flexure-bond slippage-shear type of degradation.

At large ductility, say > 3, the deformation pattern in the critical region is dominated by the shear deformation at those cracks which remain open throughout the entire beam section. For this reason this behavior has been named "Shear Sliding" and the resistance mechanism "Interface Shear Transfer" (49).

After flexural yielding occurs in both loading directions, the degradation of shear resistance and the amount of shear distortion increases with the magnitude of applied load and/or deformation as well as with each repeated cycle of reversal. The possible shear degradation mechanisms include: (a) the opening of cracks due to yielding and or slippage of the main reinforcement; (b) the spalling of the concrete cover around the periphery of the flexural critical region; (c) the degradation in the stirrup-tie anchorage due to large variations in the strains where it is crossed by inclined cracks, and/or by the splitting and spalling of the concrete cover; (d) the crushing and grinding of concrete at the crack surfaces which could lead to a less effective aggregate interlocking resistance along the open cracks; and (e) the local disruption of bond between the longitudinal steel and concrete due to the dowel action along the open cracks.

Experimental results have shown that the behavior of flexural critical regions under high shear stress can be significantly improved by the addition of diagonal reinforcement. This is illustrated in Fig. 21 which compares the results obtained with different types of web reinforcement (50). The use of diagonal reinforcement is an effective means of controlling sliding shear (14).

Beams with shear stresses higher than $6\sqrt{f'_c}$ (psi) do not perform in a way that is totally satisfactory, regardless of the type of web reinforcement that is used.

The effects of using high-strength concrete in flexural critical regions subjected to high shear has been examined, holding all other parameters constant. It appears that if design against flexure and shear is done according to present code requirements, the observed stiffness and strength deterioration in hysteretic behavior will be somewhat accentuated because of the presence of higher shear stress in these critical regions. However, this problem can be easily solved by: (1) establishing more stringent requirements regarding maximum acceptable v_u, i.e., expressing this maximum acceptable nominal shear stress as $v_{u\ max} < \alpha\sqrt{f'_c}$ where α is a decreasing function of f'_c; and/or (2) addition of diagonal web reinforcement. Therefore, no serious problem is visualized if it is necessary to use concrete with high f'_c in constructing beams of moment-resisting frame systems.

Coupling beams in shear wall structural systems -- No serious problems are visualized in coupling beams of a system subjected to monotonically increasing deformations except for the development of somewhat higher axial forces than for beams of a moment-resisting frame. The presence of higher axial forces could have some detrimental effects on the flexural and shear strength if the axial force is a tensile one, and on the deformation capacity if it is a compressive one. However, these detrimental effects would usually be insignificant, and when significant could be reduced to acceptable values by changing the configuration of the coupled shear walls and by properly selecting the flexural strength of the coupling beams. For coupled shear-wall systems that can be subjected to generalized variable repeated loading, such as those that can be induced by severe seismic excitations, the inelastic behavior of the coupling beams can be more seriously affected. The deformation capacity, number of yielding excursions and number of inelastic rotation reversals demanded from these coupling beams under severe seismic excitations are usually all very large compared with those encountered in beams of ductile-moment-resisting frames of similar dynamic characteristics. Add to these adverse factors the fact that these coupling beams are often deep relative to their span, and therefore relatively large shear forces are generated, and it becomes clear that these forces can dominate the inelastic behavior of the beams. In these cases very little will be gained by using high-strength concrete. The ductility and useful strength of coupling beams can be improved by placing principal reinforcement diagonally in the beams instead of using the conventional steel arrangement (Fig. 22).

Columns

In several structural systems, and particularly in moment-resisting frames, there are columns which are subjected to very low axial forces; under lateral load the behavior of their critical regions is controlled by flexure. There is very little difference between the behavior of these columns and that of the beams, except that the columns are usually subjected to higher shear than the beams because they are shorter. Therefore, these columns can be classified with the beams under the general denomination of flexural members. This has been recognized by certain seismic codes like the ACI 318-71 and 318-77 (11) which specify that columns shall be designed and detailed in accordance with requirements for flexural members when the maximum factored axial load P_e is not greater than $0.4\phi\ P_b$, where ϕ is the strength reduction factor and P_b is the nominal axial load strength at balanced strain conditions.

The main application of concrete with high f'_c appears to be in columns subjected to high axial forces, such as those in tall buildings. Therefore, this section of the report emphasizes behavior of concrete columns in moment-resisting space frames which are ordinarily reinforced and whose critical regions are subjected to significant axial forces. In practice a space frame structure is usually modelled as a series of planar frames. Each of these planar frames are subjected independently to the vertical and horizontal forces acting in the plane of the frame. Therefore, at first discussion will be limited to the behavior of columns loaded in one of the principle axes (1D). Later, the importance of three-dimensional loading (3D) and particularly biaxial loading will be reviewed.

Columns in 1D R/C moment-resisting frames: ordinarily reinforced and subjected to high shear forces and significant axial forces -- Most of the data available on inelastic column behavior is from experiments carried out on columns designed to resist severe seismic excitations. There is very little data available on columns whose transverse reinforcement has been designed according to the code provisions for normal loadings. However, analysis of the data which is available and field inspections of the behavior of such columns subjected to earthquakes, show that their failure is relatively brittle, unless they are laterally reinforced by circular spiral.

There is some data available on the inelastic behavior of short R/C columns (14,51) that have been designed to satisfy current seismic code recommendations for ductile moment-resisting space frame (18). Analysis of this data reveals that such columns, when subjected to high constant axial loads, can develop good inelastic deformation without any significant loss in strength under monotonically increasing loading. When subjected to cyclic shear reversals inducing full deformation reversals these columns can develop moderate inelastic deformations prior to either a brittle shear failure or significant shear resistance degradation. The word moderate should be emphasized. These observations are illustrated in Fig. 23, which shows some of the results obtained by Zagajeski et al. (51) in their study of the hysteretic behavior of short R/C columns subjected to axial loads corresponding to the balanced point of the P-M diagram and with an f'_c that varies from 4900 to 5300 psi. In this figure, the imposed shear force, H, is plotted against story drift index R, or the tip displacement Δ. As can be seen from these plots, the values obtained for R are considerably higher than the acceptable values specified by seismic code recommendations (19). In this study, three different failure modes were observed: shear-compression, bond, and diagonal tension.

In the shear-compression and bond failure modes, the failure was gradual, resulting in significant shear and stiffness degradation. However, the column was still able to maintain the design gravity load for a lateral story drift ductility μ_R of four. In contrast to this gradual failure, the diagonal tension failure was sudden and the column was unable to maintain the design gravity load. All the columns resisted high nominal shear stresses v_u ($8.5\sqrt{f'_c}$ to $10.5\sqrt{f'_c}$ psi). Recent experiments have shown that the hysteretic behavior of short columns subjected to reversals of high shear can be significantly improved by the addition of diagonal reinforcement.

It is felt that the inelastic deformation capacities found in the investigations (particularly in Ref. 51) would prove adequate when compared to the magnitude and nature of inelastic deformation demands that may be expected for columns that are components of frame systems designed on the basis of weak girder-strong column philosophy. However, these deformation capacities may be insufficient when compared to the magnitude and nature of deformation demands that may be expected in frames designed with soft stories. Furthermore, the above observations are valid for cases where there is essentially no fluctuation in axial force. The change from a ductile shear-compression failure mode in columns with a certain axial compressive force to a brittle diagonal tension mode in similar columns in which the axial load decreased, suggests the need to investigate the inelastic behavior of short columns in which the axial force varies. The axial force should be varied with shear reversal from maximum compression to either a tension value or a smaller compression.

Comparison of the behavior of columns subjected to different deformation histories demonstrates that cyclic deformation reduces the maximum inelastic deformation a member can experience in a given direction. This fact should be kept in mind when design is controlled by inelastic deformation demands. It will be necessary to specify not only the deformation level that is expected, but also the number and type of reversals (partial, full). The magnitude of the nominal shear stresses developed in some of the columns tested show that moderate ductile behavior and high shear stresses are compatible. However, it is necessary to provide sufficient and properly detailed transverse reinforcement.

A comparison of the behavior of columns with different types of transverse reinforcement indicates that the circular spiral is more effective in maintaining a member's shear strength. Its continuity and relatively close spacing provide excellent confinement for the core concrete and restrain the width of inclined shear cracks. However, the close spacing of the spiral, and the fact that it is responsible for significant spalling through the height of the column, reduces the area of concrete in contact with the longitudinal reinforcement and thus contributed to bond deterioration along this reinforcement.

A question which needs to be answered is: what are the effects on the inelastic behavior of short R/C columns of using concrete with a high f'_c? If the main purpose for using high-strength is to reduce the size of the columns, then the following effects can be expected:

1. Greater loss in compression capacity when spalling occurs (see Fig. 6)

2. A decrease in deformation capacity (ductility) particularly if the column is subjected to a lateral displacement large enough to induce significant bending moment, because in this case it will be necessary to use a higher percentage of steel, ρ, and this leads to a decrease in ductility (see Eq. 15). This decrease can be counteracted by the increase in ductility due to the higher f'_c and because the $P_{max}/f'_c A_g$ ratio will be smaller for the column with high f'_c.

3. The nominal shear stress will be larger. The effect of this increase will be considerably more detrimental for columns subjected to reversal of deformations due to abnormal loading than for columns subjected only to montonically increasing loads. For the latter case, there is an increase in the contribution of the concrete to the resistance against shear because of the larger value of f'_c.

4. Decrease in stiffness, previously discussed.

5. Possible increase in the effect of creep, particularly when the columns are subjected to large fluctuations in axial forces.

It is essential that these effects be studied before high-strength concrete is used in columns of tall buildings located in regions of high seismic risk or other types of severe abnormal loading. It is believed that once the significance of these effects are evaluated, it will be possible to find satisfactory solutions by proper design. For example, the detrimental effects of inelastic behavior due to lateral displacement can be reduced by increasing the lateral stiffness through appropriate use of shear walls. However, it should be recognized that even if these shear walls are designed to resist all the lateral forces the column will undergo lateral displacement. Thus the columns should be provided with the corresponding required axial-flexural (P-M) and shear strength, and deformation capacity (ductility).

<u>Inelastic behavior of columns under three-dimensional loading</u> -- Building columns are usually subjected to three-dimensional (3D) loading components which will vary with time. Jirsa, et. al, have

prepared a thorough review of the analytical and experimental studies that have been conducted on the behavior of columns under 3D loading until 1978 (52). A summary of the present knowledge on this subject has been presented in Ref. 14 and new studies have been reported in the AICAP-CEB Symposium on Structural Concrete under Seismic Actions (53). Although some of the results obtained by different investigators do not agree, it appears that when bending and shear reversals are applied in the direction of the two main axes of a rectangular column there is a higher degree of stiffness deterioration than that observed under one directional lateral loading. While increase in compressive axial loads increase the shear capacity slightly, tensile axial loads substantially reduces the stiffness of the column and the shear resistance at low lateral loads. There is an urgent need for studying the behavior of columns subjected to 3D loading. Emphasis should be placed on the effects of high shear reversals and the fluctuations from high compression to high tension axial loads.

Beam-Column Joints

References 13 and 14 review what is currently known about the design and the elastic and inelastic behavior of beam-column joints, particularly when subjected to severe seismic excitations. Reference 15 summarizes the seismic-resistant design criteria for this type of joint and gives a series of recommendations to improve their hysteretic behavior. Following is a summary of the major problems in such joints, and how the use of high-strength concrete can affect their design and observed behavior.

Because a failure of the joint means a failure of the column, ideally the joint should be the strongest and the stiffest element of the basic subassemblage. In the past this usually has been so. Surveys of earthquake damage usually show no evidence of joint failure, except in cases of very poor detailing and construction. However, because of numerous failures in beams, and particularly in columns, recent seismic codes have much more stringent requirements regarding design and detailing of these two elements. Therefore, the writer believes that the joint may become the weakest link in the subassemblage. This belief has been corroborated by recent experimental results in laboratories and in the field. In many cases, although there is no visible sign of distress in the joint, it has failed internally with a loss of the required anchorage to the main reinforcing bars of the beams and/or columns.

Effect of loading history on the inelastic behavior of normal strength concrete beam-column joint: monotonic vs. cyclic loading -- Figure 24 compares research results conducted using different loading histories. It can be observed from this figure that repeated cycles of deformation reversals lead to a significant degradation in strength and stiffness. The possible sources of this observed degradation have been identified as high shear and/or high bond stress through the joint. (In the specimen of Fig. 24, the intensity of shear stresses were relatively small. Thus, the main source of the observed degradation was bond stress.) Although it is not possible to completely eliminate this problem, the degradation can be reduced by either: avoiding the formation of inelastic regions at the faces of the joint core; or by selecting wider columns and beams with a low percentage of steel, having a low yielding strength and strain hardening characteristics. If this cannot be done, proper detailing of the reinforcement of the beam, column, and joint can minimize the detrimental consequences of stiffness and strength degradation (54).

Investigations into the seismic hysteretic behavior of beams and beam-column subassemblages indicate that joints of R/C frames should not be considered rigid, as is usually assumed. Two possible sources of deformation that may develop at the joint must be included to accurately predict the actual hysteretic behavior of the frame, particularly when large displacement ductility demands are expected. These two sources of deformation are illustrated in Fig. 25 and will be identified as the shear distortion of the joint, γ_j, and the fixed-end rotation at the column face, θ_{FE}. Often the most important deformation is the one due to θ_{FE}. In contrast with the amount of research carried out to improve the design of beam-column joints for shear strength, very little has been done to improve methods of predicting stiffness, deformation capacity, and energy dissipation capacity of these joints. These mechanical characteristics are controlled by the θ_{FE}, which in turn depends on the bond-slippage characteristics of the beam bars along its embedment length at the joint.

Although excellent work has been done by several investigators on bond under generalized loading (55) to the best of the author's knowledge none of these investigations specifically addressed the problem of bond deterioration developed at the joint of an interior column. For a joint in an interior column, we are dealing with bond-slippage of steel bars embedded in well-confined reinforced concrete, which can still be adversely affected by bond degradation under cyclic loading. At Berkeley, there has been an investigation of the simplified problem of bond-slippage of bars embedded in well-confined reinforced concrete, which simulates the conditions of a beam-column joint in a plane frame loaded laterally in its plane (56-58). From the results of these experimental and analytical studies it has been concluded that:

(1) The assumption that beam-column joints of moment-resisting R/C frames are rigid needs to be re-examined. The main reinforcing bars of the beams do pull out, and thereby cause beams to experience fixed-end rotation. The consequences of this behavior on the overall structural response must be examined.

(2) In the joints, it is essential to distinguish the bond of unconfined concrete in the column cover from that of the confined core. The latter is appreciably better.

(3) Under monotonically increasing loads, when the beam main bar reaches yielding the accompanying pull-out can cause a fixed-end rotation in the order of 0.001 radians.

(4) The displacement of a bar due to monotonic loading at the column face can be estimated using simple idealizations of bond stress distribution (56). The dependence on concrete strength, type of lugs, embedment length, concrete confinement, etc. requires further investigation.

(5) Significant bond deterioration occurs from cyclically applied load reversals, particularly when the applied stresses exceed yield.

(6) It appears that bond resistance deterioration is gradually stabilized at the value of friction between two concrete cylindrical surfaces which have a common diameter equal to the outer dimension of the bar, including the lugs.

(7) More comprehensive analytical models are required for generalized loading of a bar. (A model has been developed by Viwathanatepa (58)).

(8) The implications of the effect of θ_{FE} on the behavior of structural systems should be studied analytically. (A computer program that permits inclusion of θ_{FE} in nonlinear analysis has been developed by Soleimani (59)).

Effects of using high-strength concrete in the inelastic behavior of beam-column joints -- The use of high-strength concrete is recommended for the design and construction of columns carrying high axial compression loads. This is due to the fact that the concrete area of the column cross sections can be reduced in direct proportion to the strength of the concrete. This reduction increases the detrimental effects of shear and bond stresses at the joint core because the shear and bond resistance of the concrete appears to increase only as a function of $\sqrt{f'_c}$. Thus, the increase in nominal unit shear and bond stresses seems to be higher than the increase in available resistance. The degradation problem is accentuated by cyclic reversals. This was confirmed by a series of experiements recently carried out in Berkeley (60) on the bond-slippage of bars embedded in well-confined concrete. Results from experiments using concrete with a $f'_c \simeq 4.5$ ksi and $f'_c \simeq 9$ ksi have been compared. Under monotonically increasing loading there is an increase in resistance against pullout of the bar when $f'_c = 9$ ksi is used, permitting a more than 25% percent reduction in the required embedment length. However, the improvement is not so great when the bar is subjected to cyclic loading with reversals.

It should be noted that due to the pullout of the bars part of the cover of unconfined concrete becomes loose, and considerably reduces the column concrete section available to resist the combined effect of P and M.

Similar observations have been obtained by increasing the compressive axial force acting in the columns, rather than using higher f'_c. Thus using high f'_c concrete when columns are subjected to high axial compressive load appears to be advantageous as far as the bond degradation under monotonically loading is concerned.

From the above discussion it can be concluded that when using high-strength concrete, more careful attention should be paid to the problems of degradation in strength and stiffness due to shear and particularly bond stresses at the joint. The problems created by the relatively higher shear and bond stresses can be solved in a way similar to how they were solved for normal strength concrete by avoiding the formation of inelastic regions at the faces of the joint core.

Perhaps an even better solution to the problems created when high f'_c is used is to try to avoid the development of high bond stresses and shear by controlling the amount of lateral displacement that the columns can undergo. In tall reinforced concrete building the lateral forces are normally resisted by shear walls. It is not enough to design the walls to resist lateral forces. The walls must also be stiff enough to control the sway of the columns so that their beam-column joints will not undergo inelastic behavior and therefore will not affect the required P-M strength capacity of the column.

The type of aggregate used might play an important role in determining the effects of concrete with high f'_c on the behavior of the beam-column joint. Results obtained using normal-strength concretes (Fig. 26) indicate that there will not be significant effect for monotonically increasing excitations but that under cyclic loading the use of lightweight aggregate can lead to considerably higher degradation in strength and stiffness than the use of good stone aggregate (61).

It is necessary to study how the strength, stiffness, and energy dissipation capacity of real beam-column joints (including slabs) can be affected by three-dimensional loading. Some experiments are presently being carried out at the University of Canterbury, New Zealand, and at the University of Texas, Austin, Texas (53).

Shear Walls

The use of high-strength concrete not only holds promise, but has already been used by the building industry as an economic way of constructing columns and shear walls of tall buildings in regions where only normal types of excitations will be induced during the service life of the buildings. Therefore, it is of interest to analyze the data available regarding the elastic and particularly the inelastic behavior of R/C shear walls and see how this behavior is affected when high-strength concrete is used. Safety against collapse under monotonically increasing loading is of particular importance for this type of wall. Also important is whether the use of high-strength concrete can be justified in constructing shear walls in regions of very high seismic risk where the inelastic behavior controls the design.

Shear-wall buildings subjected to normal loading -- The design of this type of building is controlled by the service loads, i.e. "elastic behavior". The safety against collapse due to overloading is generally larger than the load factors specified by the code. Although very little data exists regarding inelastic behavior of shear-walls designed against normal loading conditions according to present code requirements, it is believed that they can offer certain ductility. This ductility, combined with the overstrength inherent in these walls, offers a safety factor that can be considerably larger than the load factors specified by the code, particularly for coupled wall systems. There is an urgent need to carry out research on the inelastic behavior and redistribution of forces that occur in structural systems, based on the use of shear walls designed according to standard code requirements.

The types of failure possible in high-strength concrete walls are: (1) flexural yielding; (2) shear failure (diagonal tension or crushing); and (3) instability. The ideal is that failure be triggered by flexural yielding because it is the most ductile of these three types of failure. Because the use of high f_c' concrete can lead to thinner walls, the danger of having a shear failure, and particularly an instability before flexural yielding, appears to increase with the increase of f_c'.

Shear-wall buildings subjected to abnormal loading -- Only the situation of shear wall buildings designed against severe seismic excitations is reviewed herein. The state-of-the-art for the inelastic behavior of shear walls designed according to modern seismic code provisions has recently been reviewed (15) and can be summarized as follows. It should be noted that all literature reviewed deals only with shear walls designed and constructed using concrete with $f_c' < 6000$ psi.

In reinforced concrete shear walls of frame-wall dual structural systems designed according to 1973 UBC seismic design provisions, and whose proportioning and final detailing is based on flexural yielding before shear failure, large displacement ductility ratios, (μ_δ), can develop (Fig. 27). The critical regions of these walls possess large inelastic rotations (θ_p) even when subjected to full deformation reversals, inducing nominal unit shear stresses up to $13\sqrt{f_c'}$ (psi); i.e., $V = 13\sqrt{f_c'}\,hd$, which is greater than the maximum value of $10\sqrt{f_c'}\,hd$ presently allowed by the ACI-318-77 Code. The smaller the nominal unit shear stress acting at the critical region of the wall, the larger the μ_δ and the better the overall hysteretic behavior.

Framed walls (barbell cross-section) have better hysteretic behavior than rectangular cross-section walls. Interstory drift ductility ratios, μ_δ, larger than ten have been attained for barbell cross-section walls under monotonic loading. However, this large value should not be used for seismic-resistant design since its development can lead to instability of the wall under loading reversal (Fig. 27).

The main effects of cycling wall critical regions under full deformation reversals are (Fig. 27): (1) to reduce the μ_δ from ten to about four (which corresponds to a cyclic ductility ratio of about seven); and (2) to induce a pinching effect in the hysteretic loops leading to a reduction of energy dissipation. In spite of these reductions, the total amount of energy dissipation capacity is so high that it exceeds demands of even the largest unexpected earthquake. Furthermore, at the reduced $\mu_\delta = 4$, the confined core of the edge members of the barbell cross-section remained sound and capable of resisting the effects of the axial forces (imposed by the gravity loads) combined with the effect of lateral loads at the working-load level.

For rectangular cross-section walls, precautions should be taken to prevent premature failure due to instability. Present code and suggested dimensional limitations to avoid instability are not adequate when the required μ_δ is larger than three. These limitations should depend upon the required rotation capacity of the critical region of the wall.

Diagonal (45°) reinforcement of the wall panel results in better hysteretic behavior than that of similar walls reinforced with vertical and horizontal bars.

The use of these ductile walls coupled by very ductile girders can lead to a strong-column, weak-girder system which is superior to that which can be attained with columns and beams. The combination of these ductile coupled shear walls with moment-resisting space frames holds great promise for attaining sound seismic-resistant reinforced concrete structural systems.

The above observations were obtained from both experimental and analytical studies conducted on R/C walls in which the strength of the concrete did not exceed 6000 psi. It is interesting to speculate what the effects on the observed behavior might be when concrete with an f'_c higher than 6000 psi is used. A review of the code requirements for the design of shear walls in regions of very high seismic risk shows that these shear walls shall have vertical boundary elements proportioned to carry all vertical forces resulting from factored wall dead loads, factored tributary dead and live loads, and factored horizontal loads. Compliance with these requirements and other seismic code requirements lead to the following observations:

(1) The edge members of shear walls have to be designed for very high tensile and compressive axial forces. These compressive axial forces are considerably higher than those for which the columns of a ductile moment-resisting space frame - of a similar configuration to the structural system in which the walls are used - have to be designed. Thus, the use of high-strength concrete in the construction of these edge members seems logical from architectural, engineering, and economic considerations.

(2) The fact that the edge members of the shear wall can be subjected to high tensile forces also makes it attractive to use high-strength steel in combination with high-strength concrete.

Furthermore, considering that of the three possible modes in which shear walls constructed with $f'_c < 6$ ksi and $f_y < 70$ ksi can fail--flexural yielding, shear and instability--designers have to try to prevent the last two or to delay their occurrence until sufficient energy dissipation by flexural yielding has occurred, the following speculative observations can be made about the possible effects the use of high f'_c and high f_y can have in the inelastic behavior of walls.

(1) The use of high-strength concrete can lead to relatively thinner shear walls. The thickness of the edge members can be reduced most, thus leading to earlier and higher degradation of stiffness (pinching effect in the hysteretic loops, see Fig. 27) as well as instability under lower displacement ductility than in normal-strength concrete shear walls, particularly if the rectangular, rather than the barbell, cross section is used.

(2) The use of relatively thinner edge members can lead to their earlier failure by shear.

(3) The use of higher yielding reinforcing steel can lead to a decrease in displacement ductility.

The large the axial compression and the more effective the concrete confinement, the higher the shear resistance of the concrete. Therefore, it is believed that if it would be possible to get a sufficiently high confinement effectiveness coefficient, k, with high-strength concrete, say $k \geq 4$, for a sufficiently long range of inelastic strain, the problems under observations (1) and (2) can be overcome. It is a question of formulating more stringent requirements regarding the minimum thickness of the wall panel and edge members to avoid early instability. The problems caused by the use of higher yielding steel needs to be studied. Perhaps the solution is to avoid the use of steel with $f_y > 70$ ksi.

From the above discussion, it appears that if present code provisions for seismic-resistant design of R/C structures would also be applicable to structures where high-strength concrete is used (which has yet to be proven), the structural system that would benefit most from the use of high-strength concrete would be the one based on the use of the frame-shearwall system.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

General observations applying to all structural members are presented first. Then observations for beams, columns, beam-column joints, and shear walls are presented separately. Most of these observations concerning high f'_c concrete have been derived from speculative discussion because of the lack of data regarding the behavior of such concrete.

General observations -- (1) In order to properly design concrete structures, it is necessary to predict their "elastic" as well as their "inelastic" behavior up to collapse. This requires knowledge of the complete stress-strain relationship of the reinforced concrete material.

(2) No clear definition exists of what constitutes high-strength concrete. It appears that such a classification will depend on the type of aggregate that is used. Concretes with an f'_c of 9,000 psi have already been used in several buildings and 11000 psi concrete has been used in the construction of some columns. Tests on specimens with a concrete cylinder strength of near 15000 psi have also been conducted.

(3) Although there have been numerous articles dealing with the subject of high-strength concrete, very little data exists regarding the mechanical behavior of this type of concrete, and most of the existing data has been obtained from tests on plain concrete cylinders.

(4) In building construction, at present the main application of high f'_c concrete appears to be in the columns and shear walls of tall buildings which are subjected only to normal types of excitations.

(5) If it would be practically and economically feasible to confine high-strength lightweight concrete so that its stress-strain relationship would be close to that of the elastic-perfectly plastic behavior up to a strain of 5%, it would constitute a very desirable material for seismic-resistant design, when properly reinforced.

(6) Because of their relatively low modulus of elasticity, high-strength concretes, particularly lightweight, cannot be used effectively with low yield strength steel in columns subjected to very high service axial loads.

(7) The modulus of elasticity appears to be very sensitive to the type of aggregate used.

(8) The higher the f'_c the larger the strain, ε_o, at which this f'_c is reached.

(9) The higher the f'_c the larger the rate of decrease in strength after reaching f'_c.

(10) No data exists regarding the mechanical characteristics (the complete stress-strain relationship) of confined, longitudinally reinforced high f'_c concrete.

(11) For concrete with an f'_c up to 6000 psi, its confinement -- with all types of aggregate -- is effective in developing large deformability. The ultimate strain in all cases is greater than 2% and is usually up to 5%. However, increases in strength and deformation characteristics after reaching maximum strength are very sensitive to the type of aggregate and to the relative amount of confining pressure. The yielding strength of the confining reinforcement is a very important parameter: the higher the yielding strength, the more effective the confinement. Circular spiral gives better confinement than rectangular spiral, and rectangular hoops.

(12) In developing ACI criterion for spiral reinforcement, a constant value of the confinement effectiveness coefficient, k, equal to 4.0 or 4.1 is assumed. Experimental results show that for certain types of lightweight aggregates, the value of k is considerably smaller than 4.

(13) It is suspected that the higher the f'_c the more difficult it will be to get a confinement effectiveness coefficient equal to 4, particularly at ultimate strain.

(14) The low confinement effectiveness in some concretes may lead to significant losses in compression capacity when spalling of the unconfined concrete occurs in R/C elements whose confinement is designed according to present ACI code provisions (Fig. 6). Because use of high f'_c concrete leads to a reduction of A_c/A_g, the losses when the cover spalls can be higher for elements constructed with high f'_c than normal f'_c concrete.

(15) To improve the confinement effectiveness, it appears desirable to use the best aggregate available, and high yielding strength lateral reinforcement with no plastic plateau.

(16) The P-M strength interaction curve computed on the basis of the stress-strain relationship assumed by ACI code is unconservative when stable strength at large inelastic deformations is required, particularly for axial loads higher than those corresponding to the balanced conditions. Only when high confinement pressures (high volumetric ratio of lateral reinforcement with high yielding strength) are provided, and large ultimate strains of the confined concrete are achieved, will the P-M strength computed on the basis of ACI assumptions be recovered once the cover spalls.

(17) The use of high f'_c concrete can lead to members with relatively greater slenderness and smaller stiffness, thus increasing the severity of the stability problems.

(18) Problems created by volumetric changes of high f'_c concrete require scrutiny, particularly if it is used for seismic resistant columns where detrimental tensile cracking can occur even if the whole member is not under a net tension.

(19) To attain a certain lateral displacement ductility ratio for a whole structure usually requires that the critical regions of its structural elements be capable of developing considerably higher rotation or average curvature ductility ratio.

(20) If all the variables influencing the curvature ductility of doubly R/C sections, except the f'_c, are held constant, an increase in f'_c increases the curvature ductility, μ_ϕ.

(21) For unconfined concrete, the amount that μ_ϕ can be increased is limited by the low value of its maximum strain, $\varepsilon_{c\ max}$. Thus, if large values of μ_ϕ are required, it is necessary to increase $\varepsilon_{c\ max}$ by confining the concrete.

(22) To take advantage of the largest $\varepsilon_{c\ max}$ that can be developed by properly confining the concrete, it will be necessary to prevent earlier buckling of the compression steel which might require closer spacing of the lateral reinforcement than that required by the confinement of the concrete.

Observations for Beams

(1) While beams of moment-resisting frames are usually subjected to very low axial forces, coupling beams of coupled-shear walls can be subjected to significant axial forces.

(2) In coupling beams of coupled-shear walls that can be subjected to severe seismic excitations, the demands in deformation capacity, number of yielding excursions, and number of plastic rotations are considerably higher than similar demands on beams of ductile moment-resisting frames.

(3) The increase of f'_c made little difference to the flexural strength of underreinforced beams.

(4) The inelastic behavior of beam critical regions is very sensitive to the type of excitation, particularly when they can be subjected to high shears. If the beams are subjected to only monotonically increasing curvature, the effects of shear are not of significance if the nominal units shear stress, v_{max}, is kept below $10\sqrt{f'_c}$ (psi). On the other hand, if the beam is subjected to cyclic loads inducing full reversal deformations, it will undergo considerable strength and particularly stiffness degradations if v_{max} exceeds $3\sqrt{f'_c}$ (psi). The addition of diagonal web reinforcement in the flexural critical regions having $v_{max} > 3\sqrt{f'_c}$ (psi) is an effective means of improving its hysteretic behavior. Beams with $v_{max} > 6\sqrt{f'_c}$ (psi) should be avoided if they can be subjected to several cycles with full deformation reversals at large ductility.

(5) In case of underreinforced beams of moment-resisting frames whose inelastic deformation is controlled by monotonically increasing types of excitations, there is little to gain by using concrete with high f'_c. Only when it is combined with the use of high yielding strength steel could it bring significant increase in flexural strength, but this combination can lead to a decrease in ductility and higher values of shear forces and nominal unit shear stresses which can also have detrimental effects.

(6) In case of beams of ductile moment-resisting frames whose inelastic behavior is controlled by generalized variable repeated loading inducing reversals of deformations, the use of high f'_c concrete might be detrimental when significant shear forces can be induced in their critical regions and the design against this shear is done according to present ACI code provisions.

(7) There is little to gain by using high f'_c concrete in coupling beams of coupled shear-walls particularly when these are deep relative to their span.

Observations for Columns

(1) Short columns of nominal f'_c concrete designed to satisfy current seismic recommendations for ductile moment-resisting space frame and when subjected to constant axial loads can develop good inelastic deformation without any significant loss in strength when subjected to monotonically increasing lateral displacment. When subjected to cyclic shear inducing full deformation reversals these columns can develop moderate energy dissipation capacity. This type of column has failed in three main modes: shear-compression; bond; and diagonal tension.

(2) Shear-compression and bond failure modes are gradual with significant stiffness and shear resistance degradation. However, these columns are able to maintain their high design gravity load for a lateral story drift ductility of four.

(3) It is more effective to use lateral reinforcement in the form of circular spiral than to use rectangular hoops in maintaining the shear resistance of columns under cyclic loading, inducing reversals of large inelastic lateral displacement. However, the spiral leads to an earlier and more pronounced bond deterioration along the longitudinal reinforcement.

(4) Columns forming part of a structural system which can be subjected to lateral story drift should be designed against the effects of the combined forces and moments that can be induced by such drift, even if other components have been designed to resist all lateral forces. This is of utmost importance if the drift of the whole structural system could be large enough to produce spalling of the column cover.

(5) Three-dimensional loading results in a higher degree of stiffness deterioration than that observed under one-directional lateral loading.

(6) The use of high f'_c concrete should lead to (1) greater loss in compression capacity when spalling occurs; (2) a decrease in lateral deformation capacity when the column is subjected to relatively large bending moment; (3) development of higher nominal shear stress which will be detrimental for columns subjected to reversals of lateral displacement; (4) decrease in stiffness; and (5) increase in creep effects on columns subjected to large fluctuations in axial forces.

Observations for Beam-Column Joints

(1) Inelastic behavior of beam-column joints is very sensitive to loading histories. Cyclic loading inducing deformation reversals lead to significant degradation in stiffness and strength. Sources of these degradations are the shear and bond stresses acting through the joint.

(2) Sound design requires that the joint should be the strongest and stiffest element of the basic subassemblage of a moment-resisting frame. To achieve this it is convenient to design the elements of the basic subassemblages in such a way that the joint will remain elastic even during large inelastic deformations of the whole frame.

(3) The assumption that beam-column joints of moment-resisting R/C frames are rigid needs to be re-examined. Even under monotonically increasing loads at yielding of the beam bars the resulting pullout of these bars from the joint can cause a fixed-end rotation in the order of 0.001 radians. Significant bond deterioration occurs from cyclically applied load reversals.

(4) With the other variables held constant, the higher the compression load acting in a column and the higher the concrete strength, the smaller is the pullout of the main bars.

(5) When high f'_c concrete is used in columns to reduce its size, this should lead to an increase in the detrimental effects of shear and bond stresses at the joint when it will be subjected to cyclic loading with reversals of deformation.

(6) When high f'_c concrete is used in columns, the problems created by the relatively higher shear and bond stresses can be solved as they were solved for normal f'_c concrete, i.e. by avoiding formation of inelastic regions at the faces of the joint core. Another solution is to avoid the development of such high stresses by controlling the amount of lateral displacement that the columns (and therefore the beam-column joints) can undergo, so inelastic behavior will not be developed at the joint.

(7) Type of aggregate used in the concrete with high f'_c might significantly affect the inelastic behavior of beam-column joint. Stone aggregate should give better results than commercially available lightweight aggregates.

Observations for Shear Walls

(1) Very little data exists regarding the inelastic behavior of shear walls for tall buildings that have been designed against normal excitations according to present code provisions. To have ductile failure, this failure should be triggered after some flexural yielding has been developed. Because the use of high f'_c concrete can lead to thinner walls, the danger of having a shear failure, and particularly an instability, before flexural yielding appears to increase with the increase of f'_c.

(2) Framed shear walls of frame-wall dual structural systems in which normal f'_c concrete is used and which are designed according to current seismic code provisions, can develop large lateral displacement ductility ($\mu_\delta = 4$) even when they are subjected to cyclic loading inducing full displacement reversals and nominal shear stresses up to $13\sqrt{f'_c}$(psi).

(3) The use of high f'_c concrete in the edge members of shear walls appears to be even more attractive than its use in columns of moment-resisting frames.

(4) When shear walls are subjected to cyclic loading inducing reversals of deformations, the use of high f'_c concrete can lead to earlier and higher degradation of stiffness, as well as earlier instability than the use of normal f'_c concrete.

Recommendations for Future Research and Developments

Among the recommendations formulated in this report, the following deserve special mention.

(1) Obtain reliable information about the mechanical characteristics of plain, high f'_c concrete and how these characteristics can be affected by the properties of its constituents.

(2) Improve the mechanical characteristics of lightweight aggregates presently available or develop new types.

(3) Determine the mechanical characteristics of confined high f'_c concrete and to formulate reliable constitutive models for such concrete.

(4) Investigate the validity of present ACI code methods of determining the P-M and shear strength for reinforced high f'_c concrete members.

(5) Determine the behavior of stress transfer between steel, and unconfined and confined high f'_c concrete under different types of excitations.

(6) Improve knowledge about mechanical behavior of the different basic structural elements made of high f'_c concrete when subjected simultaneously to axial forces and unidirectional as well as two-directional lateral loadings, following different possible time histories.

Concluding Remarks

From analysis of the speculative discussions and the conclusions that have been drawn regarding the structural use of high f'_c concrete, it would appear that such use will increase the severity of problems that have already been encountered with the use of normal f'_c concrete in structures that can undergo significant inelastic behavior during their service lives. However, the writer believes that even if future research confirms the increased severity of such problems, it will be possible to eliminate or minimize these problems by proper design and detailing, as has been done with the use of normal f'_c concrete. It is hoped that this report can serve as a basis for spirited discussions which will contribute to a better understanding and possible solution of these problems, so high f'_c concrete can be used efficiently in the future design of R/C structures.

ACKNOWLEDGMENTS

Appreciation is expressed to the National Science Foundation for its financial support of some of the studies reported herein. Thanks are also due to M.C. Randall and D. Ullman for their assistance in the preparation of this paper.

REFERENCES

1. CEB, Commission XI, Hyperstatique, Report on the Monaco Session, Bulletin d'Information CEB, No. 34, October, 1961, pp. 32-73.

2. ICE Research Committee, Report on Ultimate Load Design of Concrete Structures, Proceedings, ICE, (London), Vol. 21, February, October, Novemeber, 1962, Vol. 22, June, 1963.

3. Sawyer, H.A., "Status and Potentialities of Nonlinear Design of Concrete Frames," in "Flexural Mechanics of Reinforced Concrete," Proceedings of the Int'l Symposium, Miami, Fla., 1964, ACI SP-12, 1965.

4. Winter, G., and Cohn, M.Z., Contributions to the Int'l Symposium on "Flexural Mechanics of Reinforced Concrete," Miami, Florida, November, 1964, November, 1964, ACI, SP-12, pp. 581-590, 591-594.

5. Leonhardt, F., Levi, F., and Rüsch, H., Contributions to Bulletin d'Information CEB, No. 55, May, 1966, pp. 60-71, 131-153 and 154-158.

6. Bertero, V.V., "Research Needs in Limit Design of Reinforced Concrete Structures," Report No. EERC-71/4, Earthquake Engineering Research Center, University of California, Berkeley, June, 1971.

7. Bertero, V.V., "Identification of Research Needs for Improving Aseismic Design of Building Structures," Report No. EERC-75/27, Earthquake Engineering Research Center, University of California, Berkeley, September, 1975.

8. Sawyer, H.A., "Comprehensive Design of Reinforced Concrete Frames by Plasticity Factors," 9th Planning Session of CEB Symposium, Hyperstatique, Ankara, September, 1964.

9. Cohn, M.Z., and Parimi, S.R., "Multi-Criteria Probalistic Design," Journal of Structural Mechanics, No. 1, 1973, pp. 479-496.

10. Zagajeski, S.W., and Bertero, V.V., "Selection of an Optimum Moment Redistribution in Seismic-Resistant Design of R/C Ductile Moment-Resisting Frame," Proceedings of the Third Canadian Conference on Earthquake Engineering, Montreal, Canada, 1979, Vol. 2, pp. 1039-1062.

11. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-77)," American Concrete Institute, Detroit, 1977.

12. Bertero, V.V., "Strength and Deformation Capacities of Buildings Under Extreme Environments," Structural Engineering and Structural Mechanics, Proceedings of the Popov Symposium, Prentice Hall, 1980.

13. Bertero, V.V., (Organizer), "Earthquake-Resistant Reinforced Concrete Building Construction," Proceedings, Workshop, University of California, Berkeley, 1978.

14. Bertero, V.V., "Seismic Behavior of Structural Concrete Linear Elements (Beams, Columns) and their Connections," Proceedings of the A.I.C.A.P.-C.E.B Symposium on "Structural Concrete Under Seismic Actions," Rome, May 25-28, 1979. CEB Bulletin d'Information, No. 131, April, 1979, pp. 123-212.

15. Bertero, V.V., "An Overview of the State-of-the-Art in Earthquake Resistant Reinforced Concrete Building Construction," Proceedings of the Second U.S. National Conference on Earthquake Engineering. Stanford University, Stanford, Calif., 1979, pp. 838-852.

16. Chicago Committee on High-Rise Buildings, "High-Strength Concrete in Chicago High-Rise Buildings," Task Force, Report No. 5, Chicago, Illinois, February, 1977.

17. Leslie, K.E., Ratajapalan, K.S., and Everard, N.J., "Flexural Behavior of High Strength Concrete Beams," American Concrete Institute Journal Proceedings, Vol. 73, No. 8, September, 1976, pp. 517-521.

18. International Conference of Building Officials, "Uniform Building Code," 1973 and 1976 Editions, Washington, D.C., and Whittier, California.

19. Applied Technology Council, "Tentative Provisions for the Development of Seismic Regulations for Buildings," Prepared by Applied Technology Council, National Bureau of Standards, SP-510, Washington, June, 1978.

20. Mather, B., "Stronger Concrete," Report prepared for Highway Research Board, Waterways Experimental Station, Jackson, Mississippi, November, 1966.

21. Carrasquillo, R.L., Nilson, A.H., and Slate, F.O., "Very High-Strength Concrete: An Annoted Bibliography 1930-1976," Report No. 367 of the Department of Structural Engineering, Cornell University, Ithaca, New York, April, 1977.

22. Hollister, S.C., "Urgent Need for Research in High Strength Concrete," American Concrete Institute Journal, Vol. 73, No. 3, March, 1976, pp. 136-137.

23. Council on Tall Buildings and Urban Habitat, "Monograph on the Planning and Design of Tall Buildings," Five Volumes, American Society of Civil Engineers, New York, 1978.

24. Bertero, V.V., "Seismic Performance of Reinforced Concrete Structures," Argentina Academy of Science, November, 1978.

25. Nilson, A.H., "Design of Prestressed Concrete," J. Wiley and Sons, New York, 1978.

26. Kaar, P.H., Hanson, N.W., and Capell, H.T., "Stress-Strain Characteristics of High Strength Concrete," Douglas McHenry Int'l Symposium on Concrete and Concrete Structures, SP-55, American Concrete Institute, Detroit, 1978, pp. 161-185.

27. Wang, P.T., Shah, S.P., and Naaman, A.E., "Stress-Strain Curves of Normal and Lightweight Concrete in Compression," ACI Journal November, 1978, pp. 603-611. Discussion by Brock Geoffrey and Authors' Closure in ACI Journal, May, 1979, pp. 668-671.

28. Bresler, B., and Bertero, V.V., "Influence of High Strain Rate and Cyclic Loading Behavior of Unconfined and Confined Concrete in Compression," Proceedings, Second Canadian Conference on Earthquake Engineering, Hamilton, Ontario, Canada, June 5-6, 1975.

29. Bertero, V.V., and Vallenas, J., "Confined Concrete: Research and Development Needs," Proceedings, Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11-15, 1977, Vol. II, pp. 594-610.

30. Vallenas, J., Bertero, V.V., and Popov, E.P., "Concrete Confined by Rectangular Hoops and Subjected to Axial Loads," Report No. UCB/EERC-77/13, Earthquake Engineering Research Center, University of California, Berkeley, 1977.

31. Manrique, M., Bertero, V.V., and Popov, E.P., "Mechanical Behavior of Lightweight Concrete Confined by Different Types of Lateral Reinforcement," Report No. UCB/EERC-79/05, Earthquake Engineering Research Center, University of California, Berkeley.

32. Bertero, V.V., and Bresler, B.B., "Effect of Amount of Lateral Confining Pressure on the Strength and Deformation Characteristics of Normal and Lightweight Concrete," In-House Report, Department of Civil Engineering, University of California, Berkeley.

33. Shah, S.P., Naaman, A.E., and Moreno, J., "Effect of Compressive Strength and Confinement on the Ductility of Lightweight Concrete," Paper presented at the Symposium on "Nonlinear Behavior of Reinforced and Prestressed Concrete Structures," held at the ACI Convention at Houston, November, 1978.

34. Muguruma, H., et al., "Study of Improving the Flexural and Shear Deformation Capacity of Concrete Members by Using Lateral Confining Reinforcement with High Yield Strenth," CEB Bulletin d'Information, No. 132, April, 1979. AICAP-CEB Symposium, Rome, May 1979, pp. 37-44.

35. Mahin, S.A., and Bertero, V.V., "RCCOLA: A Computer Program for Reinforced Concrete Column Analysis," User's Manual, Department of Civil Engineering, University of California, Berkeley, August, 1977.

36. Mahin, S.A., and Bertero, V.V., "Problems in Establishing and Predicting Ductility in Seismic Design," Proceedings of the Int'l Symposium on Earthquake Structural Engineering, St. Louis, Missouri, U.S.A., August, 1976.

37. Mahin, S.A., "On the Use of Computers in Seismic Resistant Design of Reinforced Concrete Buildings," Proceedings, Workshop on Earthquake Resistant R/C building Construction, University of California, Berkeley, California, July, 1977.

38. Powell, G.H., "Computer Programs for Analysis of Seismic Response of R/C Buildings," Proceedings of the Workshop on Earthquake Resistant R/C Building Construction, University of California, Berkeley, California, July, 1977.

39. Clough, R.E., and Benuska, K.L., "FHA Study of Seismic Design Criteria for High Rise Buildings," Report HUD TS-3, Federal Housing Administration, Washington, D.C.,August, 1966.

40. Park, R., and Paulay, T., "Reinforced Concrete Structures," J. Wiley and Sons, 1975.

41. Bertero, V.V., and Felippa, C., "A Discussion on Ductility of Concrete," Proceedings of the International Symposium on "Flexural Mechanics of Reinforced Concrete," Miami, Florida, November, 1964, ACI Publication SP-12, pp. 227-234.

42. Wang, P.T., Shah, S.P., and Naaman, A.E., "High Strength Concrete in Ultimate Strength Design," Proceedings of the American Society of Civil Engineers, Vol. 104, No. ST.11, November, 1978, pp. 1761-1773.

43. Bertero, V.V., Bresler, B., and Liao, H., "Stiffness Degradation of Reinforced Concrete Members Subjected to Cyclic Flexural Moments," Earthquake Engineering Research Center, Report No. EERC-69/12, December, 1969, University of California, Berkeley.

44. Aoyama, H., "Restoring Force Characteristics Under Reversals of Loading of Reinforced Concrete Members and Structures," A Review of Japanese Research - Report of the United States-Japan Seminar on "Basic Research in Concrete as Related to Behavior of Structures in Earthquakes," Tokyo, February, 1967, 14 pages.

45. Funahashi, I., Kinashita, K., and Aoyama, H., "Vibration Tests and Test to Failure of a Seven-Story Building that Survived a Severe Earthquake," Proceedings of the Fourth World Conference on Earthquake Engineering, Chile, 1969, Volume I, pp. B-1, 26-42.

46. Mayes, R., and Galambos, T., "Large Scale Dynamic Shaking of an Eleven-Story Concrete Building," Proceedings of the Workshop on Earthquake Resistant R/C Building Construction, University Extension, University of California, Berkeley, 1977.

47. Freeman, S.A., Honda, K.K., and Blume, J., "Dynamic Response Investigations of Real Buildings," Proceedings of the Workshop on Earthquake Resistant R/C Building Construction, University Extension, University of California, Berkeley, California, July, 1977.

48. Ma, S.Y.M., Bertero, V.V., and Popov, E.P., "Experimental and Analytical Studies on the Hysteretic Behavior of Reinforced Concrete Rectangular and T-Beams," Earthquake Engineering Research Center, Report No. EERC-76/2, University of California, Berkeley.

49. Gergeley, P., "Experimental and Analytical Investigations of Reinforced Concrete Frames Subjected to Earthquake Loading," Proceedings, Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11-15, 1977, Vol. III, pp. 1175-1195.

50. Bertero, V.V., and Popov, E.P., "Hysteretic Behavior of R/C Flexural Members with Special Web Reinforcement," Proceedings, The U.S. National Conference on Earthquake Engineering, June, 1975, Ann Arbor, Michigan, pp. 316-326.

51. Zagajeski, S.W., Bertero, V.V., and Bouwkamp, J.G., "Hysteretic Behavior of Reinforced Concrete Columns Subjected to High Axial and Cyclic Shear Forces," Report No. UCB/EERC-78/05, Earthquake Engineering Research Center, University of California, Berkeley, 1978.

52. Jirsa, J.O., Maruyama, K., and Ramirez, H., "Reinforced Concrete Columns Under 3D Loading Histories," Paper presented at the Symposium of "ACI Convention at Houston," November, 1978.

53. Comité Europe-International du Béton, "Bulletin d'Information," No. 132, April, 1979, Paris, France.

54. Galunic, B., Bertero, V.V., and Popov, E.P., "An Approach for Improving Seismic-Resistant Behavior of Reinforced Concrete Interior Joints," Report No. UCB/EERC-77/30, Earthquake Engineering Research Center, University of California, Berkeley, 1977, 94 pages.

55. Popov, E.P., "Mechancial Characteristics and Bond of Reinforcing Steel under Seismic Conditions," Proceedings, Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11-15, 1977, Vol. II, pp. 658-682.

56. Bertero, V.V., Popov, E.P., and Viwathanatepa, S., "Bond of Reinforcing Steel: Experiments and a Mechanical Model," Proceedings, IASS Symposium on Nonlinear Behavior of Reinforced Concrete Spatial Structures, Darmstadt, Germany, July 3-7, 1978, Vol. 2, pp. 3-17.

57. Popov, E.P., Bertero, V.V., Cowell, A.D., Viwathanatepa, S., "Reinforcing Steel Bond under Monotonic and Cyclic Loadings," Paper presented at SEAOC Conference, Lake Tahoe, September 29, 1978.

58. Viwathanatepa, S., "Bond Deterioration of Reinforcing Bars Embedded in Confined Concrete Blocks," Ph.D. Thesis, Department of Civil Engineering, University of California, Berkeley, December, 1978, 454 pages.

59. Soleimani, D., "Reinforced Concrete Ductile Frames with Stiffness Degradation," Ph.D. Thesis, Division of Structural Engineering, University of California, Berkeley, California, December, 1978.

60. Cowell, A.D., "Reinforcing Steel Bond Under Monotonic and Cyclic Loadings," Masters of Engineering Thesis, Department of Civil Engineering, University of California, Berkeley, (In preparation).

61. Forzani, B., Popov, E.P., and Bertero, V.V., "Hysteretic Behavior of Lightweight Reinforced Concrete Beam-Column Subassemblages," Earthquake Engineering Research Center Report No. UCB/EERC-79/01, University of California, Berkeley, January, 1979.

TABLE 1 CONCRETE COMPRESSIVE STRENGTH, f_c, AND COMPRESSIVE MODULUS, E_c (28)

Concrete Type	Compress. Strength ksi		Observed E_c
	Maximum (f_c)	Design (f'_c) (1)	10^3 ksi
E-5	5.62	4.92+	3.07
B-3	3.52	2.97*	1.81
B-5	5.27	4.57+	2.08
R-3	3.73	3.18*	1.47
R-5	5.57	4.87+	1.75

(1) Reduced for estimated scatter in strength values in accordance with ACI 318-71, Sec. 4.2.2.1. *(f'_c) = [(f_c) - 0.55] ksi
+(f'_c) = [(f_c) - 0.70] ksi

TABLE 2 EFFECT OF CONFINEMENT ON COMPRESSIVE STRENGTH AND DEFORMATION OF CONCRETE (28)

Type of Concrete	Confinement Stress Ratio	Maximum Compression		Ultimate Compression	
		Strain Ratio	Confinement Effectiveness	Strain Ratio	Confinement Effectiveness
	(f_r/f'_c)	($\varepsilon_o^*/\varepsilon_o$)	k_o	($\varepsilon_u^*/\varepsilon_o$)	k_u
<u>Normal</u>	0.13	2.8	7.0	11.5	0
E-5	0.32	7.8	5.0	11.5	3.1
<u>Lightweight</u>	0.13	1.9	4.4	8.7	-0.5
R-5	0.32	4.0	2.0	6.7	2.0
B-5	0.13	1.35	3.9	10.6	0
	0.32	1.85	1.0	8.6	0.9
R-3	0.11	1.8	2.7	8.9	-1.0
	0.24	5.9	2.5	8.9	2.0
B-3	0.11	1.7	1.35	11.6	0
	0.24	8.0	2.1	9.0	2.1

LIST OF FIGURES

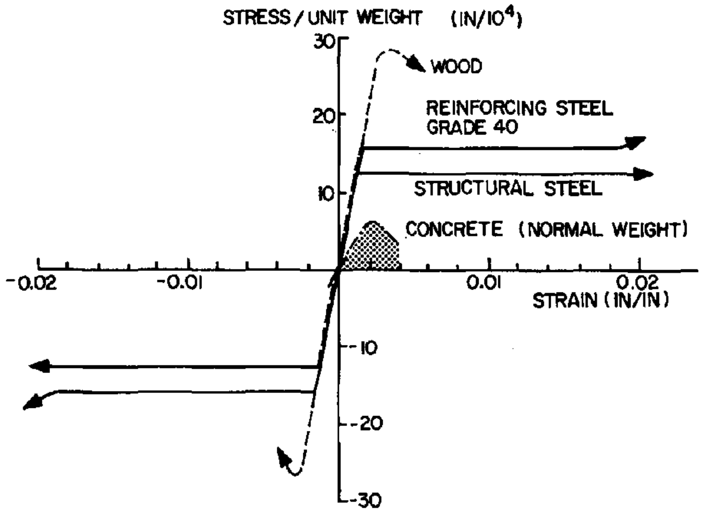


FIG. 1 STRESS PER UNIT WEIGHT - STRAIN DIAGRAMS FOR DIFFERENT STRUCTURAL MATERIALS

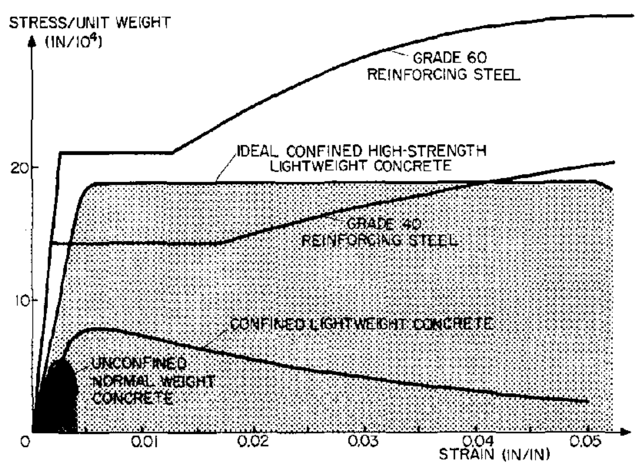
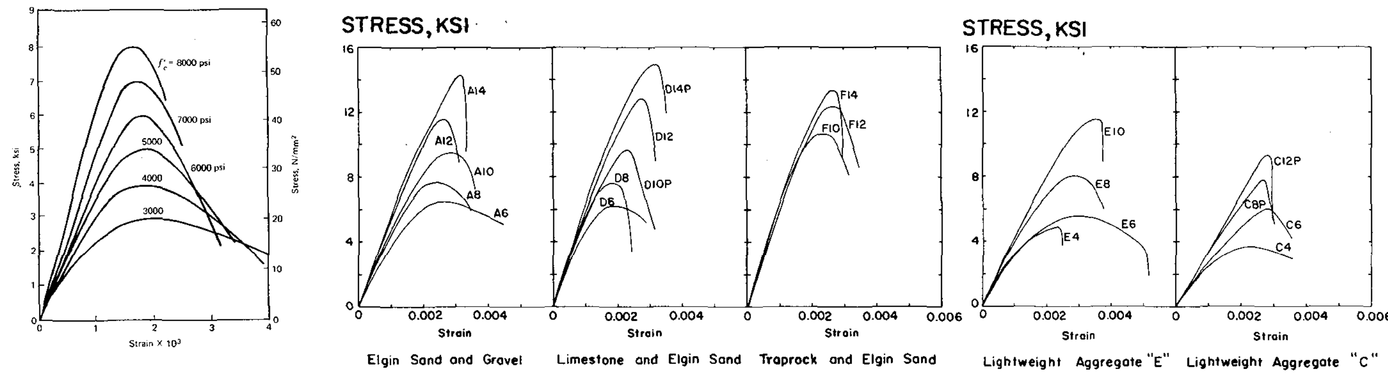
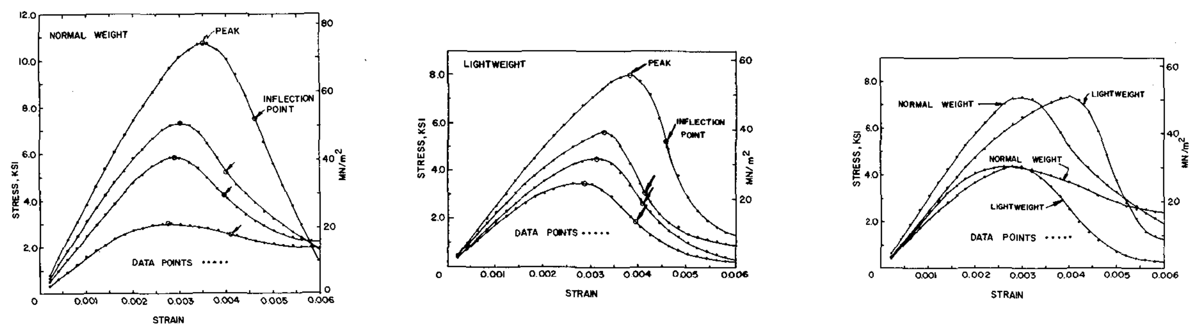


FIG. 2 STRESS PER UNIT WEIGHT - STRAIN DIAGRAMS FOR DIFFERENT CONCRETES AND REINFORCING STEELS



(a) Typical Curves (25)   (b) Curves for High-Strength Concretes Investigated by Kaar et al. (26)



(c) Typical Curves Obtained by Wang et al. (27)

FIG. 3 COMPARISON OF STRESS-STRAIN CURVES FOR UNCONFINED CONCRETE OF DIFFERENT STRENGTHS AND AGGREGATES

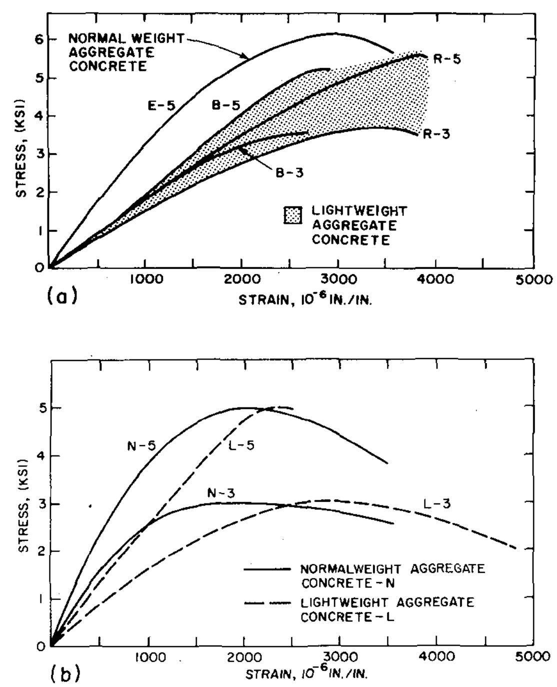


FIG. 4 COMPARISON OF STRESS-STRAIN DIAGRAMS FOR CONCRETES OF DIFFERENT STRENGTHS AND AGGREGATES, USED IN THE BERKELEY STUDIES (28)

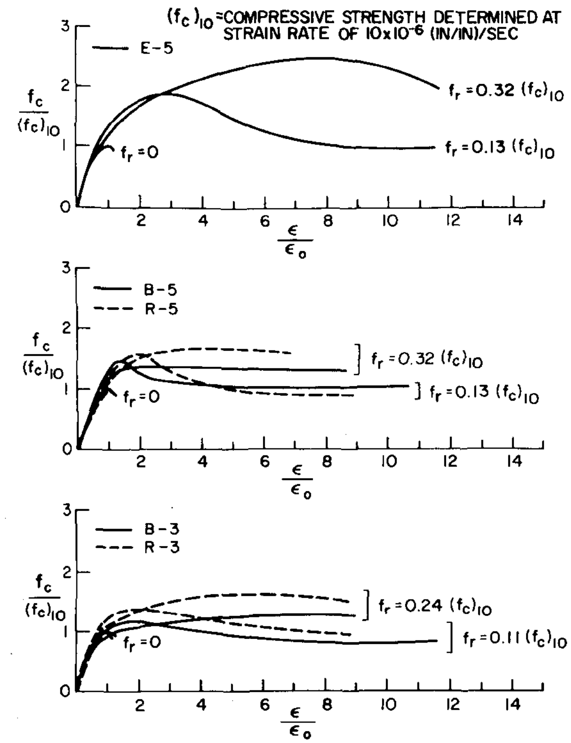


FIG. 5 EFFECT OF CONFINEMENT PRESSURE, f_r, ON THE STRESS-STRAIN DIAGRAMS OF CONFINED CONCRETE (28)

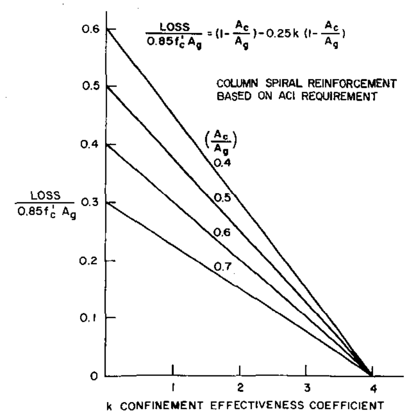
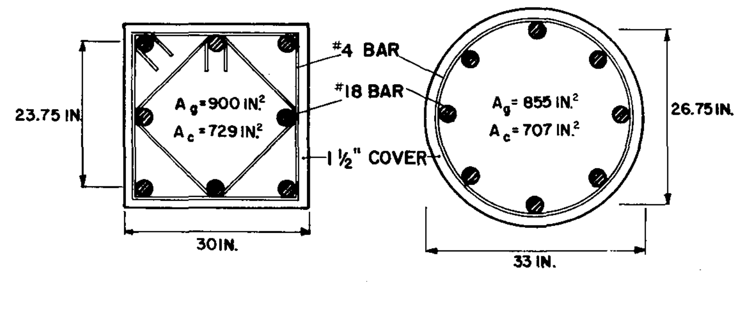


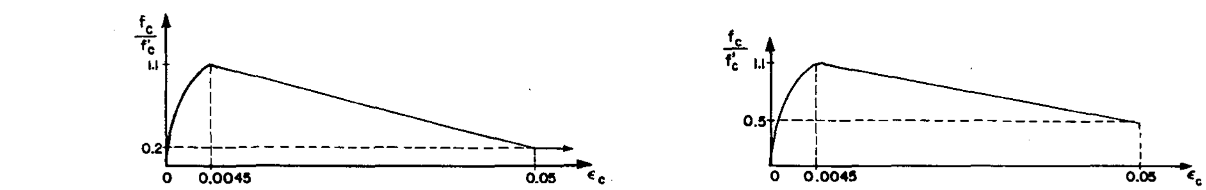
FIG. 6 LOSS OF COMPRESSIVE STRENGTH DUE TO SPALLING OF CONCRETE COVER VS. CONFINEMENT EFFECTIVENESS COEFFICIENT(28)



(a) Square

(b) Circular

FIG. 7 DIMENSIONS OF CROSS SECTIONS USED IN COMPUTING P-M STRENGTH INTERACTION DIAGRAMS (31)



(a) Circular Cross Section

(b) Square Cross Section

FIG. 8 STRESS-STRAIN RELATIONSHIPS OF CONFINED CONCRETE USED IN COMPUTING P-M STRENGTH INTERACTION DIAGRAMS (31)

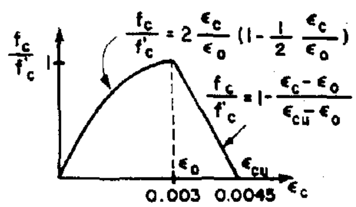


FIG. 9(a) STRESS-STRAIN RELATIONSHIP FOR CONCRETE COVER

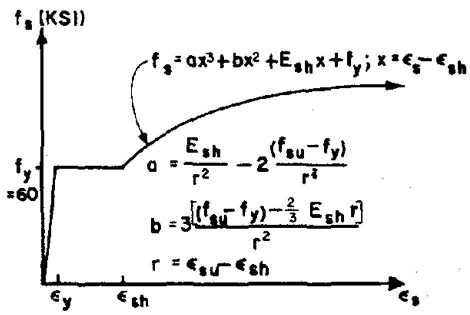


FIG. 9(b) STRESS-STRAIN RELATIONSHIP FOR LONGITUDINAL REINFORCEMENT

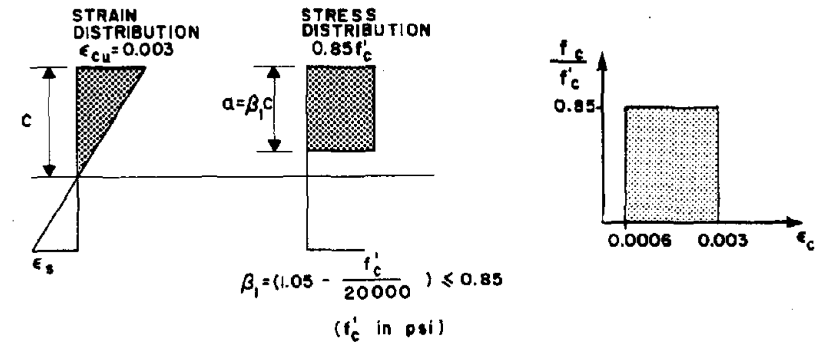


FIG. 10(a) EQUIVALENT STRESS BLOCK ASSUMPTIONS AND IMPLIED STRESS-STRAIN RELATIONSHIP FOR CONCRETE

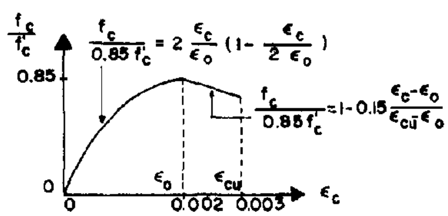


FIG. 10(b) HOGNESTAD STRESS-STRAIN RELATIONSHIP FOR CONCRETE

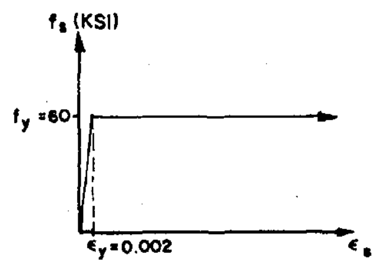


FIG. 10(c) ELASTIC-PERFECTLY PLASTIC RELATIONSHIP FOR STEEL

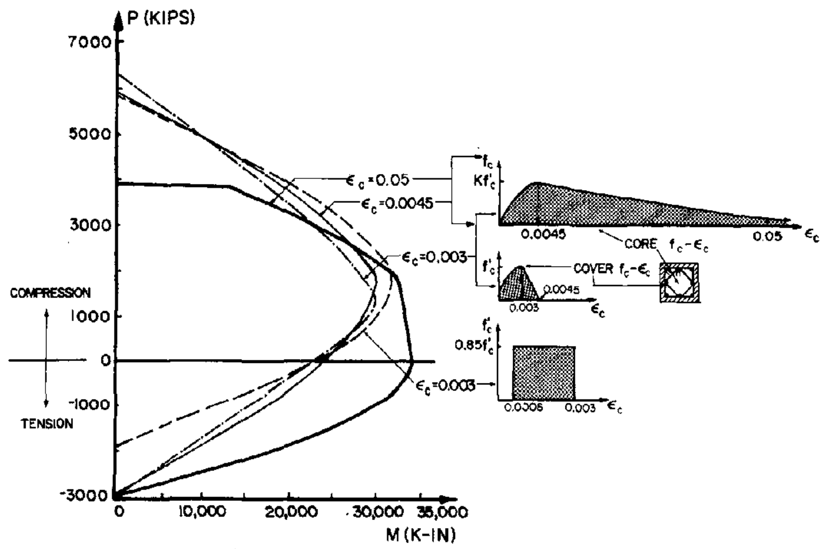


FIG. 11 COMPARISON OF P-M STRENGTH INTERACTION DIAGRAMS FOR THE SQUARE CROSS SECTION COLUMN OF FIG. 7 (31)

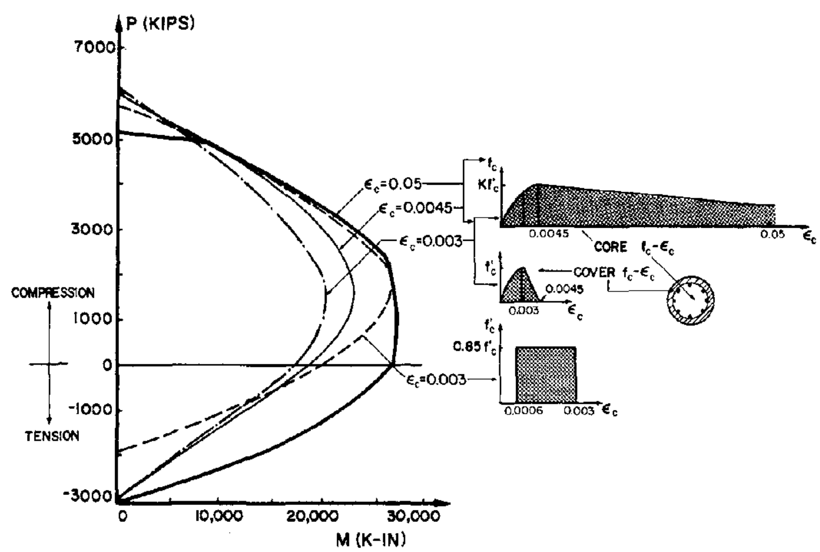


FIG. 12 COMPARISON OF P-M STRENGTH INTERACTION DIAGRAMS FOR THE CIRCULAR CROSS SECTION COLUMN OF FIG. 7 (31)

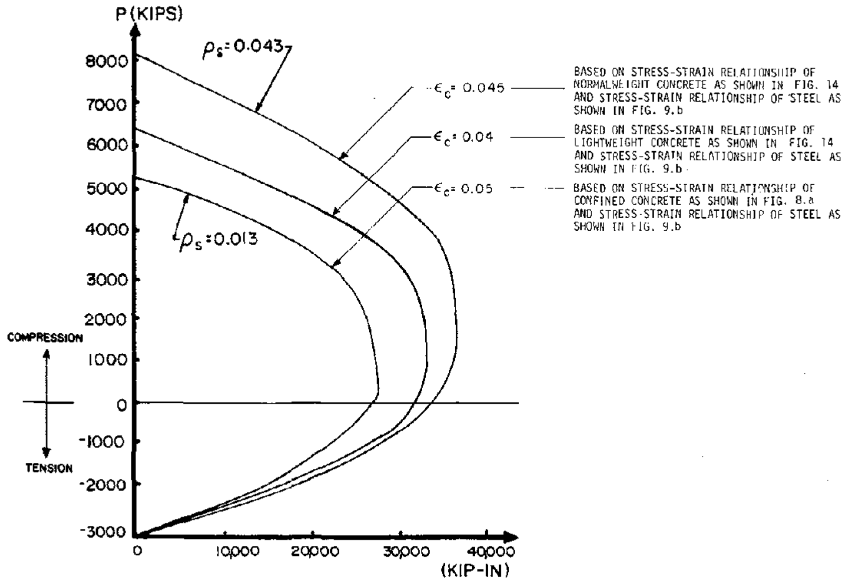


FIG. 13 EFFECT OF STRESS-STRAIN RELATIONSHIPS OF CONFINED CONCRETE (AS AFFECTED BY CONFINING PRESSURE AND AGGREGATE) ON THE P-M STRENGTH INTERACTION DIAGRAMS (31)

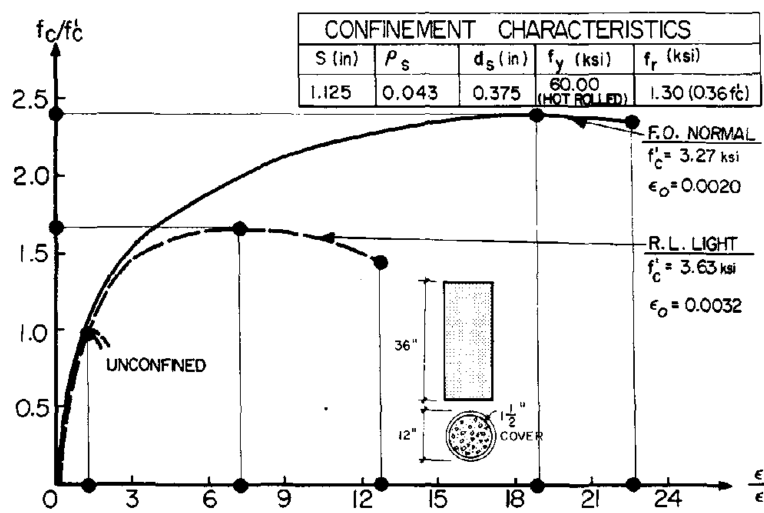


FIG. 14 NON-DIMENSIONAL STRESS-STRAIN RELATIONSHIPS FOR CONFINED, NORMAL, AND LIGHTWEIGHT CONCRETE (REF. 32)

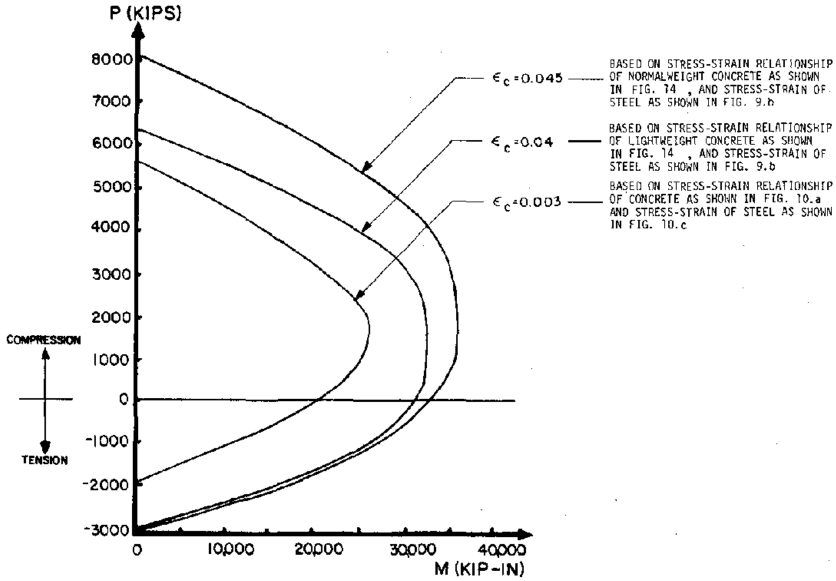


FIG. 15 COMPARISON OF P-M STRENGTH INTERACTION DIAGRAMS, COMPUTED USING THE EXPERIMENTAL STRESS-STRAIN RELATIONSHIPS, WITH THE ONE BASED ON ACI RELATIONSHIP (31)

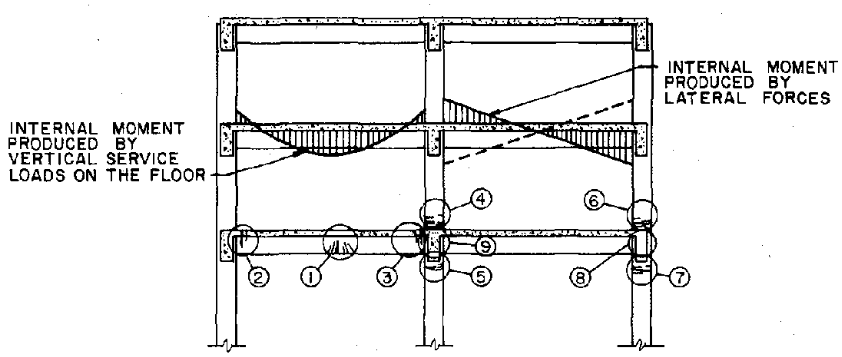


FIG. 16 TYPICAL CRITICAL REGIONS OF A REINFORCED CONCRETE MOMENT-RESISTING FRAME

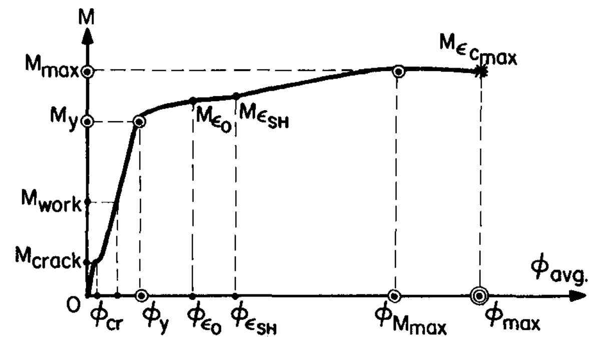


FIG. 17 TYPICAL MOMENT-AVERAGE CURVATURE RELATIONSHIP OF AN UNDERREINFORCED CONCRETE CROSS SECTION

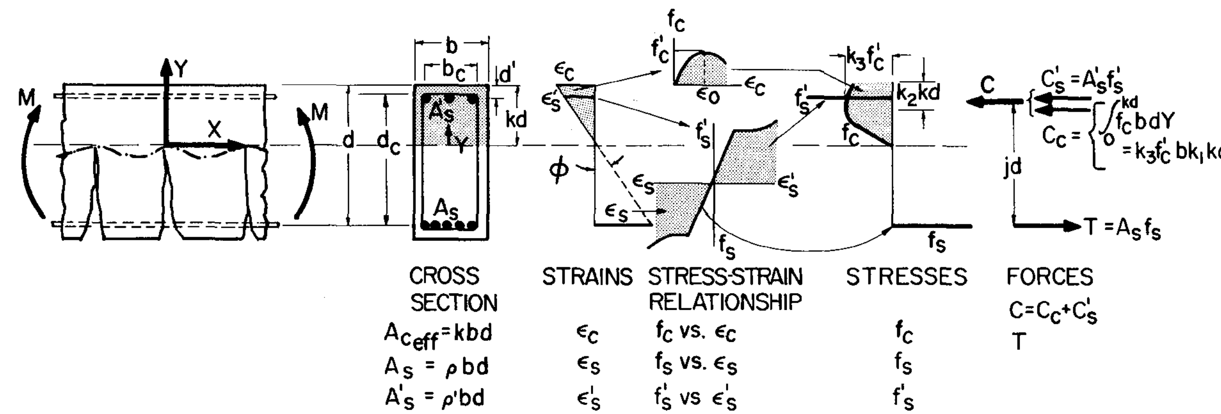


FIG. 18 DOUBLY REINFORCED CONCRETE SECTION SUBJECTED TO PURE BENDING

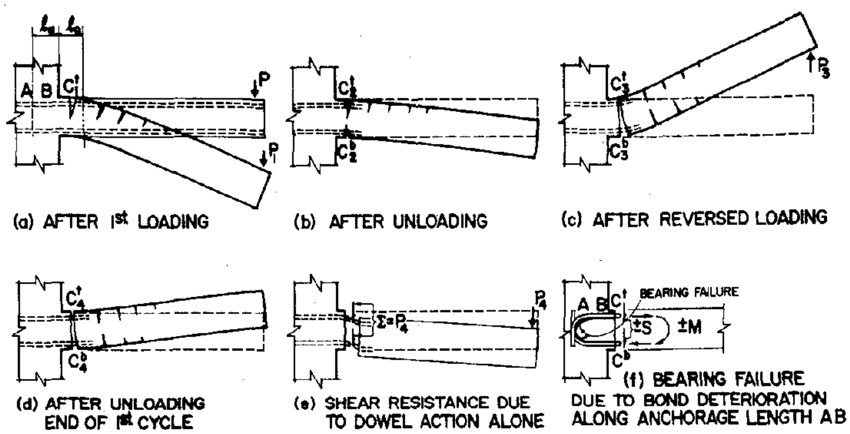


FIG. 19 EFFECT OF DEFORMATION REVERSAL ON REINFORCED CONCRETE FLEXURAL CRITICAL REGIONS (43)

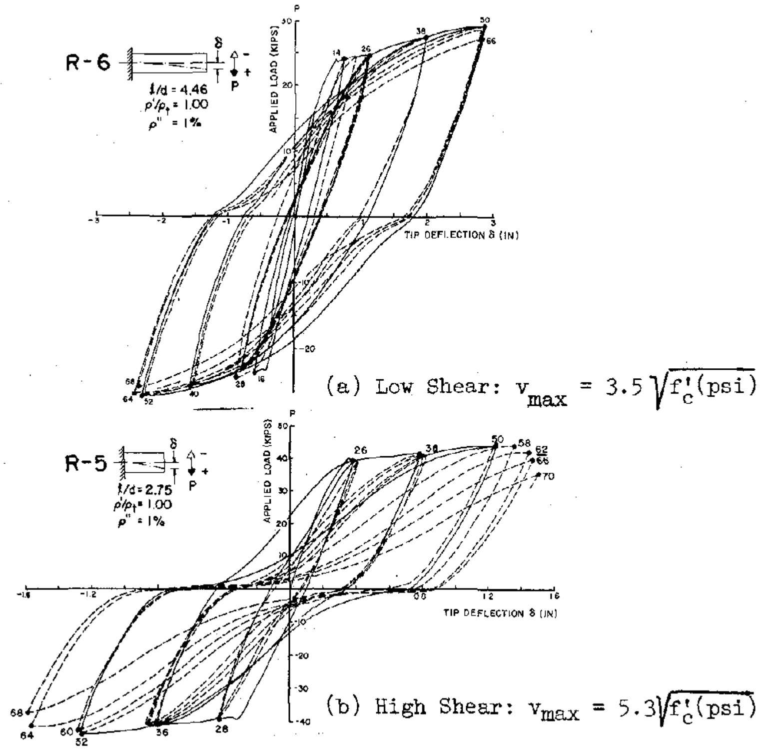
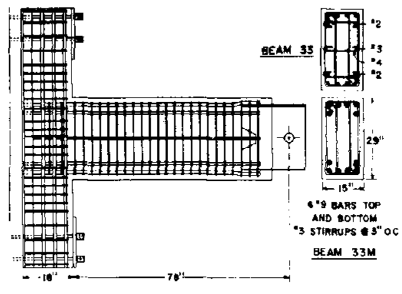
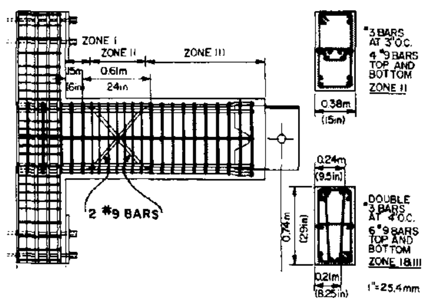


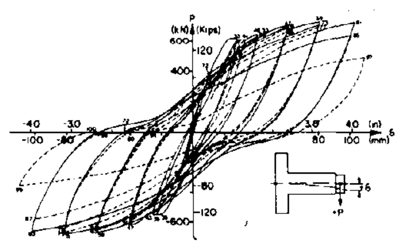
FIG. 20 COMPARISON OF HYSTERETIC BEHAVIOR OF REINFORCED CONCRETE AS AFFECTED BY DIFFERENT AMOUNTS OF SHEAR FORCE (48)



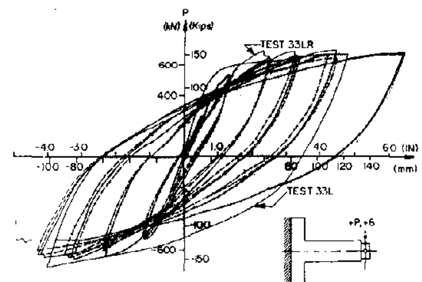
(a) Beams 33 and 33M



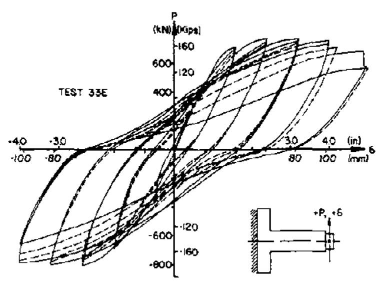
(b) Beam 33L



(c) Beam 33

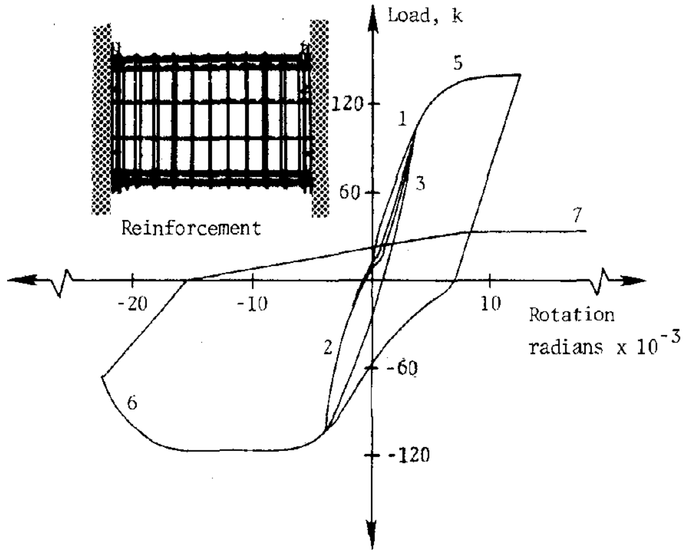


(d) Beam 33L

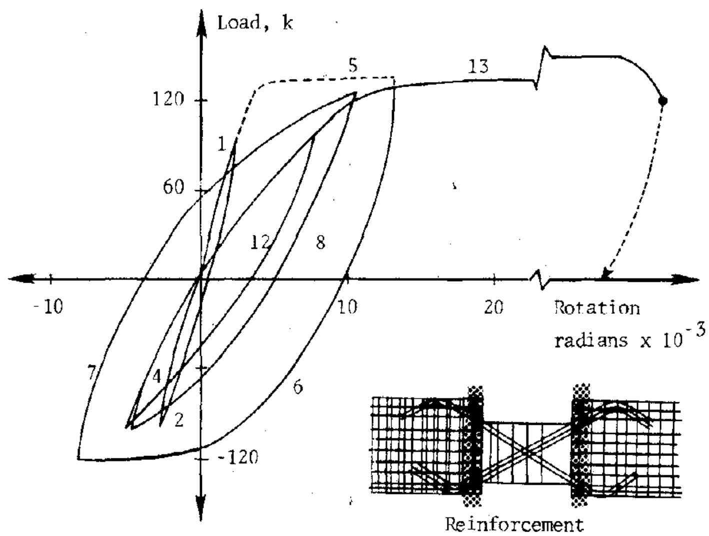


(e) Beam 33M

FIG. 21 EFFECT OF DIFFERENT TYPES OF WEB REINFORCEMENT ON HYSTERETIC BEHAVIOR OF REINFORCED CONCRETE BEAMS (50)

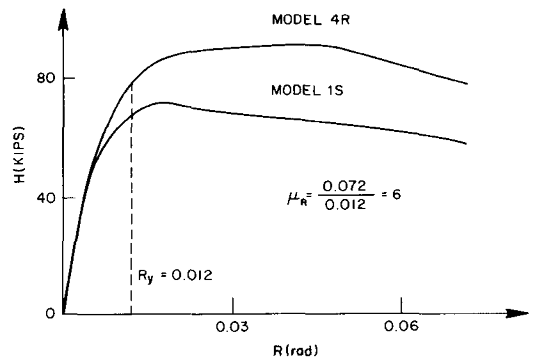


(a) Conventionally Reinforced

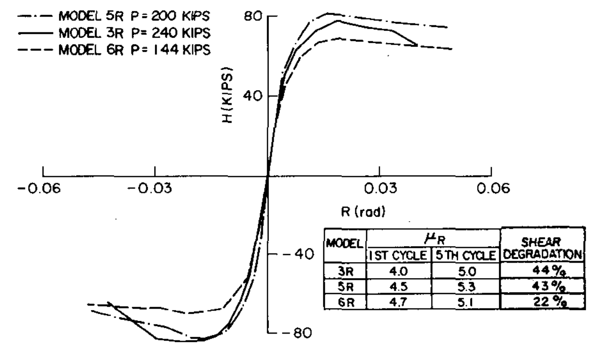


(b) Diagonally Reinforced

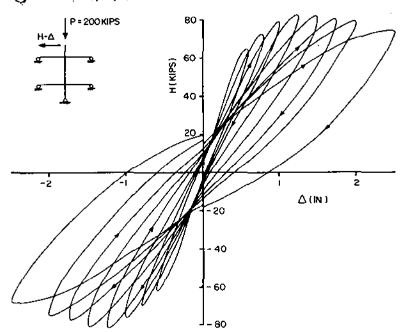
FIG. 22 LOAD-ROTATION DIAGRAMS FOR REINFORCED CONCRETE COUPLING BEAMS (AFTER PAULAY [13])



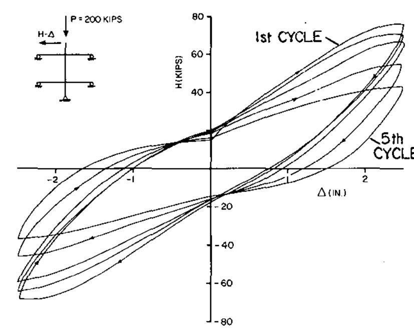
(a) Shear-Story Rotation Curves Under Monotonically Increasing Deformation



(b) Shear -Story Rotation Envelopes of Cyclically Deformed Columns



(c) First Cycle Shear-Tip Displacement Hysteretic Loops



(d) Shear-Tip Displacement Hysteretic Loops at 2.5in

FIG. 23 EFFECT OF DIFFERENT DEFORMATION HISTORIES ON LOAD-DEFORMATION RELATIONSHIPS OF SHORT R/C COLUMNS (51)

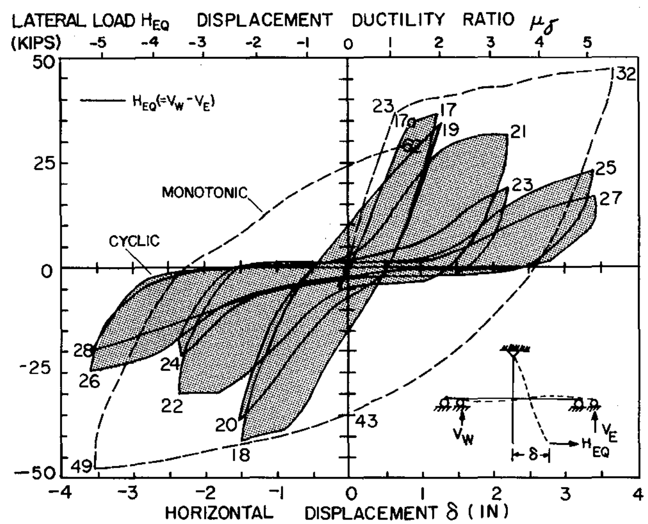


FIG. 24 INFLUENCE OF LOADING HISTORY ON LATERAL LOAD-DISPLACEMENT OF BEAM-COLUMN SUBASSEMBLAGE (61)

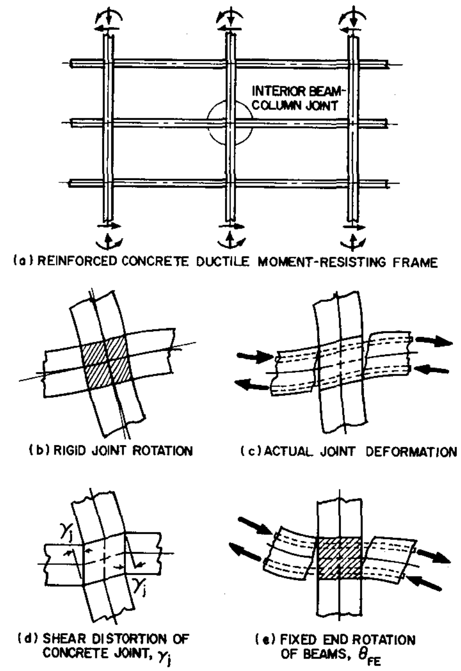


FIG. 25 SOURCES OF DEFORMATION IN AN INTERIOR BEAM-COLUMN JOINT OF A DUCTILE MOMENT-RESISTING FRAME

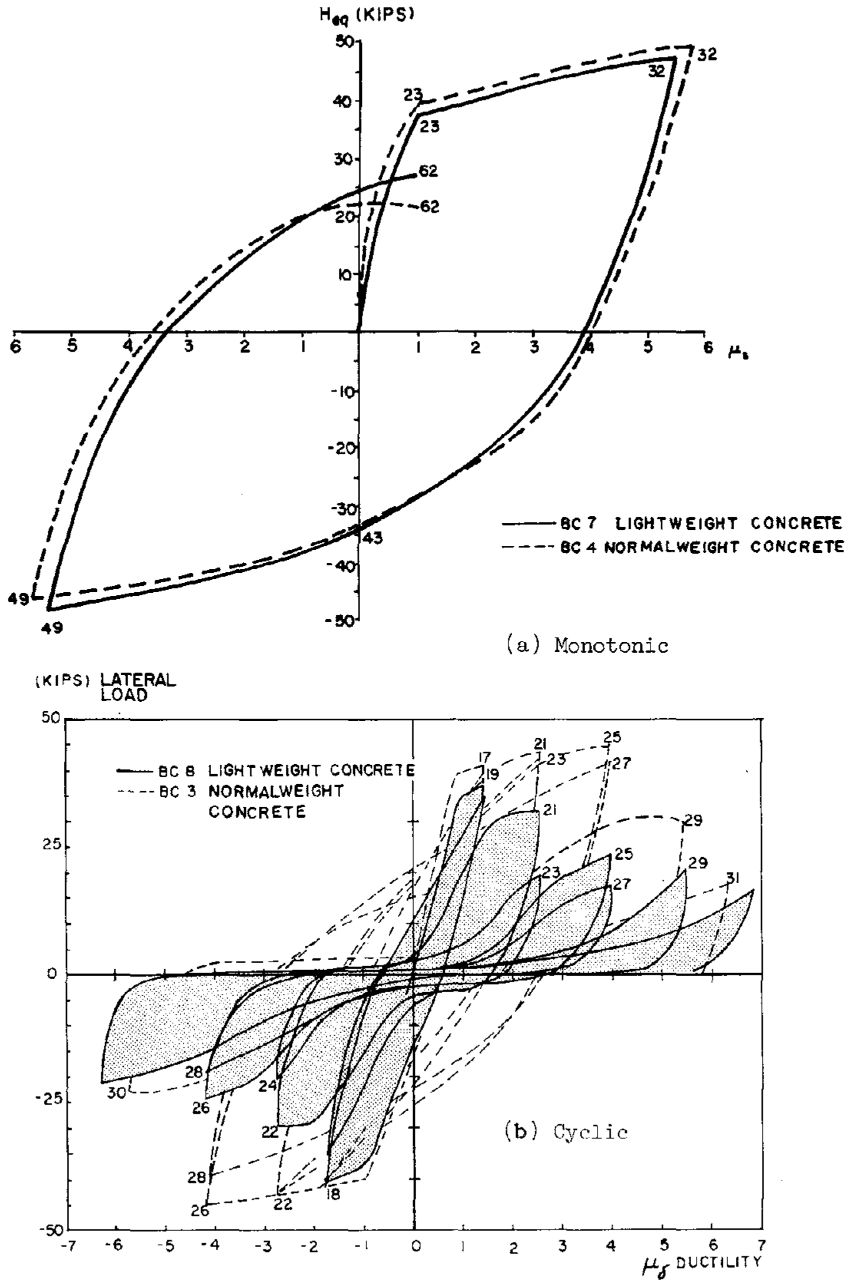


FIG. 26 COMPARISON OF BEHAVIOR OF LIGHT AND NORMAL WEIGHT CONCRETE SUBASSEMBLAGES UNDER DIFFERENT DEFORMATION HISTORIES (61)

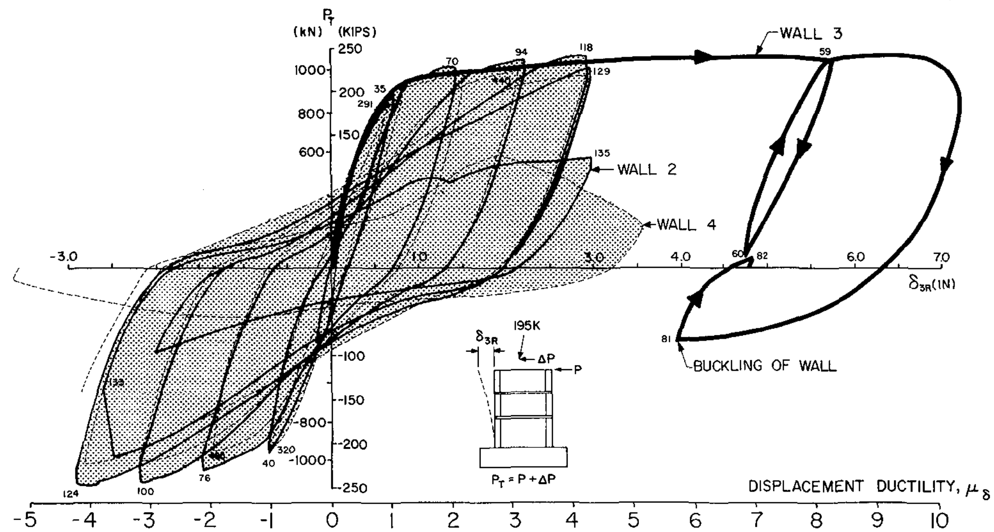


FIG. 27 COMPARISON OF BEHAVIOR UNDER MONOTONIC LOADING (WALL 3) WITH HYSTERETIC BEHAVIOR UNDER INCREASING DISPLACEMENT REVERSALS (WALLS 2 AND 4) (15)

SESSION III - DISCUSSION

INELASTIC BEHAVIOR OF STRUCTURAL ELEMENTS AND STRUCTURES

by

W.C. Schnobrich
Department of Civil Engineering
University of Illinois
Urbana, Illinois 61801

INTRODUCTION

Before deciding on what is already known and what remains to be determined related to the properties of high strength concrete so that that material can be effectively used by the profession, a preliminary question must first be answered. That question is, "What are the anticipated usages of such a high strength concrete?" Answers to this question will determine whether refinements in the knowledge regarding a particular stress-strain characteristic is necessary or not. Of course, before any realistic answers can be given to this preliminary question depends in part on our having basic knowledge of the properties of the material at the outset. Thus, to answer the question, presupposes at least some basic knowledge that this material will for example, have a high compression capacity, but accepts the fact that the tension strength behavior is still quite low compared to its compression characteristics.

If this basic question related to usage of high strength concrete were asked of a number of engineers, it is easy to speculate that their response would definitely focus on essentially the two applications already in common usage. One application is that of columns for highrise buildings if it is expected those columns are not to be subjected to significant cyclic or frequent reversals in loading. The second area involves the application to prestressed concrete structures. These applications are natural because of the dominant role of the compression mode as a load resisting mechanism for such structures. Thus, knowledge of the uniaxial and multiaxial stress-strain characteristics of the material, its limiting strain conditions, any creep characteristics, etc. must be delineated.

These aspects are discussed in the earlier session. Comments on the current knowledge of these characteristics of high strength concrete as that knowledge relates to the ability to anticipate the behavior and/or to the design of such structures built with high strength concrete elements has been very thoroughly discussed by the reporter of this session. Some of these aspects will be reiterated later in this discussion not to take objection to what is presented, but to reinforce the concepts by repetition. Before this is done, however, because the reporter has focused his attention basically only on highrise building structures composed of frames and walls, a further inquiry must be made to see if the scope should be expanded to consider any other structural elements which might possibly benefit from the use of high strength concrete.

In performing the search, structures which carry load predominantly by membrane or direct stresses in the form of compression should be likely candidates. The first obvious element or form would be various shell structures. Such structures utilize geometry in establishing their mechanism for carrying load. By proper organization of shape it is possible to achieve a structure which arches even load applied normal to it into a compression field and by that means down to its supporting structure or foundation (3,6,9). This ability to carry load by membrane or axial forces does occur for many shells, at least for a substantial area of the shell surface. So the force system or the resisting mechanism is right. In the design of reinforced concrete shells such as those used for roofs for example, the dominant controlling loading is that of dead weight (2). Therefore, should a thickness reduction be achieved because of increased strength the design load would decrease in a like manner further reducing the required section. This seems to point to an application that would produce a significantly more effective structure than is constructed from normal strength concrete. However, shells normally are such efficient shapes to begin with that the thickness is controlled not by strength requirements but by cover requirements for the steel, etc., while the stresses remain quite low. Therefore, the anticipated benefits expected from use of high strength concrete ate not achieved. Only for very long span roof shells is the thickness dictated by strength requirements. Also, when the shells being designed are cooling towers in the form of hyperboloids, keeping in mind that these shells are quite large structures, such shells are constructed with their thickness exceeding that controlled by minimum cover requirements, etc. This is very definitely the case with the five hundred foot high towers currently being used. For such shells the dead weight which produces meridional compression is a very significant design condition. However, wind induces a tension field that must be in part counterbalanced by this dead load compression field. Any decrease in dead load stress would then have to be compensated for by additional tensile steel needed to resist the design wind load (7). So even here, the bottom line rules out high strength concrete if use of that material had been selected solely on the basis of the increased strength producing reduced thicknesses and the associated reduction in dead load. It must be concluded that this failure to achieve any significant advantages with the use of high strength concrete must be true for all except the very monumental shells and even for these shells there may be other factors which decide against high strength concrete. These could be in the form of buckling or flexibility considerations.

Creep and shrinkage can be very significant factors effecting the performance of thin wall structures like shells so it is possible that high strength concrete may be used for reasons other than its compression strength.

A more positive conclusion can be drawn on the use of high strength concrete for long span prestressed concrete bridges. It has been used for a number of years. For such structures, the dominant role played by the magnitude of the dead load conditions is a major factor contributing to the advantages that the increase in compressive strength present in high strength concrete has in reducing the required cross section and therefore loading along with it.

FLEXURAL SYSTEMS

With flexural systems the desirability for underreinforced conditions produces a section whose behavior is controlled by the amount and placement of the steel. This means the level of the concrete strength will have little influence on the section's moment capacity. The obvious conclusion for such systems is that use of high strength concrete produces at best a negligible benefit. Consequently for floor systems and the like, normal strength concrete is selected as the most efficient. Determination of flexural capacities of ordinary beam proportions involves only routine calculations. Inclusion of either torsion or axial force (12) does not markedly alter or confuse this observation for structures whose design is controlled by normal loading conditions. These are some aspects of the torsion problem that still involve primarily speculation for example the manner by which a beam without web reinforcement sustains a torsion loading following cracking (12). On the whole response to these loading conditions are understood reasonably well. The change between normal or high strength concrete also does not alter or reduce the solvability of these problems.

Flexure and torsion problems are primarily three phase problems. After the initial elastic phase, the cracking phase develops and that depends upon the tensile strength of concrete. As noted by Gerstle in the earlier session (5) there is a decrease in the ratio of tensile strength to compressive strength with the higher compressive strength concretes. The final phase is initiated with yielding of the reinforcing steel. All this says then is that with the inclusion of high strength concrete, cracking is delayed for a short time, but then when it happens a slightly larger force must be redistributed by being absorbed by the steel. Even with high strength concrete the moment capacity is governed by the amount and placement of the steel. For normally reinforced beams, especially if compression steel is present, the moment capacity is not sensitive to the shape of the compression stress block so long as that shape is something reasonable (10). A similar observation can be drawn for the flexural strength of prestressed concrete beams (14). If beams or other basically flexural elements are subjected to the full combination of bending, torsion, and axial force then the analytical determination of the behavior is moved to an area where there is very little data upon which to base an evaluation of strength. This is true even for normal strength concrete, let alone any high strength concretes. Such a combination of forces may be rare as a controlling design condition. A few examples where such a combination should be considered are for columns of a cooling tower or supporting edge member for a shell, corner columns of a structure subjected to severe eccentric lateral load.

When consideration is expanded to include abnormal loading cases then the picture becomes much more vague. The reporter has directed the principal thrust of his report to delineating these problems particularly as they are generated relative to an analysis for earthquake response. The lack of any advantage for high strength concrete is noted. The importance of confinement is emphasized.

For slab problems yield line theory and/or nonlinear finite element techniques allow us to compute the flexural capacity of reinforced concrete plates. The finite element technique applied to reinforced concrete plates usually is accomplished nowadays as a layered model (8). The material properties of each layer of concrete are input consistent with the biaxial strength curves of Kupfer, Hilsdorf, Rusch (10) origin. For high strength concrete the corresponding state of this knowledge has been discussed by Gerstle (5). In addition to strength it is necessary to also define the stress-strain characteristics of the material if the behavior of the structure is to be determined. Employing an equivalent uniaxial curve is one method of estimating the stress-strain curve (4). This concept is directly applicable to high strength concrete also.

For a slab constructed from high strength concrete this data could be directed into appropriate computer programs and a load deflection curve defined therefrom for the slab. Because the flexural action doesn't really care that much about what the precise compression stress block looks like, exactly how the concrete is handled, what its stress-strain curves are, etc., are not that significant and any, almost arbitrary, shapes can even be used. The location and amount of steel are the controlling factors for the moment capacity. The details of the total displacement curve depend upon the tensile behavior that is cracking and post crack behavior (1,8). This depends not only on the material but also to a small degree on the analysis technique, geometric nonlinearities included or not, tension stiffening included or not, the element model used, treatment of support conditions, etc. Since high strength concrete does not offer any advantages for plate structures, the fact that one can compute the expected flexural behavior to within an acceptable tolerance is of no particular import to this meeting. If the slab is loaded so that moderately large deflections result, the ultimate capacity of the slab includes a significant inplane system of induced forces which can produce sizeable increases in ultimate load. Inclusion of the biaxial strength conditions for the high strength concrete coupled with a nonlinear geometric analysis would allow that influence to be computed in the rare case that one is still interested in the structure at such large displacements.

On those slabs which are used in highrise structures whose columns are cast with high strength concrete, the slab in the vicinity of the columns is also cast of high strength concrete. With the reduction in column size, to maintain punching shear capacity the stronger concrete is used and it serves the same function as a drop panel. The shear capacity is increased according to the normal square root of f'_c law. With the smoother crack surfaces associated with high strength concretes some adjustments in the basic shear transfer mechanism is to be expected.

AXIAL FORCE

A major impetus in the development and use of high strength concrete has been its utilization in columns of highrise buildings which are subjected to just normal loading conditions. With the strength increases possible, the same column size can be maintained over a sizable number of stories. Furthermore, the floor area in the lower stories is not totally eaten up by the columns. The use of high strength concrete therefore shows definite economic advantages (13). The principal areas of concern relative to the use of high strength concrete are the possible brittleness of a failure and the amount and disposition of reinforcement required to attain various degrees of confinement to the concrete. What if any descending branch exists on the stress-strain curve is strongly influenced by the presence of longitudinal and transverse steel? It is the reinforced column that is of practical interest, not just the stress-strain curve for the basic material. There is not an overabundance of data related to this problem and extrapolation of data from normal strength concrete is clouded by the somewhat different dilatancy characteristics of high strength concrete compared to normal strength concrete. Data are needed on this matter.

MEMBRANE BEHAVIOR

Walls represent another practical use of high strength concrete. Shear walls are an economic means of providing lateral load resistance to highrise buildings. In responding to these lateral loads, the walls funciton primarily in bending for a single wall while both axial force and bending are present in coupled walls. The magnitude of the axial load attainable in a coupled wall system depends upon the strength of the coupling beams. If the coupling beams have a large enough strength, it is possible to transmit a sufficiently high compressional axial force to fail the wall in a brittle manner. Therefore, the increased compressional strength obtainable with high strength concrete can be of benefit to these walls. Again an area of prime consideration is the confinement obtainable from the longitudinal and transverse steel. In order to determine the behavior of isolated walls it is basically a consideration of flexure and shear. Coupled walls include axial force. Knowledge of the biaxial stress-strain characteristics of the concrete would allow the analyst to investigate the critical lowest story of the wall. A finite element solution could be run (4). In view of the height of the walls, their basic behavior is that of a beam so that an ordinary beam analysis on the basis of an interaction curve is appropriate. The level of deformation that member can sustain is an important consideration. Cracking and location and amount of reinforcement steel are important factors that must be included. Shear transfer characteristics are very important in the overall behavior of shear walls. The nature of the cracks that develop with high strength concrete can have marked influence on the transfer mechanism. With high strength concrete a much smoother crack can result in a smaller aggregate interlock influence. Because there is a smaller tendency to push into the piece across the crack less normal force between the surfaces results from sliding one surface relative to each other. With adequate confinement, the percentage of shear transferred in the remaining compression block can be increased.

CONCLUSIONS

Numerical analysis techniques exist which are capable of delineating the overall behavior of reinforced concrete elements and structures. These techniqeus are presently being refined by analyzing reinforced concrete structures constructed from normal strength concrete. Such techniques should be capable of defining the nonlinear behavior of structures constructed of high strength concrete as well, once adequate material properties have been determined from tests in the laboratory. These laboratory tests should involve reinforced elements, not just the plain concrete itself. Currently, only very limited data exists on the mechanical properties. Because of the characteristics of high strength concrete, it has no particular advantage in flexural problems, thus ruling out many applications. In many other cases the advantages from the higher strength are lost due to service and cover requirements. Its principal applications seem still to be in columns of highrise buildings, longspan prestressed concrete bridges and some monumental shells particularly those involved in deep submergence structures.

REFERENCES

1. Bashur, F.K. and Darwin, D., "Nonlinear Finite Element Analysis of R.C. Slabs," Applications of Computer Methods in Engineering, USC Conference, 1977.

2. Billington, D.P., Thin Shell Concrete Structures, McGraw Hill, 1965.

3. Csonka, P., Membranschalen, Verlag von Wilhelm Ernst and Sohn, Berlin, 1966.

4. Darwin, D., and Pecknold, D.A.W., "Analysis of Cyclic Loading of Plane R/C Structures," Computers and Structures, Vol. 7, 1977.

5. Gerstle, K.H., "Material Behavior Under Various Types of Loading," Proceedings of Workshop on High Strength Concrete, Chicago, 1979.

6. Gould, P.L., Static Analysis of Shells, Lexington Books, 1977.

7. Gurfinkel, G.R., and Walser, A., "Analysis and Design of Hyperbolic Cooling Towers," Journal of Power Division, ASCE, Vol. 98, June, 1972.

8. Hand, F., Pecknold, D.A.W., and Schnobrich, W.C., "Nonlinear Layered Analysis of R.C. Plates and Shells," Journal of Structural Division, ASCE, July, 1973.

9. Isler, H., "Twelve Years of Application of Bubble Shells," Proceedings International Congress on the Application of Shells in Architecture, Mexico City, 1967.

10. Kupfer, H, Hilsdorf, H.K., and Rusch, H., "Behavior of Concrete Under Biaxial Stresses," ACI Journal, August, 1969.

11. Mattock, A.H., Kriz, L.B., and Hognestad, E., "Rectangular Stress Distribution in Ultimate Strength Design, ACI Journal, February, 1961.

12. Park, R. and Paulay, T., Reinforced Concrete Structures, Wiley Interscience, 1975.

13. Task Force Report, Chicago Committee on High-Rise Buildings, High Strength Concrete in Chicago High-Rise Buildings, Report No. 5, February, 1977.

14. Warwaruk, J., Sozen, M.A. and Siess, C.P., "Strength and Behavior in Flexure of Prestressed Concrete Beams," Investigation of Prestressed Reinforced Cocnrete for Highway Bridges, Part III, University of Illinois Experiment Station Bulletin 464.

SESSION III - SUMMARY OF FLOOR DISCUSSION

INELASTIC BEHAVIOR OF STRUCTURAL ELEMENTS AND STRUCTURES

James O. Jirsa
Department of Civil Engineering
The University of Texas at Austin
Austin, Texas 78712

Following the excellent report by Mr. Bertero and discussion by Mr. Schnobrich, several participants presented short prepared remarks and then the floor was open for general discussion. The comments are grouped by topic rather than by chronology. The recorder wishes to apologize in advance to any of the participants whose comments he has unintentionally omitted or misquoted.

GENERAL DISCUSSION

The floor discussion appeared to focus on two areas which cannot be totally separated: (1) the problem of obtaining response--force (stress, load, moment, etc.) vs. deformation (strain, deflection, curvature, rotation) characteristics for high strength concrete members or structures, and (2) the type of test conducted or procedure used to obtain that response. There was considerable discussion regarding the approach to be used with a dichotomy of views reflecting the research interests of the participants. For example, a material scientist may provide the information for producing high strength concrete and may also develop stress-strain characteristics for ingredients or the composite material. The engineer may then build on this information by using the stress-strain diagram to analytically produce stress-strain relationships for a member and subsequently for a structure. However, he may decide to determine the response experimentally to verify his analytical studies or to bypass difficulties which arise in the analytical approach. Finally, the structural designer is interested in the product of the research--how he can safely use the high strength concrete in structural applications.

SPECIFIC COMMENTS

Constitutive Relations--Mr. Pister hypothesized that for an inelastic material, a monotonic stress-strain diagram is meaningless because strain history becomes important. Constitutive relations need to include the time effect. Mr. Gerstle questioned why the stress-strain concept was not useful because the characteristics of composite sections could be obtained from material properties. Mr. Taylor felt that there was no controversy because any constitutive relations were based on assumptions which must be understood when applied or used. Mr. Bazant indicated that fracture mechanics concepts needed to be incorporated in order to develop constitutive relations for concrete because cracks develop and tangential forces are present along the cracks. Mr. Chen pointed out that generalized stress-strain relationships are different for reinforced concrete because so many variables enter in and scale becomes important. Mr. Dougill stated that from an analyst's point of view, once a model of material behavior is adopted, everything is real, even though the tests on which the model is based may not be real. Mr. Arya perhaps expressed the reaction of many of the participants when he stated that he was confused by the discussion, but that he felt we needed to continue to work toward defining material properties.

Confined Concrete--There was considerable discussion regarding the need to define the characteristics of confined concrete. Mr. Ghosh discussed the concept of confinement index as an equivalent hydrostatic pressure in which the spacing and area of ties, and material strengths are considered. He stressed the need to develop better equations for determining the effect of confinement. Mr. Gesund questioned the relative efficiency of tied and spiral reinforcement and if shear reduced the effect of ties as confinement. Mr. Bertero replied that ties will never be as efficient as a spiral because the workmanship will not be the same. Mr. Nilson asked at what point the concrete becomes a loose material whose characteristics are controlled by the confining steel. Mr. Bertero indicated that the stage at which this occurs is difficult to define.

Type of Test--The problems in defining force-deformation characteristics can be approached by conducting various types of tests. Mr. Ghosh pointed out the difficulty in defining rotational or curvature ductility and relating that to structural deformations. In addition, axial loading may help carry shear and improve ductility. Mr. Slate indicated that tension shear and compression shear are quite different. If compression is present, the crack surface will be able to transfer shear regardless of its smoothness. He pointed out that since concrete is a brittle material which fails in tension anything which prevents the development of tensile strains should improve behavior. Mr. Slate indicated that if the system being tested is unstable with time, the time effect must be included. Mr. Dougill pointed out the similarities between earthquake loading and fire resistance in terms of demands for load ductility. Mr. Shah discussed the importance of the loading method used--load vs. deformation control--and the type of loading--cyclic vs. monotonic. The concept of an envelope curve appears to extend to high strength and lightweight concrete. Mr. Uzumeri called attention to the large gap in member vs. material tests and the purpose for which each is conducted. In order to optimize progress tests of both kinds will continue to be needed. Mr. Nilson felt also that both kinds of testing should continue.

Techniques of High Strength Concrete Production--Mr. Ramakrishnan felt that there was not enough emphasis on the production of high strength concrete. He described the use of fiber reinforced concrete to improve energy absorption and impact resistance. Mr. Taylor discussed the use of three different shapes of fiber reinforcement and their efficiency in improving material characteristics. There was a brief discussion of the uses of high strength concrete. Mr. Bazant pointed out the advantage of high strength concrete in box girders. Mr. Pomeroy indicated that many applications can be developed by changing member or section configurations.

CONCLUDING REMARKS

The discussion centered on the approaches to be used to study high strength concrete materials or structures. It is difficult to suggest that one approach is better than another and all may play a role in the systematic advancement of technology. Table 1 shows a hierarchy of test procedures which will serve to illustrate the problem. For example, it should be possible to start at Level I and build up to Level VI by systematically developing the response at each level from the information derived at the previous level or levels. However, this approach presumes that all appropriate force-deformation relationships are available when proceeding to the next level. However, it can be seen that such an approach will be very time-consuming and will not permit the assimilation of new material into practice until all levels of response have been fully developed.

Therefore, certain levels may be bypassed entirely to develop usable information quickly. For example, it may be possible to determine structure response (Level VI) by studying mechanical properties (Level II), and then sectional response (Level IV). Or, all levels are bypassed and the entire structure (Level VI) is tested immediately.

There are obvious advantages to both approaches and merit in combining approaches. Therefore, the materials scientist should continue to develop and improve the technology associated with the production of the material and the materials engineer continue to study mechanical properties and constitutive relationships. The structural engineer cannot wait until all mathematical modeling questions are solved--he must develop response of members or structures using whatever information is available. In the end, progress will be made when the results from all levels of tests are contributed to the general knowledge in the field and communication between groups of researchers is open and candid.

TABLE 1 HIERARCHY OF TEST PROCEDURES

	Level of Response	Type of Test	Response as a Function of -
I	Material-Microstructure	Chemical Analysis, Microscopy	Ingredients, "recipe" used in production
II	Mechanical Properties	"Coupon" tests	Loading history, time, stress level, duration of load, exposure or environment
III	Constitutive relations	Isolate desired response, e.g. shear-friction bond slip, confined concrete	How materials are combined, e.g., type of transverse reinforcement, loading
IV	Sectional Response	Simple elements	Compatibility; slip, cracking
V	Member Response	Members, beams, columns	Scale
VI	Structure Response	Entire structure	Scale, type (massive or frame)

SESSION IV - REPORT

STRUCTURAL DESIGN CONSIDERATIONS FOR HIGH STRENGTH CONCRETE

by

Paul Zia
Department of Civil Engineering
North Carolina State University
Raleigh, N. C. 27650

ABSTRACT

Current available information on stress-strain relationship and maximum compressive strain of high strength concrete are summarized and some of the code limitations are discussed. Design considerations with respect to ductility, confinement of concrete, cracking, and connections and joints are described. Required researches are indicated.

INTRODUCTION

High strength concrete is a relative term. Not too long ago, while concrete of 7,500 psi (52.5 MN/m^2) to nearly 10,000 psi (70 MN/m^2) was not uncommon to the precast and prestressing industry [11], 5,000 to 6,000 psi (35 to 42 MN/m^2) would be considered high strength for ready-mixed or site-mixed concrete, and unattainable in certain areas of the country. However, within the last decade, materials scientists and technologists have made great strides in developing new and improved concrete materials. With careful selection of proper cementing medium and high-quality aggregates, along with careful mixing, placing, and curing, high strength concrete in the range of 9,000 to 12,000 psi (63 to 84 MN/m^2) is now attainable [3, 5, 10, 26]. The present techniques of ready-mix concrete, using conventional materials, can produce concrete of more than 9,000 psi (63 MN/m^2) compressive strength and it has been used in a number of high-rise buildings here and abroad [2. 6, 7, 25].

In a review of the state-of-the-art, Harris [12] described a variety of methods which have been used for the making of high strength concrete of at least 14,000 psi (98 MN/m^2). These methods involve compaction by compression with vibration, improvement of matrix and aggregate bond, the use of fibrous reinforcement, etc. Other significant developments include polymer-impregnated concrete, artificial aggregates, and sulfur-infiltrated concrete [2, 12, 14, 16, 20, 21, 22].

In his recent paper [24], Saucier classified high-strength concrete into three categories: (1) the present range of 5,000 to 10,000 psi (35 to 70 MN/m^2), (2) the available range of 10,000 to

15,000 psi (70 to 105 MN/m^2), and (3) the exotic area of 15,000 psi (105 MN/m^2). To produce concrete of the first category, normal practices would include the use of high cement factor, low W/C ratio, carefully selected course aggregate, more coarse sand (FM $\leq$ 3), water-reducing admixtures, fly ash, more stringent requirements of cement, and more coordination and quality control efforts. Concrete of the second category can be obtained with presently available materials and equipment but special processes, such as slurry mixing, no-slump concrete, compaction by pressure, new admixtures, closer control, longer curing, and polymer material. To extend into the exotic area of beyond 15,000 psi (105 MN/m^2), special materials and techniques will have to be considered such as artificial aggregates, vibration combined with pressure, silica-lime bond, and discontinuous reinforcement combined with polymer-portland cement matrix.

From the standpoint of practical and economical applications, and in terms of design considerations, it would be appropriate to define high strength concrete in the range of 8,000 to 12,000 psi (56 to 84 MN/m^2) for the purpose of this duscussion. Within this range, perhaps the most common application of high strength concrete is for columns in high-rise buildings [2, 6, 7, 25]. Experiences have shown that not only minimum cost is derived from the use of high strength concrete, the reduction in column size which provides more usable floor space is also a very significant economic advantage. That high strength concrete is not more widely used in other structural applications is due in large measure to the lack of sufficient information on its engineering properties. The following discussions will focus on some of the special design considerations relative to high strength concrete.

STRESS-STRAIN RELATIONSHIP

Obviously, one of the important properties of high strength concrete relative to flexure design is the stress-strain relationship. Recent studies [18] at Cornell University with high strength concrete specimens under both concentric and eccentric compression tests indicate "a fairly linear response up to maximum stress in contrast to the non-linear behavior of the medium and low strength mixes. The shape of the descending branch also differs, becoming more pronounced and smaller in terms of available post-peak strain as the strength increases." These observations confirm the test results obtained by Leslie, et. al. [15]. In addition, the studies further indicate that the maximum concrete strain ε_u was generally greater than 0.003, the value presently specified by the ACI Building Code [4], but there were enough other data below 0.003 to suggest a more conservative value of 0.0025. Based on an analysis of the characteristics of the observed stress-strain curves, the Cornell researchers suggested that an idealized trapezoidal stress-strain curve could represent conditions for the full range of concrete strengths. Such a curve was indeed used by the author in a previous study [28].

In contrast to the results of the Cornell studies, Wang, et. al [26] obtained a generalized non-linear stress-strain curve with a distinct descending portion up to a maximum concrete strain of 0.006, Fig. 1. Using such a relationship for a series of non-linear flexural analysis, they closely predicted the ultimate moment of the high strength concrete beams tested by Leslie, et. al. [15].

It is important to note the difference in test methods used by the various research groups. The test performed by Leslie, et. al., and at Cornell, used either standard axial compression test in which increasing loads were applied rather than increasing deformations, or eccentric compression test with deformation controlled loading. In either case, the release of energy stored in the testing machine, when the specimen was unloading, greatly influenced the shape of the descending portion of the stress-strain curve. On the other hand, Wang, et. al. used a test method in which the concrete specimen was loaded concentrically along with a case hardened steel tube so that there was no release of energy from the testing machine, and the loading was deformation controlled without strain gradient. The strain was not independently measured on the concrete specimen and, therefore, included the deformations of the thin capping materials and those of the end zones of the concrete specimen where the state of stress was not purely uniaxial. Moreover, the loading rate was 10×10^{-6} in/in/sec., about 2.5 times that used in the Cornell studies.

It may seem perplexing that by using a quite different stress-strain curve, Wang, et. al. [27] were able to make close predications of the test results obtained by Leslie, et. al. [15]. In reality, this is not surprising since the test results were all obtained from under-reinforced beams, and as such, the ultimate moment is insensitive to the shape of the stress-strain curve. Nor is it sensitive to the maximum compressive strain ε_u.

CODE LIMITATIONS

In a previous study by the author [28], it was demonstrated that since the equivalent rectangular stress block and the maximum concrete strain ε_u are assumed conservatively by the ACI Code, the use of the Code values could underestimate the beam capacity by as much as 40 to 50 percent for high strength concrete in the range of 10,000 psi (70 MN/m^2). To ensure ductile behavior, it is a common practice to limit the maximum amount of reinforcement in a beam to 75 percent of the balanced reinforcement ratio ρ_b, which is significantly influenced by f_c', f_y, and ε_u. Fig. 2 shows that the ACI limitation, based on $\varepsilon_u = 0.003$, is rather restrictive particularly for designs using concrete strength between 8,000 to 12,000 psi (56 to 84 MN/m^2). It is even more so, if the design ultimate strain ε_u is greater than 0.003.

Fig. 3 shows the effect of the ultimate concrete strain ε_u on the moment capacity of a section. It is obvious that with concrete of ordinary strength, the moment capacity of a section is not at all sensitive to the variation of ε_u. On the other hand, with high strength concrete, a variation of ε_u greatly affects the amount of the balanced steel ratio ρ_b. To develop the full potential of high strength concrete, a large percentage of high strength steel should be used. However, the section capacity as well as the mode of failure are also affected significantly when the section is reinforced with higher percentages of steel. Therefore, from the standpoint of design, it is essential that, in future researches, the magnitude of the ultimate concrete strain ε_u and the question of ductility should be fully examined.

When high strength concrete is used with high strength prestressed reinforcement, it can be shown [28] that the limiting value of prestressed reinforcement index ω_p and that of percentage of prestressing steel ρ_p are heavily dependent on f_c' and ε_u. Fig. 4 shows the moment capacity as a function of ω_p and Fig. 5 shows the moment capacity as a function of ρ_p. According to the ACI Code [4], the value of ω_p shall be limited to 0.30 in order to assure ductile behavior associated with "under-reinforced" member. However, if $\varepsilon_u = 0.003$ is assumed in design, as is the case with the current ACI Code, the limiting value of ω_p would be considerably smaller than what is specified by the Code. Furthermore, the value actually decreases with increasing concrete strength. For the section with ω_p exceeding the ACI limiting value of 0.30, the Code would limit the section capacity to a constant value whereas, in reality, the section capacity continues to increase with increasing values of ω_p.

The theoretical limiting value of ρ_p increases with increasing concrete strength. Thus, the use of high strength concrete enables a beam to be more heavily prestressed. Beyond the theoretical limiting value of ρ_p, there is a very significant increase in the ultimate moment capacity, but the beam would have the undesirable mode of compression failure.

In view of these observations, it can be concluded that for designs with high strength concrete, the current ACI Code provisions for ω_p and for the section capacity of "over-reinforced" members are quite inadequate, and appropriate modifications would be required.

DUCTILITY

It is generally understood that concrete becomes more brittle as its strength increases. With increasing strengths, the maximum concrete strain ε_u becomes progressively smaller and the descending portion of the stress-strain curve becomes shorter and steeper. Thus, there is a common concern that beams of high strength concrete may not have sufficient ductility.

Test data available in the literature [8] seem to justify the value of $\varepsilon_u = 0.003$ as specified by the current ACI Code. However, using a computerized analysis, Wang, et. al. [27] have shown that the maximum concrete strain ε_u in a beam depends on not only the concrete strength f_c' but also the amount of tension and compression reinforcement among other factors. While the value of ε_u does, in general, decrease rapidly as the concrete strength increases, it in fact increases for a singly reinforced beam of normal weight concrete with $\rho = 0.75\rho_b$, and for all beams of light weight concrete. Furthermore, the computed values of ε_u were all greater than the ACI value of 0.003, most of them being well over 0.004 [8]. It should be further noted that the computed values of ε_u were based on the stress-strain curves of virgin, unconfined concrete cylinders loaded monotonically without any preloading. Lateral confinement, preloading or fiber reinforcement, etc., could all alter the characteristics of the stress-strain curve, and thus affect the value of ε_u.

The studies by Wang, et. al. [27] also indicated that the computed ultimate curvatures for beams and eccentrically loaded columns were considerably greater than the predictions based on the ACI Code. It was equally true for the flexural ductility factor defined as the ratio of the ultimate curvature to the curvature at yield. In addition, it was clearly shown that the flexural ductility factor would be increased significantly by the addition of compression steel and the reduction of tension steel. It would also increase with increasing concrete strength.

The above observations were also made by Ghosh and Chandrasekhar [8], who further indicated that the use of lateral reinforcement in the critical zone of a beam would likewise increase its flexural ductility factor, although it is less effective than the use of compression steel.

Obviously, the ductility of a member can also be enhanced by a proper selection of the structural shape. For example, a tee beam would have a similar effect as a rectangular beam with compression steel where the depth of compression zone is reduced and thus the ultimate curvature is increased while the curvature at yield is decreased. As a result, the ductility of the tee beam is increased.

The curvature and thus ductility are also heavily influenced by the axial load. For an eccentrically loaded column, at axial load levels greater than the balanced failure load, the ductility is negligible. However, at load levels less than the balanced load, the ductility increases with decreasing load level [19].

CONFINEMENT OF CONCRETE

The strength and ductility of concrete are greatly increased if it is under confinement. The classical work of Richart, et. al. [23] demonstrated that the difference between the axial compressive strength of concrete under confining fluid pressure and that of unconfined concrete was four times the confining pressure. In addition, there was a very significant increase in ductility. The lateral confinement reduces internal cracking and volume increase of the concrete just prior to failure.

In practice, the confinement to concrete is usually provided by transverse reinforcement such as spirals, hoops, or closed stirrups. At low levels of stress, the concrete is virtually unconfined. As the concrete stress approaches the uniaxial strength, the transverse strains become progressively higher as internal cracking develops. The concrete then bears out against the transverse reinforcement which, in turn, exerts a confining force to the concrete. Since circular spirals are usually closely spaced and are in axial hoop tension, they provide a continuous confining pressure around the circumference. They are far more effective than square or rectangular hoops which can only exert confining pressures near the corners of the hoops because the sides of the hoops tend to bend outward under the pressure of the concrete.

The confinement can also affect significantly the shape of the stress-strain curve at high levels of strain, depending on the effectiveness of the confining reinforcement, strength of concrete, and the rate of loading. A number of idealized stress-strain curves for confined concrete with rectangular hoops are described in Reference [19]. It is not unreasonable to expect similar behavior for high strength concrete, but test data are needed to confirm these stress-strain curves.

CRACKING

In the recent study at Cornell University [18], the internal state of cracking at different levels of strain was observed for both normal and high strength concretes. At comparable strain level, expressed as a percent of the strain at maximum stress, the amount of microcracking in normal strength concrete was substantially larger than that in high strength concrete. Failure of high strength concrete was in a brittle manner and no surface cracks were noticed on the specimen before failure. Furthermore, the failure surface in both tension and compression specimens passed through aggregate and mortar without bias in high strength concrete specimens, whereas in normal strength concrete specimens failure occurred primarily at the aggregate-mortar interface. It appears that unless carefully selected high strength aggregate is used, the aggregate in the high strength concrete may fail to serve as "crack arrester" and thus provides less resistance to crack propagation. This is potentially a serious problem particularly at high stress level produced by repeated or reversal loading. Research in this area is urgently needed.

From the standpoint of structural design, cracking is of considerable concern for strength, ductility, and serviceability, especially when the steel stress level is high under service load condition. Current design criteria for minimum reinforcement and crack control are largely based on test data. Researches are needed to confirm the validity of the empirical parameters associated with these criteria. For special applications, discontinuous fiber reinforcement could be used effectively to serve as "crack arresters" [1] and to improve the elastic properties, strength, and ductility of high strength concrete if it can be justified economically. In this area, additional research is needed to gain more fundamental knowledge regarding the mechanism of bond between fibers and the concrete, and to provide more data on physical properties so that rational design procedures can be established.

CONNECTIONS AND JOINTS

The design of connections and joints may be grouped in two broad categories: corbels or brackets for precast structures, and beam-column joints of monolithic construction. Corbels or brackets usually support a single vertical concentrated load, such as the reaction of a precast girder. This concentrated load may also be accompanied by a horizontal force caused by shrinkage, creep, or temperature shortening of restrained precast girders attached to the corbel or moving load effect of a crane. The behavior and failure mechanism of a corbel are comparatively easy to visualize. In spite of the complexity of the stress pattern, a simple design procedure, based on an internal linear arch mechanism, can be followed. For very short corbels, the sliding shear along the face of the column may become critical, and the load resistance by the shear friction concept would be more appropriate. The current ACI design method is based on several extensive test programs [15, 17]. Limiting values are established for the nominal shear stress computed across the deepest section of the corbel. A logical question that must be raised is: Are the limiting values and the empirical parameters valid for design with high strength concrete?

Beam-column joints in a monolithic structure present a much more complex situation. The basic ideas in designing these joints are: (1) they must have adequate performance under service load; (2) they must have adequate strength for the most adverse load combination that the joining members will carry; (3) they will not govern the strength of the structure, thus allowing the joining members to develop their respective strength fully; and (4) they should be simple and easy to construct. The structural demand on the joints is greatly affected by the loading, whether monotomic or reversed. Confinement of the concrete is often developed by closed stirrups and ties. Structural members framed into a joint can also develop partial confinement of the concrete depending on the layout. Thus, the concrete in the joint is in a complex multiaxial state of stress.

The primary concern of the design is to provide ductile behavior of the joint. Limiting amounts of reinforcement are often specified depending on the strengths of concrete and reinforcement.

Due to the very complex state of stress, particularly so under reversed loading, possible cracking due to diagonal tension must be controlled. What must be considered are the questions as: How is the bond and anchorage performance of bars affected by the state of surrounding concrete? How can compression and shear be transferred through concrete cracked in diagonal tension? The current design approach, largely based on empirical experience, must be confirmed by design studies and additional experimental research. Attention should be focused on triaxial stresses. The mechanism of failure should be determined and the pattern of reinforcement evaluated.

OTHER CONSIDERATIONS

To take full advantage of the reduced weight-strength ratio of high strength concrete, lighter and slender structures can be proportioned. However, the issue of deflection and stability may become a serious concern since the modulus of elasticity of high strength concrete is only slightly larger than that of normal strength concrete. In addition, a large number of design parameters in the current practice are implicitly related to the tensile strength of concrete, such as development length, minimum reinforcements for flexure, shear and torsion, and maximum nominal stress for shear and torsion. Whether these design parameters are applicable to high strength concrete remains to be examined. Some questions here may be resolved by design studies such as the minimum reinforcement for flexure [28] and others may require experimental verifications.

CONCLUSIONS

Material scientists and technologists have made significant progress in recent years in improving the strength of concrete materials. Increasingly, structural engineers find the economic advantages in using high strength concrete in the range of 8,000 to 12,000 psi (56 to 84 MN/m^2). However, the primary application has been limited to columns in high-rise buildings. Other general structural applications have not been developed due in large measure to the lack of sufficient information on the engineering properties of high strength concrete.

Current knowledge regarding the stress-strain relationship and the maximum compressive strain ε_u are conflicting and incomplete. Tests should be conducted on over-reinforced beams in order to accurately assess these properties.

The mechanism of internal failure and the development of microcracking of high strength concrete, as well as its time-dependent properties under different states of stress must be thoroughly examined. Use of high strength steel as reinforcement should also be considered. Based on the results of these fundamental studies, the current design theories should then be verified or extended in order that high strength concrete may be utilized effectively and economically in more structural applications.

REFERENCES

1. ACI Committee 544, "State-of-the-Art Report on Fiber Reinforced Concrete," Journal of American Concrete Institute, Vol. 70, No. 11, Nov. 1973, pp. 729-744.

2. Bickley, John A. and Payne, John C., "High Strength Cast in Place Concrete in Major Structures in Ontario," paper presented at the 1979 Annual Convention, American Concrete Institute, Milwaukee, Wisconsin, March 18-23, 1979.

3. Blick, R. L., "Some Factors Influencing High-Strength Concrete," Modern Concrete, Vol. 36, No. 12, April 1973, pp. 38-47.

4. Building Code Requirements for Reinforced Concrete (ACI 318-77), American Concrete Institute, Detroit, Michigan, 1977.

5. Carrasquillo, R. L., Nilson, A. H., and Slate, F. O., "The Production of High-Strength Concrete," Report No. 78-1, Department of Structural Engineering, Cornell University, Ithaca, N. Y., May 1978, 91 pp.

6. Chicago Committee on High-Rise Buildings, "High-Strength Concrete in Chicago High-Rise Buildings," Task Force Report No. 5, Feb. 1977, 63 pp.

7. Day, K. W., "Quality Control of 55 MPa Concrete for Collins Place Project, Melbourne, Australia," Paper presented at the 1979 Annual Convention, American Concrete Institute, Milwaukee, Wisconsin, March 18-23, 1979.

8. Discussions of Reference 15, ACI Journal, Vol. 74, No. 3, March 1977, pp. 140-145.

9. Flajsman, F., et. al.,"Polymer-impregnated Fiber-reinforced Mortars," Journal of American Ceramic Society, V. 54, No. 3, March 1971, pp. 129-130.

10. Freedman, S., "High-Strength Concrete," (IS176.01T) Portland Cement Association, 1971, 19 pp.

11. Gerwick, Ben C., Jr., Construction of Prestressed Concrete Structures, Wiley, N. Y., 1971, pp. 42-48.

12. Harris, A. J., "Ultra High Strength Concrete," PCI Journal, Vol. 12, No. 1, February 1967, pp. 53-59.

13. Hollister, S. C., "Urgent Need for Research in High-Strength Concrete," ACI Journal, Vol. 73, No. 3, March 1976, pp. 136-137.

14. Kakacak, L. E., et. al., "Concrete-Polymer Materials, Fifth Topical Report," Brookhaven National Laboratory Report BNL 50390, December 1973.

15. Leslie, K. E., et. al., "Flexural Behavior of High-Strength Concrete Beams," ACI Journal, Vol. 73, No. 9, September 1976, pp. 517-521.

16. Malhotra, V. M., "Development of Sulfur-Infiltrated High-Strength Concrete," ACI Journal, Vol. 72, No. 9, September 1975, pp. 466-473.

17. Mattock, A. H., and Hawkins, N. M., "Shear Transfer in Reinforced Concrete-Recent Research," Journal of PCI, Vol. 17, No. 2, Mar/Apr. 1972, pp. 55-75.

18. Nilson, A. H., and Slate, F. O., "Structural Properties of Very High Strength Concrete," Second Progress Report, Dept. of Structural Engineering, Cornell University, Ithaca, N. Y., Jan. 1979, 62 pp.

19. Park, R. and Paulay, T., Reinforced Concrete Structures, Wiley Interscience, N. Y. 1975.

20. Poad, M. E., et. al., "Engineering Properties of Fiber-Reinforced and Polymer-Impregnated Shotcrete," Report of Investigations RI 8001, U. S. Bureau of Mines, Spokane, Washington, 1975.

21. Polymers in Concrete, Special Publication SP40, Smerican Concrete Institute, Detroit, Michigan, 1973.

22. Proceedings of Conference on New Materials for Concrete Construction, University of Illinois, Chicago, Ill., December 1971.

23. Richart, F. E., Brandtzaeg, A. and Brown, R. L., "A Study of the Failure of Concrete Under Combined Compressive Stresses," Univ. of Illinois Engineering Experimental Station, Bulletin No. 185, 1928, 104 pp.

24. Saucier, K. L., "High-Strength Concrete, Past, Present, Future," paper presented at the 1979 Annual Convention, American Concrete Institute, Milwaukee, Wisconsin, March 18-23, 1979.

25. Schmidt, W. and Hoffman, E. S., "9,000 psi Concrete - Why? Why Not?", Civil Engineering, Vol. 45, No. 5, May 1975, pp. 52-55.

26. Wang, F., Shah, S. P., and Naaman, A. E., "Stress-Strain Curves of Normal and Lightweight Concrete in Compression,"Journal of ACI, Vol. 75, No. 11, Nov. 1978, pp. 603-611.

27. Wang, P. T., Shah, S. P., and Naaman, A. E., "High-Strength Concrete in Ultimate Strength Design," Journal of Structural Division, ASCE, Vol. 104, No. ST 11, Nov. 1978, pp. 1761-1773.

28. Zia, P., "Structural Design with High Strength Concrete," Report No. PZIA-77-01, Dept. of Civil Engineering, North Carolina State University, Raleigh, N. C., March 1977, 65 pp. (also available as NTIS Report No. PB270215, Springfield, Va.)

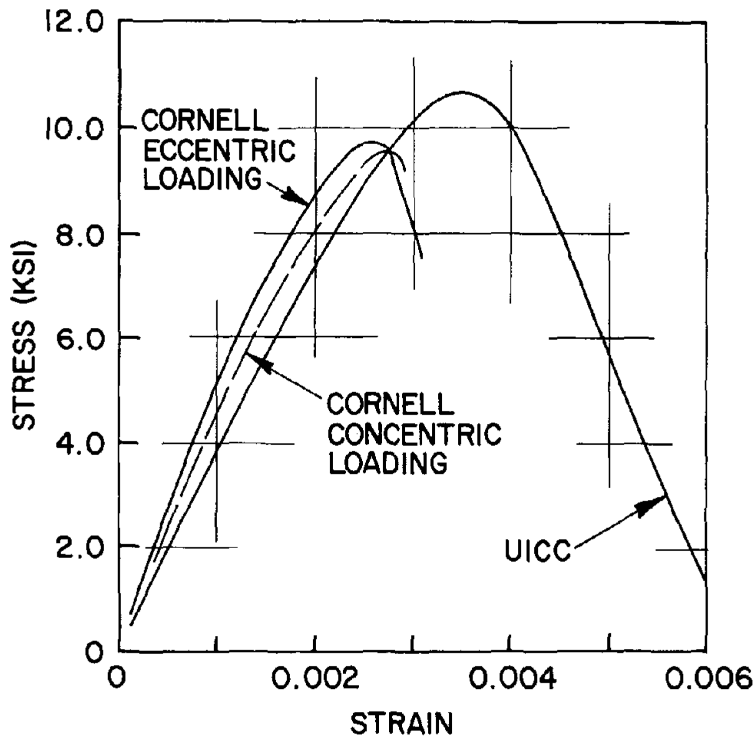


FIG. 1 TYPICAL STRESS-STRAIN CURVES

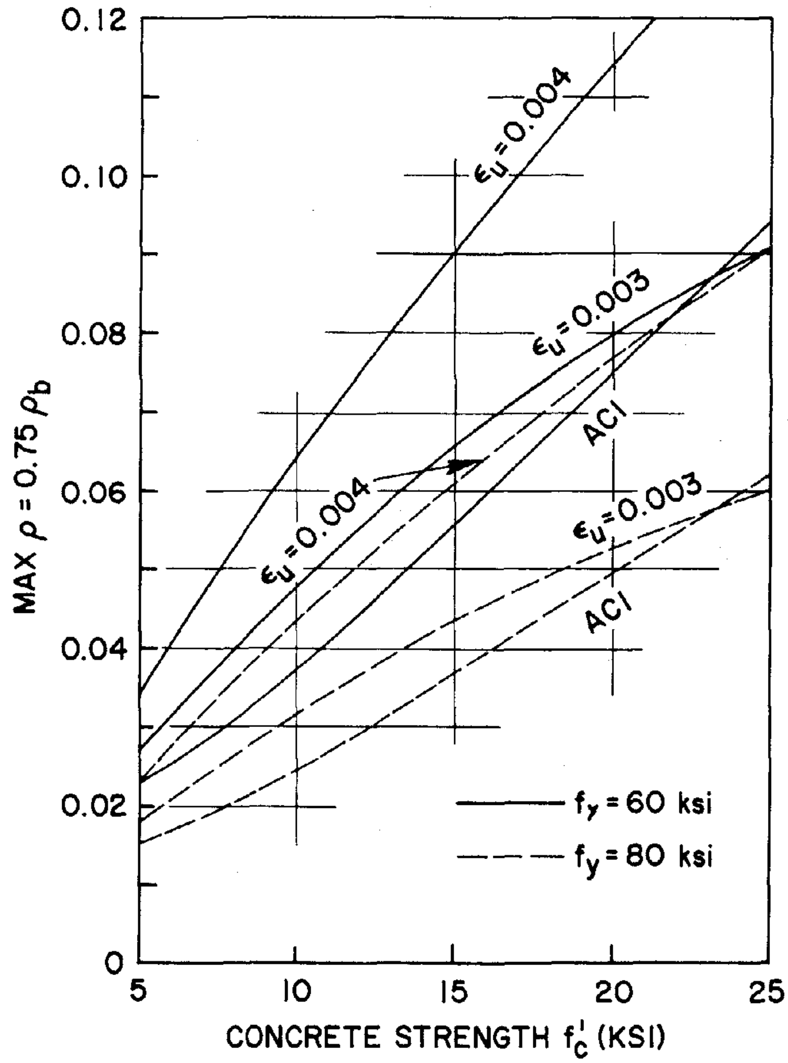


FIG. 2 MAXIMUM STEEL RATIO, MAX ρ

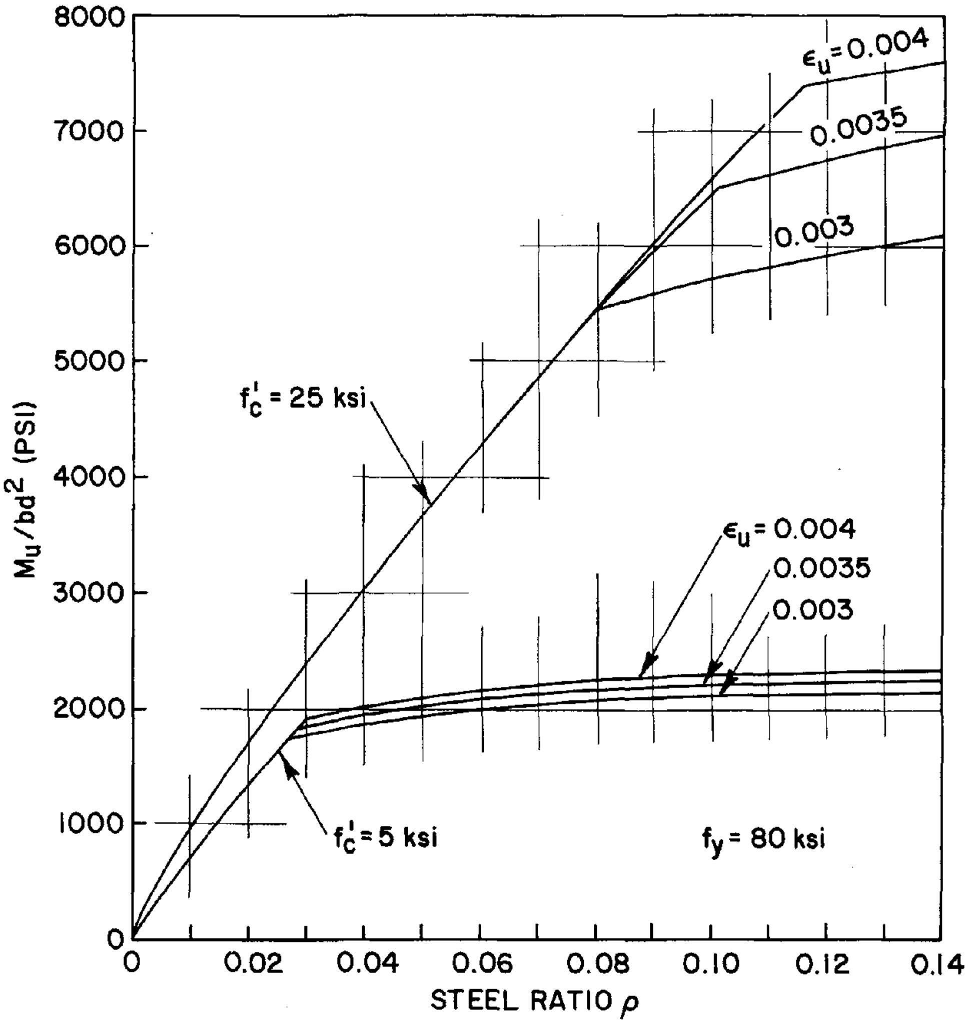


FIG. 3 EFFECT OF ULTIMATE CONCRETE STRAIN ϵ_u ON MOMENT CAPACITY OF SINGLY-REINFORCED SECTION

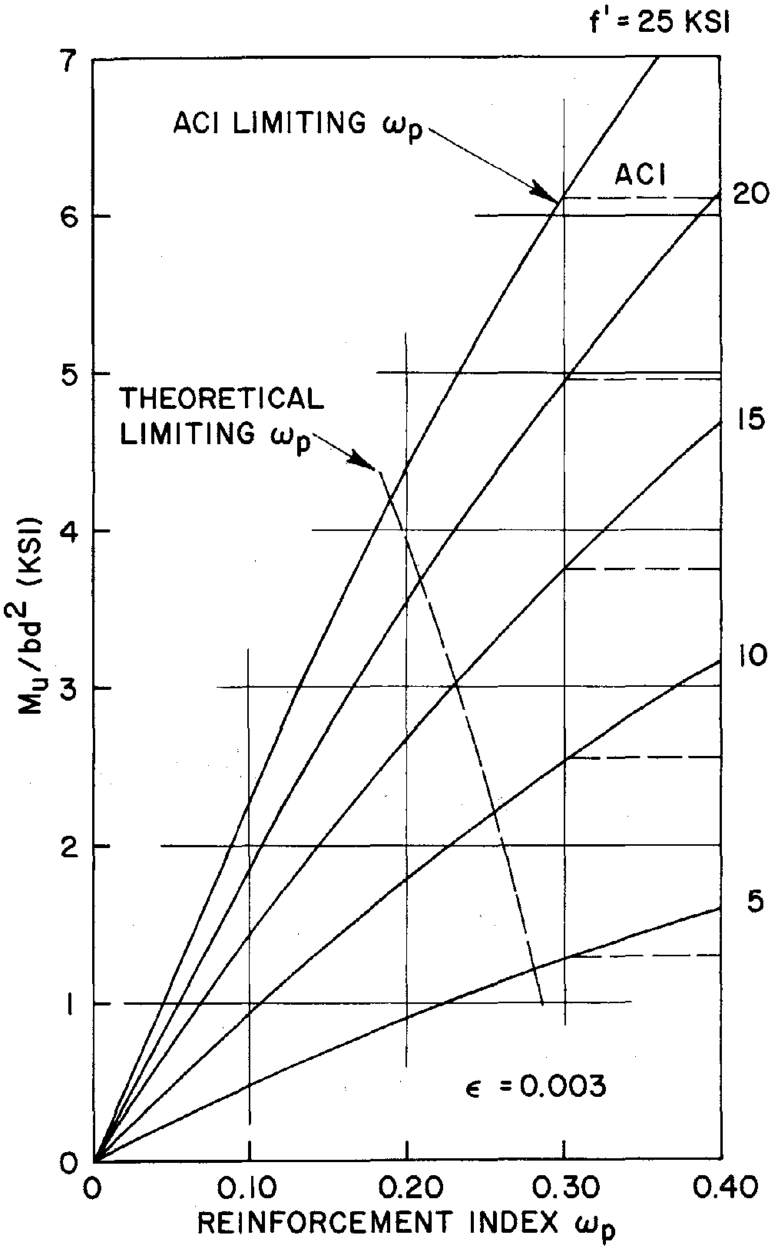


FIG. 4 MOMENT CAPACITY AS FUNCTION OF ω_p FOR f'_c = 5, 10, 15, 20, 25 KSI

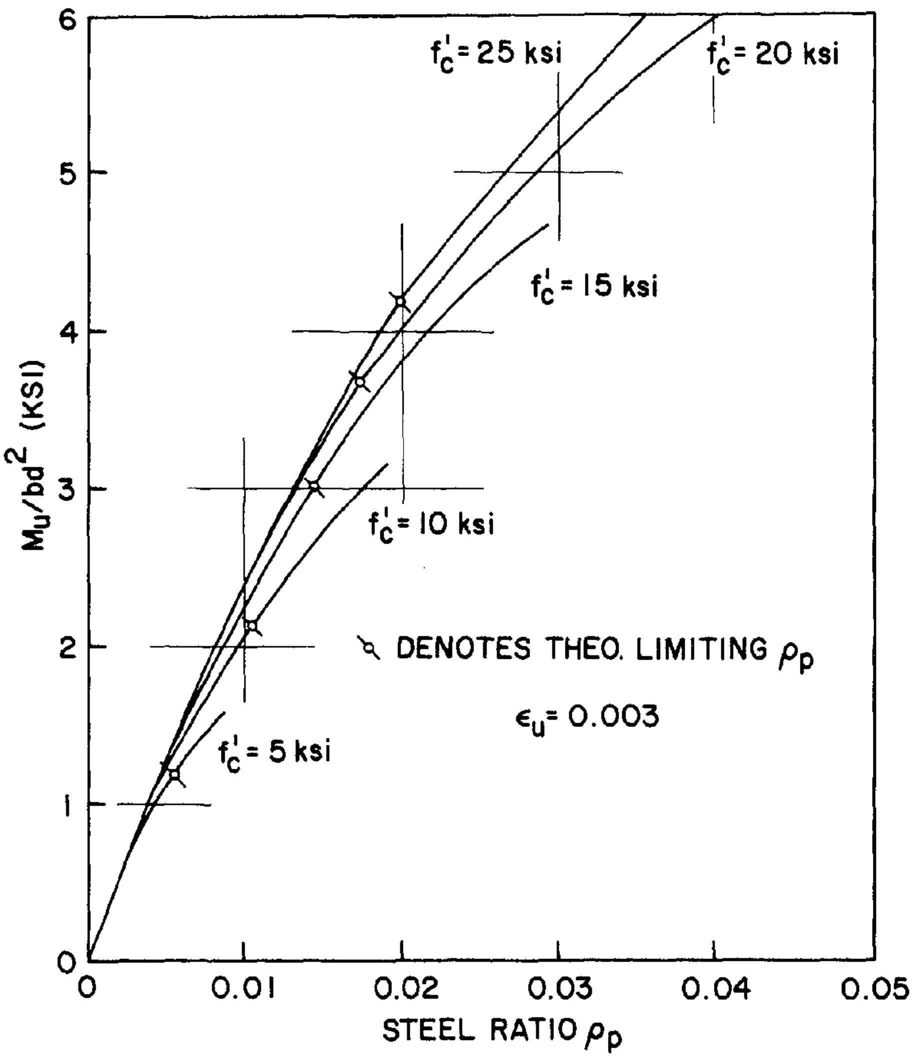


FIG. 5 MOMENT CAPACITY AS FUNCTION OF ρ_p FOR $f_c' = 5, 10, 15, 20, 25$ KSI

SESSION IV - DISCUSSION

STRUCTURAL DESIGN CONSIDERATIONS FOR HIGH STRENGTH CONCRETE

by

Antoine E. Naaman
Department of Materials Engineering
University of Illinois at Chicago Circle
Chicago, Illinois 60680

ABSTRACT

The existence of the descending branch of the concrete stress-strain curve in compression, methods to obtain it and its influence on the ductility of structural members are discussed. A new concept described as the Intrinsic Limit Strength of a concrete made with given components is proposed to explain the brittle type of failure often associated with high strength concrete. Material versus structural ductility and code limitations on reinforcement ratio and reinforcing index are addressed. It is hoped that a global design approach covering reinforced, prestressed and partially prestressed concrete beams will be arrived at in the future. Research needs in the fracture properties, cracking at the material and structural level, shear resistance creep, shrinkage and other physical and mechanical properties of high strength concrete are briefly reviewed.

INTRODUCTION - DEFINITION

It is true that the term high strength concrete is a relative term; it is relative in many ways depending on the type of application, the level of available technology, the type of aggregate used, the commercial and marketing aspects and even the geographic location. This is, in a way, a plus to concrete as it adds to its already known versatility.

It is essential, however, to realize that every material, in its mechanical properties, has its own theoretical limits and today we have the challenge to define the upper strength limit of concrete.

Concrete is a composite material; theoretically, the available range of strengths for concrete varies from zero to the maximum compressive strength of its components, i.e., either pure paste or the coarse aggregate which can exceed 210 MPa (30 ksi). Laboratory tests have shown that compressive strengths of more than 180 MPa (26 ksi) are achievable for a concrete composite and higher values possible. Although special precautions must be taken to guarantee such strengths, I am reluctant to consider as "exotic" (quoting K. Saucier (1)) the range above 100 MPa (15 ksi); similarly to the term "high strength," "exotic" is also a relative term. No one has ever considered the lower strength limit of concrete as exotic, though it is certainly easier to achieve.

In order to achieve the extensive utilization of concrete, we have been use to classify it in different groups often corresponding to different applications (normal weight concrete, lightweight concrete, cellular concrete, heavy aggregate concrete) and to consider their properties, say compressive strength from only an "average" viewpoint; that is, given average available components for the group, determine the proper proportions to achieve a desired level of strength (a level that is reasonable given the feasible range for that group). Seldom we approach the problem systematically and in its entirety where the above groups are all part of the same population; that is, given a wide range of components with a wide range of properties, given different techniques of production, select the proper components, their proportions and the proper technique to maximize the desired property. If we add to the usual concrete constituents newly studied additives such as fibers and polymers, then indeed we have a global optimization problem to solve for which global boundaries must be defined.

TESTING METHODS AND STRESS-STRAIN RELATIONSHIPS

As testing methods and recorded stress-strain curves are directly related to each other, they are better addressed simultaneously. Here what is often said in computer circles, quote "garbage in, garbage out," also applies to testing machines and testing procedures.

Definition of Strain

The observed or experimentally derived stress-strain relationship of concrete in compression depends on many parameters, many of which are completely foreign to the intrinsic properties of the material. Although the definition of stress or average stress seems to be easier to handle, the axial strain is not singly defined. Bertero and Vallenas (2) have pointed out that for non-reinforced concrete cylinders, everything else being equal, various strain measurements can be recorded depending on the gage length used and the size of the specimen. Similarly, different values are obtained if preloading is used. The definition of strain itself can be different depending on whether nominal strain is measured or the strain within a gage length and is affected by time and the rate of loading.

Testing Method

Once the stress and the strain to measure have been clear defined, the testing method (machine and controls) has to be appropriately selected especially if the descending portion of the stress-strain curve of concrete is to be recorded.

In a recent paper, Ahmad and Shah (3) have pointed out the major difficulties in obtaining the descending portion of the stress-strain curve of concrete and methods of overcoming these difficulties. Their conclusions were supported with many experimental results. Assuming that the material properties are such that a descending portion exists, it may or may not be possible to experimentally record it depending on the influence of one or a combination of the following factors (3):

1. The relative stiffness of the testing machine to that of the specimen. The stiffness of the machine is a machine constant while that of the specimen can be modified by changing the specimen's dimensions, namely, diameter and length. It can be shown that unstable (violent) failure will occur when the slope of the force-displacement curve of the specimen in the descending portion is larger in absolute value than the machine stiffness, i.e.,

$$|f'(\delta_s)| \geq K_m$$

   The above equation assumes that specimen failure is generalized and thus applies to small specimens. If failure is localized a different relationship applies as discussed in reference 13.

2. The method of application of the load. Most testing machines apply the load through a pre-specified constant rate of deformation. However, in this case it is difficult to control the strain energy release rate of the machine on the specimen during specimen unloading. An alternate approach is to use a closed-loop servo controlled testing machine in which an experimental output is used as the control signal. The signal can be the axial deformation of the specimen, or better, its circumferential deformation as both increase rapidly during unloading. (Note that the feedback transducer must be appropriately placed for maximum sensitivity in detecting the controlling variable.)

3. The speed at which the testing system respond to a given signal. Even in a closed-loop system the load on the specimen must be reduced at a sufficiently fast rate to balance (and eliminate) the rate of the strain energy release of the testing system. The speed at which a testing system can respond depends on the frequency response or response time of the system (electronic as well as hydraulic). For a given testing system, if the energy release rate is faster than the frequency response of the system then the machine specimen interaction cannot be avoided.

4. The effect of the applied strain rate by the machine. The higher the applied strain rate, the more difficult it is to control failure.

5. The velocity of crack propagation within the failing specimen. The more brittle the specimen is, the higher the velocity of crack propagation at failure and the more difficult it is to record and control the failure rate.

Existence of the Descending Portion of the Stress-Strain Curve

Now that the major influencing factors for obtaining the complete stress-strain curve of concrete have been clarified, let us approach the question of whether the descending portion of the curve exists and if it does whether it is significant or not.

There is an overwhelming agreement among researchers that a non-vertical descending portion of the stress-strain curve exists for concrete, at least up to a certain strength. There is disagreement on the value of that strength. Research at Cornell by Nilson and Slate (4) seems to indicate that the descending branch in uniaxial compression vanishes, for all practical purposes, for concretes of strengths above about 50 MPa (7 ksi).

Little evidence, however, exists to show that particular care was taken in their testing procedure to account for the effects of the above discussed influencing factors. In the same study, stress-strain curves obtained from testing flexural specimens are reported to show portions of descending branch for strengths up to 67 MPa (9.6 ksi). Research at UICC by Wang, Shah and Naaman (5) has indicated that the descending branch of the stress-strain curve in uniaxial compression is obtainable and reproducible; their tests covered strengths up to 90 MPa (13 ksi) at which the descending branch became very steep. Two testing methods were applied, one using a steel tube loaded in parallel with the specimen and the other using a closed-loop servo controlled system with the axial deformation as the feedback control signal. To vary the strength, only the age and water content of the concrete were varied. It was observed that the higher the strength, the steeper and the less extensive was the descending branch of the stress-strain curve.

That the descending branch exists seems undeniable to this author; the question is "how steep it is" for a given concrete material to be considered almost vertical and therefore insignificant. In addition to the experimental evidence, at least two arguments can be proposed to support the existence of the descending branch: first, if the descending branch is declared inexistent, one might arrive at the conclusion that for a given concrete the higher the strength the higher the toughness (energy to failure), a trend opposite to that generally observed in the science of materials, and second, high strength reinforced concrete beams (6,7) do not show under test the brittle type of failure otherwise expected if the material was assumed purely brittle. Finally, and for the sake of continuity, one cannot assume that the descending branch suddenly disappears after a certain limit strength for all types of concretes: it is already known that such limit strength would be different for normal weight or lightweight concrete.

Proposed Concept of Intrinsic Limit Strength

In spite of the above diverging viewpoints (between Cornell and UICC), the author believes that there is a point of conciliation: the descending branch of the stress-strain curve of concrete in compression becomes almost vertical at a certain limit strength defined here as the intrinsic limit strength of the concrete material used. The intrinsic limit strength would be mainly dependent on the properties of the constituent materials, mostly the coarse aggregates and their ability to arrest and deviate the path of the propagating cracks during unloading. The above concept is easy to accept. We already know that a lightweight aggregate concrete shows a steeper descending branch than a normal weight concrete of equal strength (Fig. 1). This suggests that if with a given type of aggregate and matrix it is possible to attain for instance a 140 MPa (20 ksi) maximum limit strength, then at that strength it is likely that the descending branch would be almost vertical; however, if for the same concrete the water content is increased to obtain a 70 MPa (10 ksi) strength, then that concrete would have a distinctive descending branch. The influence of the aggregate on the intrinsic limit strength of the material is quite well illustrated in the work of Tognon, et al. (7); they pointed out that, everything else being equal, the maximum attainable strength of their concrete doubled to a value of about 180 MPa (26 ksi) by only replacing calcareous aggregates with crushed quartz aggregates. The concept of intrinsic limit strength could also be correlated with other properties of the material such as limit strain, ductility or fracture toughness. As limit strength is associated with a steep descending portion due to rapid and brittle crack propagation through the aggregate, it is very likely that it could also be explained by measuring the fracture properties of the material.

Thus in order to give a general representation of the stress-strain curve of concrete at any strength f_c', it is necessary to refer to the corresponding intrinsic limit strength $(f_c')_{IL}$ of that concrete. Such a representation could be as described in Fig. 2 where possible values of intrinsic limit strengths are also reported.

MATERIAL DUCTILITY VERSUS STRUCTURAL DUCTILITY

There seem to be some confusion about the word ductility and a clear definition is necessary.

Ductility describes the capability of the material or structure to sustain substantial deformation beyond a certain level of loading. Ideally this deformation should not be accompanied by a loss of resistance for the member; however, a material can still be described as ductile even if the resistance drops beyond a peak value. It is essential to provide a descriptive measure that accounts for all cases.

For concrete, as a material, a good measure of ductility seems to be the area under the stress-strain curve beyond the peak or maximum stress. It is, in a way, a measure of toughness or energy absorption beyond the peak. Figure 3 shows that the ductility of high strength concrete is smaller than that of a low strength concrete using same components. Another measure of ductility could be defined as the ratio of the area under the post-peak portion of the curve to the area under the pre-peak portion. This definition might have the advantage of providing a relative measure independent of the system of units used.

Structural ductility of reinforced concrete members is generally defined as the ratio of curvature at ultimate (maximum) load to that at yielding of the reinforcing steel. The curvature at ultimate which is directly related to the maximum compressive strain in the concrete ε_{cu} at ultimate depends on many factors including the shape of the section, the reinforcement ratio and the stress-strain curve of the concrete. This is illustrated in Fig. 4 taken from (3) where three cross-sectional shapes and two reinforcement ratios are studied; values of ε_{cu} and corresponding curvatures at ultimate can be more than twice those predicted by the ACI code. Thus assuming that ε_{cu} is a constant and often confusing its value with ε_o (the strain at peak concrete stress) (Fig. 5) might be an erroneous approach when evaluating ductility; note, on the other hand, that it does not seem to be critical in the evaluation of ultimate moment.

Often ductility of the structural member is associated with the ductility of the constituent material, here concrete. They may, however, show different trends; for instance, an overreinforced member will show little ductility even if made with a low strength ductile concrete; similarly, a member made with high strength concrete may show significant ductility if the reinforcement ratio is kept at low values. This is illustrated in Fig. 6 taken from the work of Tognon, et al. (7) on reinforced concrete beams using concrete with 160 MPa (23 ksi) compressive strength. (Note that in Fig. 6 the reported stress-strain curve of their concrete material shows no descending portion after the peak.)

Study by Wang, et al. (8) has shown that the ductility of reinforced concrete beams 1) increases with concrete compressive strength, if the reinforcement ratio is kept constant, 2) remains almost constant if the reinforcement ratio is kept as a constant fraction of the balanced ratio and 3) increases with the amount of compressive reinforcement (Fig. 7).

It seems therefore that there is no particular difficulty in achieving desired levels of structural ductility using high strength concrete. This will be even easier to reach if confinement is used (such as for columns) or if additives such as fibers are added to the concrete matrix. Confinement or the presence of fibers will increase the ductility of the concrete material and will influence the shape of the stress-strain curve of high strength concrete beyond the peak stress to make it look like that of low strength concrete (Fig. 8).

CODE LIMITATIONS

Stresses and Strains at Ultimate

It has been shown in many investigations that the ACI assumptions of the rectangular stress block representing the concrete compression zone lead to a very good prediction of the ultimate moment capacity of the section. Generally, predicted values are within 10% of the experimental values observed for low strength as well as high strength concretes and on the safe side (9). The simplicity of the rectangular stress block, however, does not lead to a loss in accuracy. In effect the predicted ultimate moment seems to be insensitive, in many cases, to the shape of the stress distribution in the concrete compressive zone. This is illustrated in Table 1 where four hypothetical types of stress distributions are examined for two types of sections and two concrete strengths. It can be seen that in some instances even an inverted triangular stress shape does not influence much the results obtained, especially if the reinforcement ratio is kept reasonably below the balanced value. Note that in predicting the ultimate moment the value of ε_{cu} does not intervene. It is only important in the prediction of ultimate curvature and ductility and the code might have to specify an ε_{cu} value which depends on the many parameters known to influence its value. Anyway, it is unlikely that a change in ε_{cu} will substantially affect the ultimate moment capacity, even if an exact nonlinear analysis is performed and especially for high strength concrete. This is so because little is gained in moment resistance by adding the influence of the steep descending portion of the stress-strain curve. The above conclusion is different from that obtained by Zia (10) because he assumed an elasto-plastic stress-strain curve of concrete even for very high strengths.

Reinforcement Ratio and Reinforcing Index

The ACI code limit the reinforcement ratio of reinforced concrete beams to $\rho_{max} = 0.75\,\rho_b$ where ρ_b is the balanced ratio given by:

$$\rho_b = k\beta_1 \frac{f'_c}{f_y}\left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y}\right) \tag{1}$$

where

ε_{cu} = maximum compressive strain in concrete at ultimate capacity

f'_c = compressive strength of concrete

ε_y = yield strain of reinforcing steel

f_y = yield stress of reinforcing steel

k, β_1 = factors describing the stress block at ultimate

Note that ρ_b increases with an increase in f'_c and decreases with an increase in f_y. Thus ρ_{max} and ρ_b will be higher for high strength concrete than for low strength concrete.

As in prestressed concrete a balanced ratio cannot be as clearly defined as in reinforced concrete, the ACI code limits the reinforcing index q_{max} to 0.30. The author believes that the concept of reinforcing index should be extended to cover reinforced, prestressed and partially prestressed low and high strength concrete. The reinforcing index of a beam without compressive reinforcement is defined by:

$$q = \frac{A_s}{bd}\frac{f_y}{f'_c} + \frac{A_{ps}}{bd}\frac{f_{ps}}{f'_c} \tag{2}$$

where A_s and A_{ps} are the areas of reinforcing and prestressing steel and f_{ps} the stress in the prestressing steel at ultimate. The reinforcing index is proportional to the forces in the steel or in the concrete at ultimate. This can be seen from writing the equation of force equilibrium in the section at ultimate:

$$0.85\, f'_c\, b \beta_1 c = A_s f_y + A_{ps} f_{ps} \tag{3}$$

Dividing all terms by bdf'_c leads to the definition of

$$q = \frac{0.85\, f'_c\, b\beta_1 c}{bdf'_c} = \frac{A_s f_y}{bdf'_c} + \frac{A_{ps} f_{ps}}{bdf'_c} \tag{4}$$

or

$$q = \rho_s \frac{f_y}{f'_c} + \rho_{ps} \frac{f_y}{f'_c} \tag{5}$$

As from the viewpoint of the concrete it does not matter whether the force comes from the reinforcing steel or the prestressing steel, it seems quite rational to limit the value of the reinforcing index instead of the reinforcement ratio. Everything else being equal when f'_c increases, q decreases. However, in considering the limiting value of q, when f'_c increases, ρ_b increases thus ρ_{max} increases and it is not clear if q_{max} (Eq. (5)) should decrease. In fact, it is very likely that it will remain about constant.

If we use the ACI equation to define ρ_b for reinforced concrete and then derive from it the value of q_b, it seems that q_b will vary little. Thus $q_{max} = 0.75\, q_b$ will also vary little and a limit of $q_{max} \simeq 0.30$ seems quite reasonable for high strength as well as normal strength reinforced prestressed and partially prestressed concrete.

The above result can also be derived from numerical data obtained by Wang, et al. (8) on nonlinear analysis of reinforced concrete members where the exact ρ_b was calculated and compared to the ρ_b predicted by ACI at f'_c up to 14 ksi (Fig. 9). A least square line fit of the data giving ρ_b versus f'_c for normal weight and lightweight concretes led to the following equation:

$$\rho_b = 0.00675\, f'_c + 0.0053 \tag{6}$$

where f'_c is in ksi.

From the above equation a q_b can be derived such as:

$$q_b = \rho_b \frac{f_y}{f'_c} = 0.00675\, f_y + \frac{0.0053}{f'_c} \tag{7}$$

As the intercept (second term) of the above equation is negligible, we end up with:

$$q_b \simeq 0.00675\ f_y \tag{8}$$

and

$$q_{max} \simeq 0.75\, q_b \simeq 0.00506\, f_y \tag{9}$$

For $f_y = 60$ ksi, $q_{max} \simeq 0.303$. Thus limiting q_{max} to 0.30 for all cases seems quite reasonable and this value should not change significantly for high strength concrete.

CRACKING

Cracking will be addressed either at the material level or at the structural level.

At the material level, the study of the extent of microcracking and the path of the cracks can be correlated to the type of failure under load. Specifically, it seems that a brittle or a ductile failure can be obtained depending on whether the cracks go through the aggregates or around them. As in most concrete, the aggregates occupy more than 50% of the total volume of concrete; the quality of the aggregate and their capability to arrest cracks is instrumental in the type of failure obtained. Although microscopic studies of the extent of microcracking before failure and studies of the shape of the stress-strain curve in compression and the type of failure under compression do give a qualitative indication of the "ductility" or "brittleness" of the material, they are not sufficient to describe quantitatively these properties. It is very likely that the application of fracture mechanics to concrete will provide some positive answers (11). There is a substantial research need not only to systematically evaluate the fracture properties of concrete, but to evaluate the applicability of existing fracture mechanics approaches and test methods to concrete. The nature of concrete requires specimen sizes much larger than those usually used for steels and polymers and thus more appropriate test methods may have to be developed. It is hoped that the descriptive behavior of concrete as brittle or ductile material will be quantitatively measured by an appropriate fracture parameter.

From the standpoint of structural design, it does not seem that the variables that influence crack spacings and widths in normal strength concrete will be different from those in high strength concrete. However, the values of these variables and other coefficients used in the crack width prediction equations might be substantially different. For example, it seems that the bond strength at the concrete-steel interface for high strength concrete does not increase in direct proportion to its compressive and tensile strength. This means that, everything else being equal, the higher the concrete strength the higher the crack spacing and the larger the crack width in either flexural or tensile members. A solution to this problem would be to reduce or limit the size of the reinforcing bars used with high strength concrete in order to increase the total bonded area at the interface.

Another point that might require special attention is the magnitude of the sudden stress increase in the reinforcing steel immediately after cracking and the corresponding jump in bond stress increase on either side of the crack.

As the tensile strength of high strength concrete is substantially higher than that of normal strength concrete, a higher load is needed to crack a reinforced flexural or tensile member. Thus the sudden tensile stress increase in the steel and the corresponding bond stress increase would also be higher. Studies by Goto (12) have indicated that this sudden jump in bond stress might lead to debonding along a certain length of rebar on either side of a crack and thus crack spacings and widths would be higher. Research is needed to evaluate this phenomenon in high strength reinforced and partially prestressed concrete members.

SHEAR STRENGTH

Little information exists on the shear properties of high strength concrete. In designing shear reinforcement, the current ACI code takes into consideration the shear crack resistance of the concrete (although a crack is assumed formed). This is due to interlocking of the aggregate and essentially leads to the capability of a rough crack to provide notable resistance to loading. As in high strength concretes, a brittle type of failure may occur accompanied by a relatively smooth crack, the above provision of the ACI code may have to be substantially revised such as to reduce to zero the shear contribution of concrete when its strength reaches its intrinsic limit strength.

TENSILE STRENGTH, MODULUS OF RUPTURE, ELASTIC MODULUS, SHRINKAGE AND CREEP

Recent investigation at Cornell (4) by Nilson and Slate on concretes of strengths up to 84 MPa (12 ksi) led to the following predictive equations for the split cylinder tensile strength, the modulus of rupture and the modulus of elasticity in psi units:

$$f'_{se} = 6.8\sqrt{f'_c} \tag{10}$$

$$f'_r = 11.7\sqrt{f'_c} \tag{11}$$

$$E_c = 18.3w^{3/2}\sqrt{f'_c} + 1.6 \times 10^6 \tag{12}$$

More research is needed to ascertain if the above relationships would be still valid for f'_c up to 1800 MPa (26 ksi). Note that the above equation for the elastic modulus leads to smaller values than those predicted by the current ACI code equation. This will certainly have an important effect on observed deflections and their limitations. Such limitations may offset the advantages of using high strength concrete in flexural members.

In the same investigation (4) information was provided on the shrinkage and creep properties of high strength concretes. Corresponding conclusions indicate that the ultimate shrinkage strain and the rate of shrinkage are higher than expected from observations on normal strength concretes. On the other hand, a tendency toward a decrease in creep strain with an increase in strength was observed under same relative stresses. The above investigation is being continued and will generate additional needed information on the above properties.

CONCLUSIONS

It is clear from the above discussion that more research is needed to evaluate the properties of high strength concrete and the particular design aspects related to its usage in structural applications. The above research should not necessarily be limited by the need to find and justify a type of application. High strength concrete is not widely used today not only because of the lack of sufficient information on its engineering properties but also because of economic consideration as well as the limited number of known applications--say column--where its usage can be justified.

In spite of this, however, we researchers, scientists and structural engineers specializing in concrete ought to consider the challenge of finding the limits of this material and providing useful information on its utilization up to these limits independently of the type of application we now know of. We today cannot yet foresee the whole array of applications in which high strength concrete might play an important role. Full exploration of the oceans, space and the underground may demand information on and the use of high strength concrete even before we have had the time to provide a complete answer.

REFERENCES

1. Saucier, K. L., "High Strength Concrete, Past, Present, Future," Paper presented at the American Concrete Institute Annual Convention, Milwaukee, Wisconsin, March 1979, 11 pp.

2. Bertero, V. V. and Vallenas, J., "Confined Concrete: Research and Development Needs," Proceedings, Workshop on Earthquake Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 1977, Vol. II, pp. 594-610.

3. Ahmad, S. and Shah, S. P., "Complete Stress-Strain Curve of Concrete and Nonlinear Design," Proceedings of the CSCE-ASCE-ACI-CEB International Symposium on NonLinear Design of Concrete Structures, University of Waterloo, Waterloo, Ontario, Canada, Aug. 1979.

4. Nilson, A. H. and Slate, F. O., "Structural Properties of Very High Strength Concrete," Second Progress Report, Department of Structural Engineering, Cornell University, Ithaca, N.Y., Jan. 1979, 62 pp.

5. Wang, P. T., Shah, S. P. and Naaman, A. E., "Stress-Strain Curves of Normal and Lightweight Concrete in Compression," Journal of the American Concrete Institute, Vol. 75, No. 11, Nov. 1978, pp. 603-611.

6. Leslie, K. E., Rajagopalan, K. S. and Everard, N. J., "Flexural Behavior of High Strength Concrete Beams," Journal of the American Concrete Institute, Vol. 73, No. 8, Sept. 1976, pp. 517-521.

7. Tognon, G. Ursella, P. and Coppetti, G., "Design and Properties of Concretes with Strength Over 1500 kg/cm^2," Presented at the 1979 Annual Convention, American Concrete Institute, Milwaukee, Wisconsin, March 1979.

8. Wang, P. T., Shah, S. P. and Naaman, A. E., "High Strength Concrete in Ultimate Strength Design," Journal of the Structural Division, ASCE, Vol. 104, No. ST11, Nov. 1978, pp. 1761-1773.

9. Mattock, A. M., Kriz, L. B. and Hognestad, E., "Rectangular Concrete Stress Distribution in Ultimate Strength Design," Journal of the American Concrete Institute, Vol. 32, No. 8, Feb. 1961, pp. 875-926.

10. Zia, P., "Structural Design with High Strength Concrete," Report No. PZIA-77-01, Department of Civil Engineering, North Carolina State University, Raleigh, N.C., March 1977, 65 pp. (NTIS No. PB270215).

11. Bazant, A. P. and Kim, S. S., "Plastic-Fracturing Theory for Concrete," Journal of the Engineering Mechanics Division, ASCE, Vol. 105, No. EM 3, June 1979, pp. 407-428.

12. Goto, Y., "Cracks Formed in Concrete Around Deformed Tension Bars," Journal of the American Concrete Institute, Vol. 68, No. 4, April 1971, pp. 244-251.

13. Bazant, Z. P. and Panula, L., "Statistical Stability Effects in Concrete Failure," ASCE Journal of the Engineering Mechanics Division, Vol. 104, No. EM5, Oct. 1978, pp. 1195-1212.

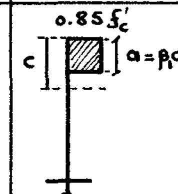
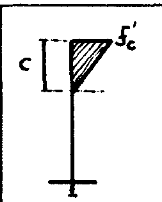
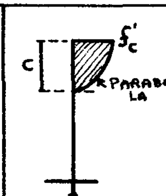
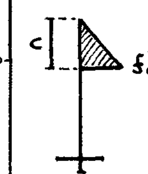
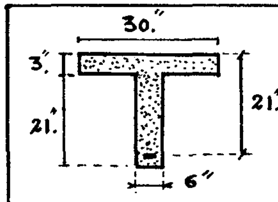
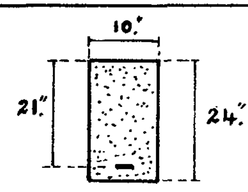
		$1000\left[\frac{M_u}{f'_c\,bd^2}\right]$			
	f'_c ksi (MPa)				
 $\rho = 0.1\,\rho_b$	5 (35)	17.6	17.5	17.6	17.3
	10 (70)	16.1	16.2	16.0	15.9
 $\rho = 0.5\,\rho_b$	5 (35)	177.3	174.2	178.4	147.2
	10 (70)	147.8	145.7	148.5	127.9

TABLE 1 Results Illustrating the Non-Sensitivity of Ultimate Moment to the Shape of the Stress Diagram Assumed at Ultimate Behavior

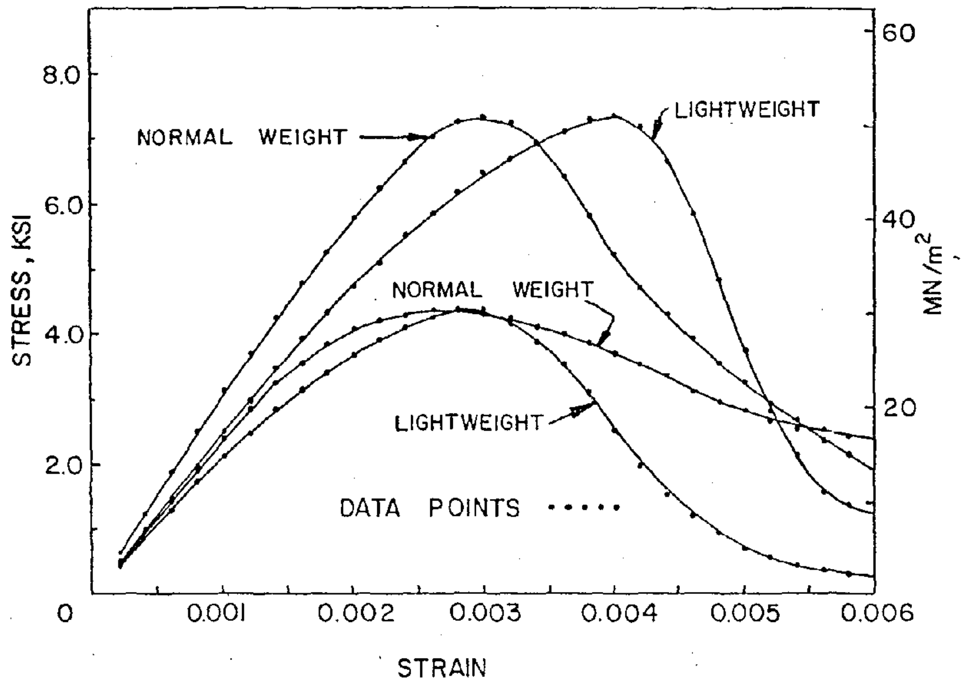


Fig. 1 Effect of Aggregates on the complete stress-strain curves of concrete (Ref. 5)

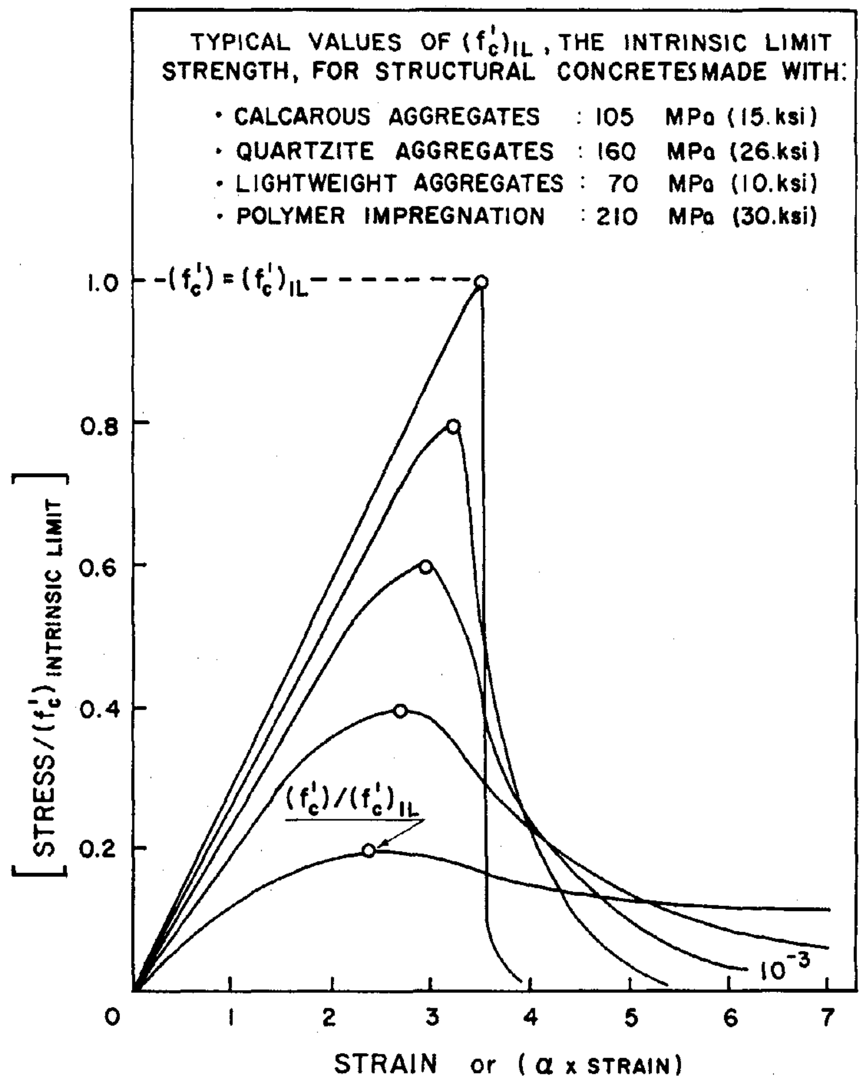


Fig. 2 Conceptual representation of the stress-strain curve of concretes in function of the intrinsic limit strength

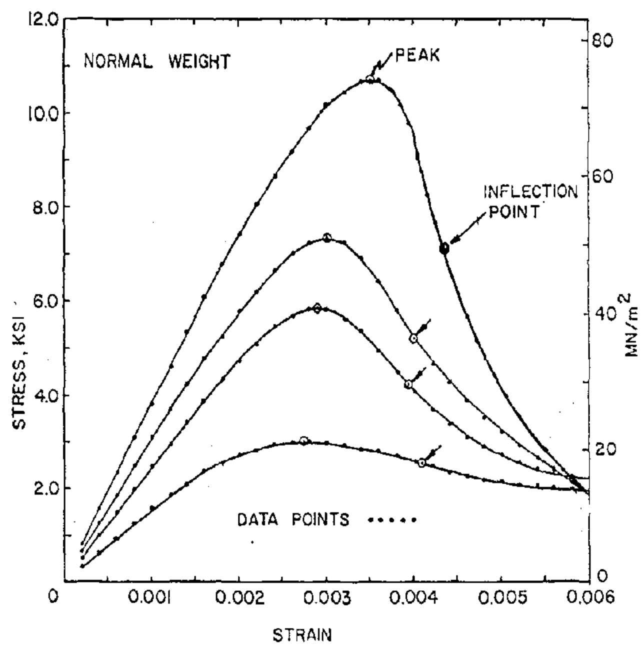


Fig. 3 Typical stress strain curves of same concrete tested at different ages (Ref. 5)

		Nonlinear Analysis			ACI Code		
Section Shape	Reinf. Ratio	ε_{cu}	M_u K.in.	$\phi_u x10^{-3}$ Rad/in.	ε_{cu}	M_u K.in.	$\phi_u x10^{-3}$ Rad/in.
Rectangular	$0.50\rho_b$	0.0044	2960.	0.849	0.003	2900.	0.577
	$0.75\rho_b$	0.0028	4129.	0.367	0,003	4054.	0.372
T	$0.50\rho_b$	0.0043	11055.	0.837	0.003	11009.	0.646
	$0.75\rho_b$	0.0027	14472.	0.318	0.003	14120.	0.303
Triangular	$0.50\rho_b$	0.0084	340.	2.083	0.003	377.	0.709
	$0.75\rho_b$	0,0036	478.	0.747	0.003	528.	0.579

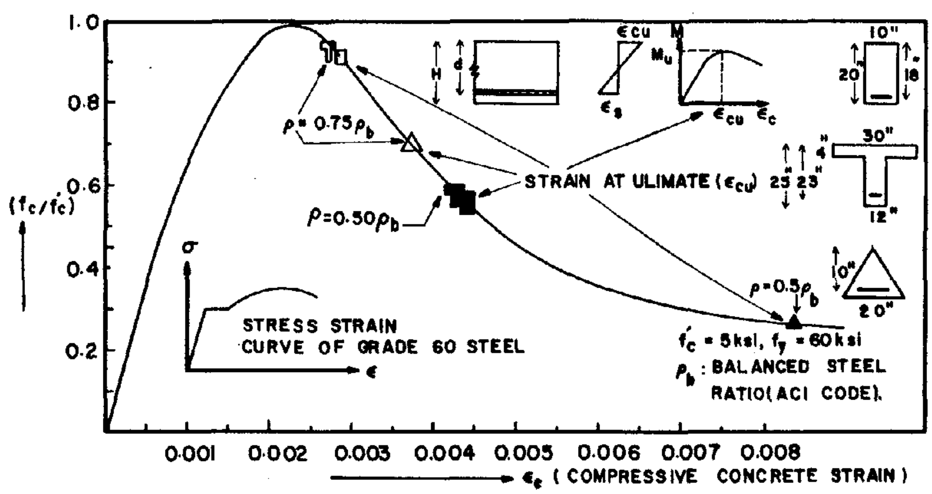


Fig. 4 Effect of cross sectional shape and reinforcement ratio on concrete strain ε_{cu} at ultimate Ref. (3)

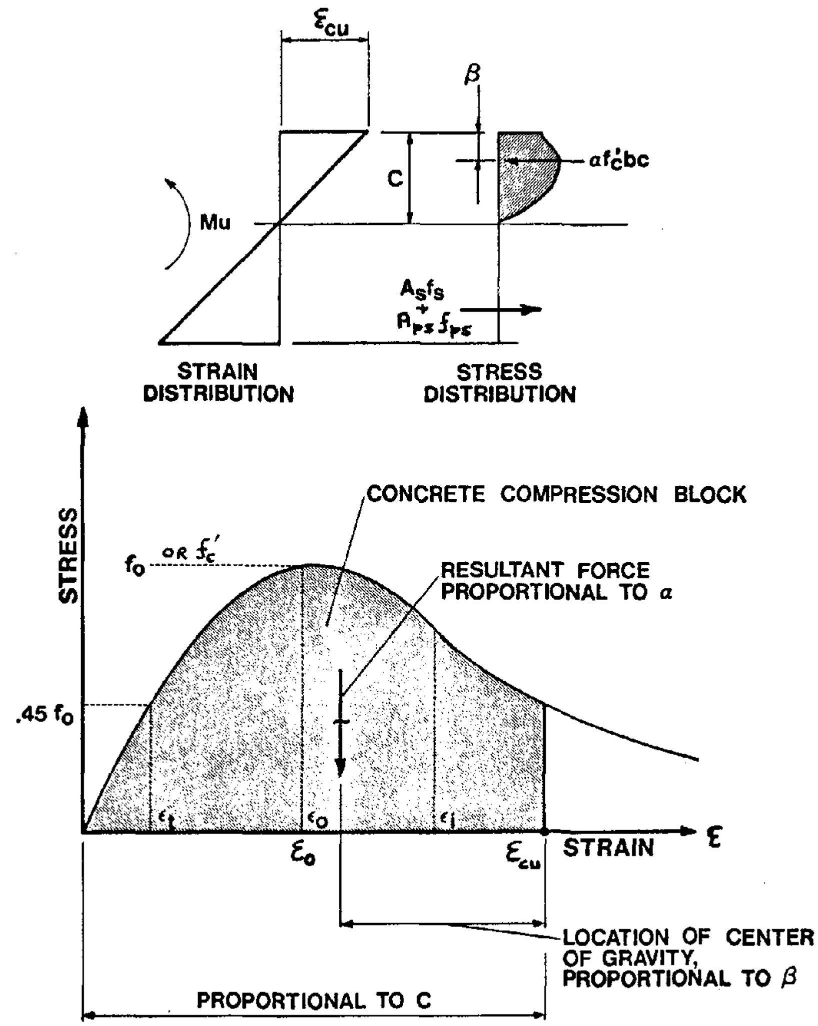


Fig. 5 Non identity of the peak strain of the concrete material ε_o and the strain at ultimate capacity of the member ε_{cu}

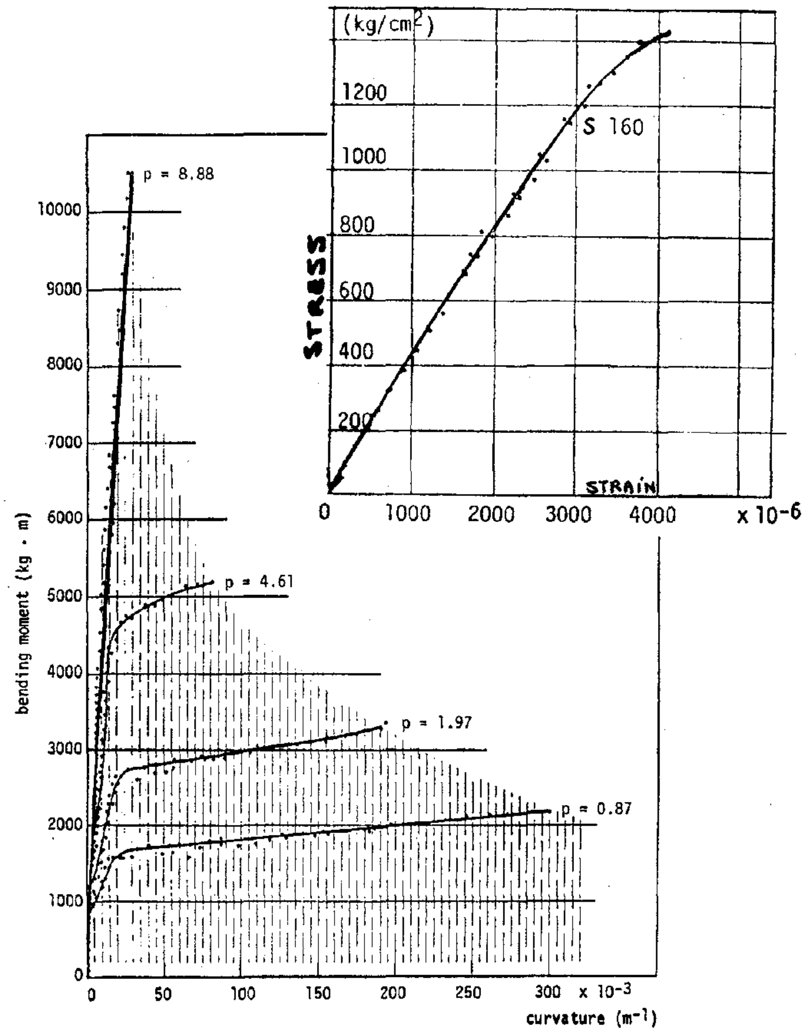


Fig. 6 Curvature versus bending moment for beams made with 160 MPa (23 ksi) concrete. (Ref. 7)

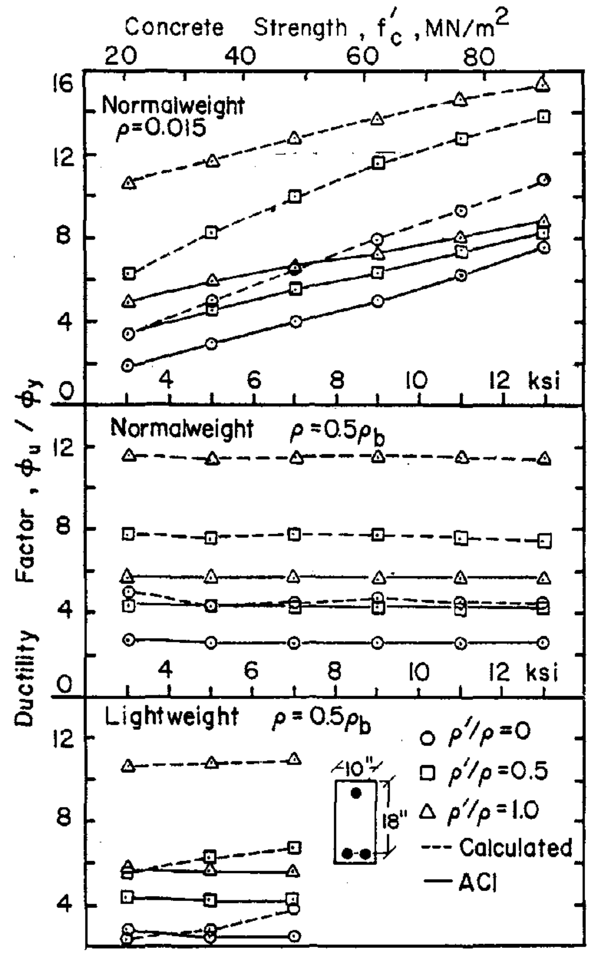


Fig. 7 Ductility ratios versus f'_c for reinforced concrete beams (Ref. 8)

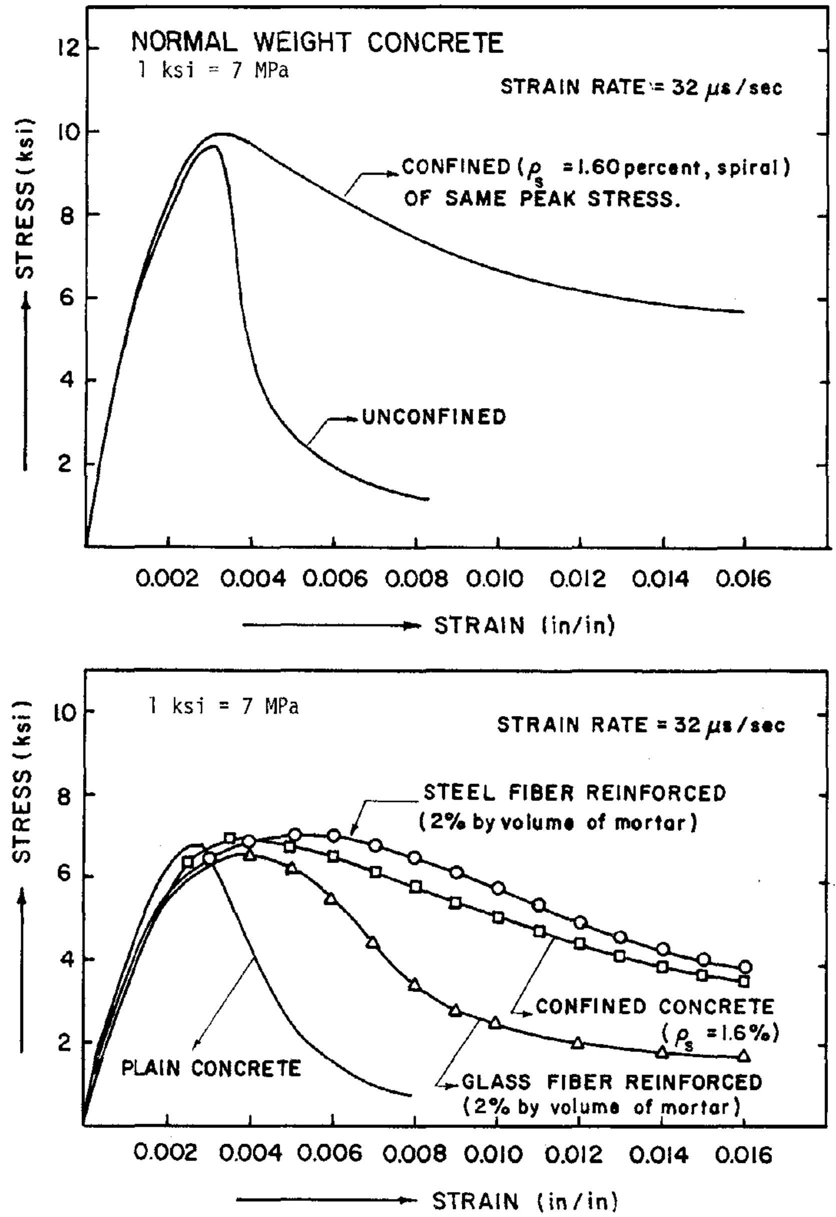


Fig. 8 Effect of confinement and fiber reinforcement on the ductility of concrete.

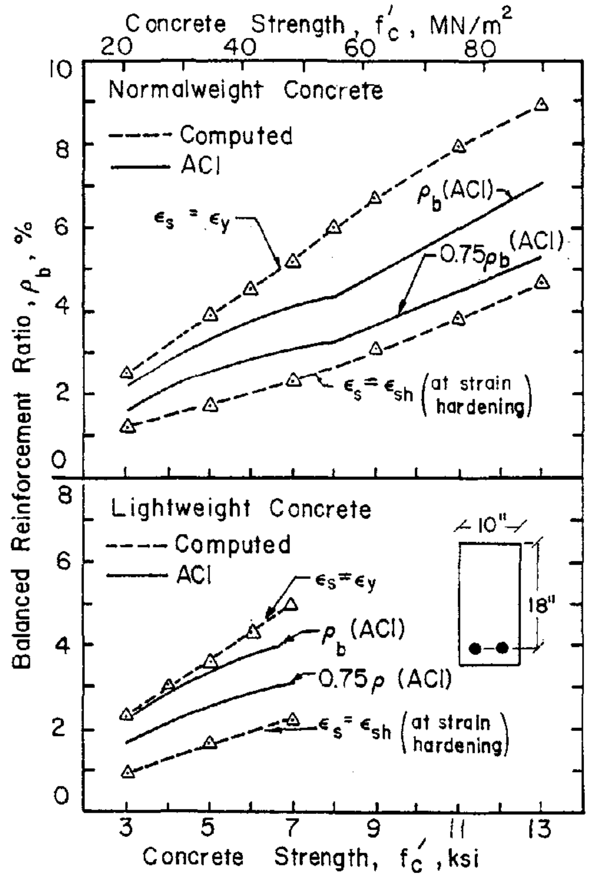


Fig. 9 Balanced reinforcement ratio versus f'_c (Ref. 8)

SESSION IV - SUMMARY OF FLOOR DISCUSSION

STRUCTURAL DESIGN CONSIDERATIONS FOR HIGH STRENGTH CONCRETE

by

Arthur H. Nilson

Department of Structural Engineering
Cornell University
Ithaca, New York

The value of the workshop in general, and Session IV in particular, was greatly enhanced by the comments from practicing engineers with experience in the production and use of high strength concrete, as well as by the airing of different points of view by researchers and academics. Invited discussions were presented by two engineers working in the Chicago area.

Sherwin P. Asrow of S. P. Asrow Associates described the use of high strength concrete in tall buildings in the Chicago area, designed by his firm in the past decade with the First National Bank building and with Lake Point Tower as early examples. The latter project, designed in collaboration with William Schmidt, attained a height of 645 feet with 70 stories, and made use of concrete having compressive strength of 7500 psi (52 MPa). The newest building, Watertower Place, having 76 stories, is presently the world's tallest all-reinforced concrete building. Mr. Asrow described Watertower Place in some detail, noting that, in the columns of the 100 x 200 foot tower, 9000 psi (62 MPa) concrete was used, with a steel ratio of 8 percent. The same concrete was used in the main girders of the theatre area. In many parts of the structure 4000 psi (28 MPa) lightweight concrete was used in conjunction with 9000 psi (62 MPa) normal density concrete. In spite of marked differences in shrinkage and creep behavior of the two materials, no difficulty was experienced. Commenting on the mix used for the high strength concrete, Mr. Asrow noted a maximum water-cement ratio of 0.40 and use of 100 pounds of fly ash per cubic yard.

Jaime Moreno of Materials Service Corporation described the experiences of his firm in providing concrete with compressive strength of the order of 9000 psi (62 MPa) for the past eight years. He stressed the need for close collaboration between the supplier, contractor, and engineer. He offered as a possible definition that high strength concrete be considered as the maximum possible strength concrete that can be obtained in a particular area optimizing the use of available material. Thus what is high strength concrete in Des Moines may be only ordinary strength concrete in Chicago. Accordingly, he stressed the need to develop a research program on how to optimize the use of materials. In his opinion, the use of fly ash is mandatory in order to provide the required slump, particularly necessary for placement in columns. In addition, the fly ash provides free lime for the mix. Mr. Moreno noted that high strength concrete is only a small part of the production of his firm. However, its experience with the closer quality control required for high strength material has resulted in a reduction in the coefficient of variation for production of ordinary concrete, the reduction being from 12 percent to about 7 or 8 percent. Present attention is being given to the role of superplasticizers. In economic terms, Mr. Moreno noted that the present price of normal strength concrete is about $40 per yard in the Chicago area, compared with about $62 per yard for the high strength material; however the ratio of strength gain is higher than the ratio of cost increase. In conclusion, he observed that concrete with strength of 9000 psi (62 MPa) is now accepted, and engineers he works with are developing confidence in concrete having strength as high as 11,000 psi (76 MPa).

The discussion continued with comments from the floor. David W. Fowler described the performance of polymer-impregnated post-tensioned I beams tested in his laboratory in Texas. The I beams were characterized by high compressive strength and low creep. Compressive strength from 15,000 to 20,000 psi (103 to 138 MPa) was reported, with creep about one-tenth the ordinary amount.

David Manning discussed certain applications of high strength concrete in highway bridges in Ontario. He noted present practical limitations based on economics, durability, and analytical difficulties. Relative to economics, he chose the example of a post-tensioned prestressed voided bridge superstructure. The design chosen was far from the lightest cross section in terms of concrete weight per foot of span, yet it was selected because forming and other construction advantages permitted the lowest cost. With respect to durability, Canadian engineers find it necessary to incorporate air-entraining agents in the concrete mix because of conditions of exposure. However, the use of air-entrainment inevitably leads to low strength, not high strength material. Present methods of analysis are felt to be inadequate to utilize the high strength concrete.

Henry Russell spoke on the subject of research needs for high strength concrete in buildings. He introduced certain practical problems of interest, such as the use of normal strength concrete in flat plate slabs together with high strength concrete in the slab in the immediate vicinity of the columns. Given that the high strength concrete is necessary at the slab-column joint in order to transmit the heavy vertical column loads, can the engineer then take advantage of the higher concrete strength in reviewing the adequacy of the slab in shear and negative bending at the column? Some concern had been expressed relative to the generation of excessive heat in the concrete while curing. Mr. Russell reported measurements of temperatures up to 150 degrees fahrenheit without detrimental effect. An additional point was made concerning the age-strength relation. Prefacing his remarks with the note that there are substantial differences between the age-strength curves for concrete at the site, in the plant, and in cores, he noted that the contractor sees real advantage in rapid strength gain, as this permits early removal of forms and avoidance of reshoring. Mr. Russell implied that rapid strength gain is associated with high strength concrete, although there was some dissenting opinion from the floor. In conclusion, he noted that there may be many reasons for using high strength concrete, including not only (a) its high strength, but also (b) its low creep characteristics, (c) its high early strength, (d) low deflection of members resulting from high elastic modulus, and (e) reduced loss of prestress force because of lower creep deformation.

V. Ramakrishnan described experimental research in South Dakota investigating the effectiveness of superplasticizers. With such admixtures, water-cement ratios as low as 0.28 have permitted 8 inch slump, and have yielded concrete compressive strengths in the range from 12,000 to 15,000 psi (83 to 103 MPa). The main interest in the program was durable concrete for bridge deck replacement, and the high strength attained was only incidental. Fiber reinforcement was added for toughness.

Zdenek Bazant commented on earlier discussion by Antoine E. Naaman in which Dr. Naaman had stated that sudden failure of a uniaxial compressive test specimen would be obtained when the stiffness of the machine is less than that of the test specimen. Professor Bazant observed that this is an over-simplification that works only for very small specimens. Actually one must consider the combined stiffness of the machine and that part of the specimen that unloads, and compare this combined stiffness with that of the part that fails. This point was an elaboration of a similar consideration introduced earlier by Professor Arne Hillerborg in discussing the importance of gage length and gage placement in defining the total shape of the stress-strain curve.

SESSION V - SUMMARY OF FLOOR DISCUSSION

by

Francis J. Young
Department of Civil Engineering
University of Illinois
Urbana, IL 61801

Henry G. Russell
Structural Development Department
Portland Cement Association
Skokie, IL 60077

DISCUSSION

The general theme developed during the concluding session was that research at all levels should be encouraged, but that practice cannot wait until research provides the answers. High strength concrete is being used now and will continue to be used, but, as Mr. Asrow pointed out, we must continue to study the material to determine its limitations and prevent wrong applications. Professor Diamond and Dr. Pomeroy pointed out that we can learn from applications in other areas, such as spun pipe or blast-resistant structures.

Several speakers did however address specific research needs which can be conveniently summarized under three main headings:

1. Materials Development
2. Micro- and Macro-Mechanics
3. Material Behavior

Materials Development

Dr. Frohnsdorff made a plea for research into methods of characterization of powder materials, e.g., cement and fly ash, which are used in high strength concrete. Until we can adequately characterize these materials, we cannot adequately predict the performance of the concrete which they help to create. This was supported by Professors Young and Diamond who pointed out that different cements have widely differing potential strength development, perhaps as much as a factor of two for Type I cements. The optimum cement can only be determined by laboratory testing. Dr. Pomeroy drew attention to the possibilities of new powder materials such as silica fumes which offer the possibilities of extending the limits of high strength concrete.

Professor Young also pointed out that total porosity and hence water/cement ratio is the dominant factor controlling strength in pastes probably up to 15,000-20,000 psi. Beyond that, "fine-tuning" would be achieved by control of microstructure to increase the strength at a given water/cement ratio. With use of superplasticizers which have been extensively used abroad, there is presently a potential to achieve 20,000 psi by reduction of water/cement ratio alone. However, Mr. Asrow observed that in many areas of the country high strength concrete is limited by the properties of the available aggregate for concrete.

Micro- and Macro-Mechanics

The interaction of both aggregate and paste comes back to the fact that we must understand the role of the two components in order to model the material along the lines suggested by Professor Wittman. Such modelling should provide principles which can allow the design of high strength concretes with less favorable aggregates. This is a present need. Mr. Asrow noted that it has not been possible to exceed 6000 psi using less desirable aggregates as found in the Minneapolis area. Future research should therefore consider all geographic areas.

Several overseas attendees indicated the highest strength levels being used in their country. In Sweden and Germany, strengths up to 10,000 psi are used for precast concrete with lower strengths for cast in place construction. In Japan, a prestressed concrete railway bridge has been built with 10,000 psi concrete. However, concrete with a compressive strength of 6,000 psi was more usual. The British Code has provisions for 10,000 psi in prestressed concrete but this strength is not achieved at present.

Professor Brown noted that we will need to introduce bond characteristics into our analytical models and research will be needed to improve our knowledge of bond performance. Also, the effect of smooth cracks on the behavior of large beams, such as transfer of shear stresses, is a facet of macro-mechanics that must be addressed.

Professor Uzumeri expressed the belief that eventually we may be able to develop continuum mechanics from the micro-level, even though that day is far off. In the meantime, overall behavior should be investigated for structural design. The purpose of testing sub-assemblages was to determine the mechanism of behavior and to verify mathematical models. Professor Dougill observed that moving from the micro- to macro-level will involve putting together sub-assemblies that have been studied separately. This approach can take us right up to complete buildings. However, at all stages, we must be careful to deal with stable elements because the association of unstable sub-assemblies will eventually lead to collapse.

Material Behavior

Several speakers reiterated the importance of properties other than strength; durability in particular. Dr. Manning emphasized that the relationship between high strength and high quality may make high strength concrete attractive not for its strength, but for its long term service performance. Dr. Pomeroy suggested that we should consider durability during the mix design stage. For example, the high cement contents that are typical of high strength concretes could conceivably cause alkali-aggregate problems at alkali levels normally considered safe.

Professor Darwin pointed out that in testing engineering behavior of concrete we should take more care to make sure that we test what we mean to test. For example, we should try to reduce the biaxial end restraint in the uniaxial compression test and avoid edge restraints in multiaxial testing. This observation was perhaps prompted by earlier discussions concerning the inherent limitations and approximations of some present tests and their effects on interpreting the behavior of high strength concrete in structural components.

CONCLUSION

The lively discussion that characterized this final session was ample evidence of the appropriateness of the topic for bringing together diverse specialists to address a common interest. There appeared to be an improved awareness of the potential contributions that can be made at each level or sub-assembly in the applications of high strength concrete. The meeting should result in some new directions of research activity hitherto neglected. However, its most important contribution is the opening of a dialogue between individuals and a better appreciation of the problems that are of concern and the possible pitfalls that must be avoided or eliminated.

LIST OF ATTENDEES

S. K. Arya
Department of Applied
Mech. & Engrg. Sciences
University of California
La Jolla, California 92037

S. P. Asrow
Asrow Associates
224 S. Michigan Avenue
Suite 1509
Chicago, Illinois 60604

Robert Ayre
National Science Foundation
1800 G Street
Room 419
Washington, D.C. 20550

M. A. Azimi
Dept. of Civil Engineering
Kyoto University
Yoshida Hommachi Sakyo-Ku
606 Kyoto
Japan

P. Balaguru
Rutgers University
Dept. of Civil & Environmental
Engineering
P. O. Box 909
Piscataway, New Jersey 08856

Z. P. Bazant
Dept. of Civil Engineering
Northwestern University
Technological Institute
Evanston, Illinois 60201

Richard L. Berger*
University of Illinois
at Urbana
3211 Civil Engineering Bldg.
Dept. of Civil Engineering
Urbana, Illinois 61801

V. Bertero
University of California
Dept. of Civil Engineering
783 Davis Hall
Berkeley, California 94720

D. Branson
University of Iowa
Materials Engrg. Division
Iowa City, Iowa 52240

C. Brown
University of Washington
Dept. of Civil Engineering
223 More Hall
Seatle, Washington 98195

Luigi Cedolin
Instituto di Scienza E
Tecnica
Delle Costruzioni Del
Politecnico di Milano
Plaza Leonardo di Vinci, 32
20133 Milano, Italy

K. Chao
Hydraulic Engineering Dept.
Dalian Institute of Tech.
Dalian 116024
China

Wai-Fah Chen
School of Civil Engineering
Purdue University
West Layfayette, Indiana 47907

James Clifton
National Bureau of Standards
Building Composites Group
Center for Building Research, NEL
Building 226, B340
Washington, D.C. 20234

M. Z. Cohn
Department of Civil Engineering
University of Waterloo
Waterloo, Ontario N2L 3G1
Canada

David Darwin
Department of Civil Engineering
University of Kansas
Lawrence, Kansas 66045

Sidney Diamond
School of Civil Engineering
Purdue University
West Lafayette, Indiana 47907

John Dougill
Department of Civil Engineering
University of London at
Kings College
Strand, London WC2R 2LS
England

L. T. Eby
United States Gypsum Company
1000 E. Northwest Highway
Des Plaines, Illinois 60016

Mark Fintel
Portland Cement Associates
5420 Old Orchard Road
Skokie, Illinois 60076

David Fowler
University of Texas
Department of Civil Engineering
Austin, Texas 78712

Geoffrey Frohnsdorff
National Bureau of Standards
Building 226, Room B-248
Washington, D.C. 20234

Hans Gesund
University of Kentucky
Department of Civil Engineering
Lexington, Kentucky 40506

K. Gerstle
University of Colorado
Department of Civil Engineering
Boulder, Colorado 80309

S. K. Ghosh
Portland Cement Associates
5420 Old Orchard Road
Skokie, Illinois 60076

Odd. Gjorv*
Department of Civil Engineering
University of Trondheim, NTH
N-7034 Trondheim-NTH
Norway

Arne Hillerborg
Div. of Building Materials
Lund Institute of Technology
Box 725
S-200 07 Lund
Sweden

Brian Hope
Department of Civil Engineering
Queens University
Kingston, Ontario K7L 3N6
Canada

Thomas Hsu
University of Miami
Department of Civil Engineering
P. O. Box 8294
Coral Gables, Florida 33124

Anthony Ingraffea
Cornell University
School of Civil & Environmental
Engineering
Hollister Hall
Ithaca, New York 14853

Daniel Jenny
Prestressed Concrete Institute
20 N. Wacker Drive
Chicago, Illinois 60606

James Jirsa
University of Texas
Dept. of Civil Engineering
Austin, Texas 78712

Luc Lachance
Dept. of Civil Engineering
Laval University
Quebec City G1K 7P4
Canada

M. Maage
Dept. of Civil Engineering
Purdue University
West Lafayette, Indiana 47907

D. Manning
Ministry of Transportation
and Communications
1201 Wilson Avenue
Downsview, Ontario M3M 1J8
Canada

Jaime Moreno
Material Service Corp.
300 W. Washington Street
Chicago, Illinois 60606

Antoine E. Naaman
Dept. of Materials Engineering
University of Illinois
P. O. Box 4348
Chicago, Illinois 60680

T. Nagafuchi
Denki Kagaku Kogyo K.K.
Omi-machi Nishikubi Kigun
Niigata Pref.
Japan

Dan Naus
Oak Ridge National Lab.
P. O. Box Y
Oak Ridge, Tennessee 37830

V. Nicolayeff (K. Cheung)
General Atomic Company
P. O. Box 81608
San Diego, California 92138

Arthur Nilson
Dept. of Structural Engineering
Cornell University
324 Hollister Hall
Ithaca, New York 14850

Yoshihiko Ohama*
Dept. of Architecture
Nihon University
Koriyama, Fukushima-Ken 963
Japan

D. J. Parrott
University of Illinois
3230A Civil Engineering Bldg.
Dept. of Civil Engineering
Urbana, Illinois 61801

Edward Pfrang
National Bureau of Standards
Room 8386, Building 226
Washington, D.C. 20234

K. S. Pister
Dept. of Civil Engineering
University of California
Berkeley, California 94720

C. D. Pomeroy
Cement & Concrete Assoc.
Wexham Springs
Slough SL3 6PL
England

Sandor Popovics
Drexel Institute of Tech.
Dept. of Civil Engineering
Philadelphia, PA 19104

V. Ramakrishnan
Dept. of Civil Engineering
South Dakota School of Mines
and Technology
Rapid City, SD 57701

Francisco Robles
Departmento de Materiales
Universidad Autonoma
Metropolitana UAM
San Pablo s/n
Azcapotzlaco 16, D.F.
Mexico

Henry Russell
Portland Cement Assoc.
5420 Old Orchard Road
Skokie, Illinois 60076

G. Schickert
Bundersantalt fver
Materialprufung (BAM)
Uner den Eichen 87
D-1000 Berlin 45
West Germany

William Schnobrich
University of Illinois
Dept. of Civil Engineering
Urbana, Illinois 61801

Charles Scribner
University of Illinois
Dept. of Civil Engineering
3148 Civil Engineering Bldg.
Urbana, Illinois 61801

S. P. Shah
Department of Materials Engrg.
P. O. Box 4348
University of Illinois
Chicago, Illinois 60680

J. Skalny (W. Klemm)
Martin Marietta Lab.
1450 S. Rolling Road
Baltimore, Maryland 21227

Floyd Slate
Cornell University
173 Hollister Hall
Dept. of Civil Engineering
Ithaca, New York 14853

Shan Somayaji
University of Pittsburgh
Dept. of Civil Engineering
949 Benedum Hall
Pittsburgh, PA 15261

Harry Stavrides*
Department of Buildings
City Hall, Room 900
121 N. LaSalle Street
Chicago, Illinois 60602

Stein Sture
Dept. of Civil Engineering
Virginia Polytechnic and
State University
Blacksburg, Virginia 24061

Michael Taylor
Dept. of Civil Engineering
University of California
Davis, California 95616

S. Uzumeri
University of Toronto
Dept. of Civil Engineering
Toronto M5S 1A4
Canada

Michael Ward
University of Calgary
Dept. of Civil Engineering
Calgary, Alberta T2N 1N4
Canada

Douglas Winslow*
Purdue University
School of Civil Engrg.
West Lafayette, Indiana 47907

Folker Wittmann
Delft University of Tech.
Dept. of Civil Engineering
Stevinweg 1
P. O. Box 5048
2600 GA Delft
Holland

Francis J. Young
Dept. of Civil Engineering
University of Illinois
3209 Civil Engineering Bldg.
Urbana, Illinois 61801

Robert Yuan
Dept. of Civil Engineering
University of Texas
Arlington, Texas 76010

Paul Zia
Dept. of Civil Engineering
North Carolina State University
P. O. Box 4993
Rayleigh, NC 27650

*In Absence