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Shanghai Research Institute of Building Sciences

1985

Final Report

Assignment to Shanghai (11 August - 31 August, 1985)
Acct. No. DP/CPR/81/026/11-55

Submitted by
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Department of Civil Engineering
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I was assigned by UNIDO to technically assist Shanghai Research Institute of Building Sciences. On arrival in Beijing, I met Mr. Zhou Qin and Mr. Gu Zhang-Zhao of the Shanghai Research Institute of Building Sciences (SRIB) who accompanied me to Shanghai. In Beijing, I also met Mrs. Pan Xue Wen of the China New Building Materials Corporation. Mrs. Pan came with me to Shanghai and very capably interpreted all my lectures as well as all the technical discussions in Shanghai. Her familiarity with the technical subjects (she spent 2 years at Northwestern University doing research under my supervision) and her excellent knowledge of English were invaluable for this assignment. Mr. Zhou Qin was an extremely helpful and hard working general interpreter. His assistance to my wife and me made our trip to China a very enjoyable one.

In Beijing, I gave a lecture on Fiber Reinforced Concrete at the China Building Materials Institute. We also discussed their very useful research on Glass Fiber Reinforced Concrete Panels.

At SRIB, the director Dr. Wang Pu introduced me to the Shanghai Municipal Construction Committee as well as the leaders of various research teams at the Institute. I had lengthy and fruitful technical discussions especially with the following key people at SRIB: 1) Dr. Wang Pu, 2) Mr. Lu Ji-guang, 3) Mr. Li Tie-Liang, 4) Mr. Shen Dan-Sheng, 5) Mr. Gu Zhang-zhao, 6) Mr. Hu Shao-long, 7) Mr. Chen Jin-an and 8) Mr. Zhou Jia-zheng.

At SRIB I gave several lectures and conducted several technical discussions. Mr. Zhou Jia-Zheng distributed copies of my relevant publications before each lecture/discussion session which was helpful to the participants. He also duplicated about 200 of my technical slides for their future use. The participants at these lecture/discussion sessions included not only researchers from SRIB but from 26 other institutes in China (see the attached list of organizations which were represented at my lectures).

The lectures and the subsequent discussions covered the following topics: 1) high strength concrete (normal weight and light weight), 2) fiber reinforced concrete, 3) fracture mechanics, 4) new materials in concrete construction, 5) constitutive modeling of concrete, 6) fatigue of concrete structures, 7) impact loading of concrete, 8) offshore structures, 9) earthquake and wind design.

The discussions that followed my lectures showed a keen interest and an acute desire to seek knowledge on the part of the participants. Based on my visit I can make the following recommendations.

Recommendations

1. SRIB is serving a very vital function of helping and guiding the building construction community of Shanghai (and other parts of China) to modernize their construction and design techniques. Their excellent research has already helped innovative construction of several high rise buildings. SRIB is essential in assuring that the materials and energy resources of China are used most efficiently and economically for the rapidly increasing building construction of Shanghai.

2. In order to acquire the most up-to-date research know-how, their researchers should spend a year or more at the Universities in the U.S. At Northwestern University we will be glad to have researchers from SRIB conduct joint research in the topics mentioned in my lectures (see above).

3. The current research at Northwestern University which is relevant to needs at SRIB include: 1) offshore construction, 2) fatigue of concrete structures, 3) imnpact, earthquake and wind loading, 4) fracture mechanics, 5) fiber reinforced concrete and 6) high strength concrete.

4. SRIB should be helped so that they can acquire state-of-the-art equipment.

5. Research cooperation between SRIB and Northwestern University's Center for Cement, Concrete and Geomaterials should be established by visits from SRIB researchers to NU and vice-versa.

6. SRIB should be aided so that they can expand their library-acquisitions.

In summary, I found my visit to SRIB enjoyable and worthwhile. I hope that we can continue with the help of the UN the cooperation between NU and SRIB.

上海市建筑科学研究所
SHANGHAI RESEARCH INSTITUTE OF BUILDING SCIENCES

In addition to engineers from Shanghai Resrach Institute of Building Sciences, following institutions sent their representatives to attend Prof.S.P.Shah's lectures in Shanghai:

1. Scientific and Technological Commission of Shanghai Municipal Construction Committee
2. East China Design Institute
3. Shanghai Design Institute for Civil Buildings
4. Shanghai Urban Construction Colledge
5. Shanghai Research Institute for Housing Management Technique
6. Shanghai Research Institute for Municipal Works
7. No.9 Design Institute of China Ship Corporation
8. Shanghai Garden Design Institute
9. Research Institute for No.3 Bureau of Navigational Matters
10. Shanghai University of Industry
11. Design Division of Shanghai Branch of China Acamemy of Sciences
12. Design Division of Shanghai Municipal Education Bureau
13. Shanghai Design Institute for Light Industry
14. Shanghai Yiao Hua Glass Plant
15. Shanghai No.7 Plastic Plant
16. Shanghai Sparetime School for Civil Engineering
17. Shanghai No.1 Construction Corporation
18. " No.2 " "
19. Shanghai No.8 " "
20. Resisential Building Construction Company of Chang

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Ning District

21. No.1 Construction Corporation of No.3 Bureau of Navigational Matters
22. Nanjing Building Engineering Colledge
23. Nanjing Engineering Institute, Civil Engineering Department
24. Jiangsu Research Institute for Building Sciences
25. Zhejiang Building Design Institute
26. Shuzhou Concrete & Cement Products Research Institute

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TECHNICAL ABSTRACTS

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The summaries of the following topics covered by Professor S. P. Shah during his visit to Shanghai Building Research Institute, August 1985 are attached.

1. Post-Peak Tensile Response of Concrete
2. How to Specify Performance of Fiber Reinforced Concrete
3. High Strength Concrete
4. Rate Sensitivity of Mode I and Mode II Fracture
5. Constitutive Modeling for Dynamic Loading of Concrete
6. A Modified Instrumented Charpy Test For Cement Based Composites.
7. Test Methods for Impact Resistance of Fiber Reinforced Concrete
8. Properties of Steel Fiber Reinforced Concrete Subjected to Impact Loading
9. Mechanical Properties of Materials Subjected to Impact
10. A Strain Rate Sensitive Damage Model for the Biaxial Behaviour of Concrete
11. Concrete and Fiber Reinforced Concrete Subjected to Impact Loading
12. Application of Fracture Mechanics to Cementitious Composites
13. Parameters for Constitutive Modeling
14. Orthotropic Model for Complete Stress-Strain Curves of Concrete Under Multiaxial Stresses
15. Crack Propagation Resistance of Fiber Reinforced Concrete
16. A Two Parameter Fracture Model for Concrete

POST-PEAK TENSILE RESPONSE OF CONCRETE

Failure of reinforced and prestressed concrete structures, in a majority of cases, is initiated by cracking of plain concrete. For example, in a reinforced concrete beam the failure is initiated by cracking of plain concrete in the tension zone of the beams. Once concrete is cracked, it is a common practice to ignore the resistance of cracked concrete. However, it has been known that plain concrete subjected to uniaxial tension has some post-cracking resistance. This post-cracking resistance has been often termed as the descending part of the concrete tensile stress-strain curve or the post-peak tensile response, or the strain-softening part of the response of concrete. Ignoring of the post-cracking resistance of concrete is not critical in determining the ultimate resistance of reinforced and prestressed concrete structures. However, recently it is increasingly realized that for rational calculations using nonlinear finite element analysis, it is necessary to include post-cracking resistance of concrete for accurate predictions of deflection, crack-width, bond transfer, and tensile stiffening contribution of concrete between the cracks.

The most commonly referred to complete tensile stress-strain curves are those by Evans and Maurate, as shown on these slides. These curves labeled 1 and 2 and shown by dotted lines, were obtained by testing concrete specimens in parallel with steel rods. This type of loading enabled the author to obtain the post-peak response and avoid the undesirable instability due to soft testing machines. On this same slide are also shown some recent tensile stress-strain curves obtained by Pickerson in Sweden. These curves are labeled 3 and 4 and are shown by solid lines. The comparison of the curves by Petersson were obtained by loading concrete specimens in a rigid testing machine, where the load was applied by increasing the temperature of the aluminum column parallel to the concrete specimen. A comparison of these two curves indicate a substantial discrepancy in many of the values. For example, Evans and Marate's modulus is much lower than Petersson. Their peak strains are also much larger and their optical crack widths are much smaller than indirectly calculated crack width of Petersson. The scope of the research being reported here is shown in the next slide. Our main purpose was to obtain reliable post-peak response of concrete specimens and from that obtain analytical equations which can be used for the finite element calculations. Specimens are being tested in our closed-loop 40 kip MTS servo-controlled testing system, as shown in the next slide.

We are measuring displacement at various gage lengths. We are also doing testing recording crack width measured optically, as well as recording strains along several points, both longitudinally and laterally. Specimens are loaded so as to maintain constant average displacement immediately across the notches, as shown in the next slide.

The purpose of providing a notch was to: 1) it assured that the crack will form at essentially predertimed plane and, therefore, we knew where to optically measure the crack width during the post peak ridging. The second purpose was to enable stable post-peak response For the feed-back control we chose the displacement right across the notch, since these displacements are generally the largest. We were able to obtain a very stable post-peak response. If one uses larger gage lentghs for the feed-back control, then one is not assured of stable response because that does not prevent the local strain right near the crack, from blowing up. Tension testing of weak brittle materials like concrete piers problem with the grids, which were solved by designing special grids as shown in the next slide. These were wedge type grids where the wedge was designed with steel. Next to it was aluminum, and next to it was rubber, and then the specimen. The overall testing set-up is shown in the next slide.

A typical result of concrete specimens are shown in this slide. The mix proportions of concrete specimens are also shown there. The water to cement ratio was .45 and specimens were tested after 28 day for moist curing. The results of both notched and unnotched specimens are shown. It was found that the notch did not influence the average response of the specimen. That is, the average stress versus average deformations were not influenced by the presence of the notch. However, when the specimens were not notched, it was not always possible to obtain a stable response of the specimen during the post-peak region. This is because the crack may form outside the gage length where the displacements are measured for the feed-back control. These specimens are not notched. Here the results of four specimens are shown, three of them were loaded monotonically, while one specimen was loaded in incremental strain cycling. It can be seen that the peak of the cyclic curves coincide with the monotonic curves and that means that the concept of envelope curve is also valid for concrete subjected to uniaxial tension. The concept of the envelope curve has been shown to be valid for concrete in uniaxial compression and it is a useful design tool for concrete subjected to seismic excitation. Note that the progressive degradation due to cyclic loading of concrete in uniaxial tension is also similar to that reported for uniaxial compression. In this curve, average stress was deformation as measured with a gage length of 3.25 in. as reported. Why we did not calculate strain from this deformation is shown in the next slide.

In this slide stress-strain curve of a single specimen of mortar is plotted. The strains are measured with strain gages along various points as well as strains as computed from displacement measured with 3.25 in. gage length are shown. It can be seen that in the post-peak region the measurement of strain depends upon the gage length. Thus, one cannot talk about tensile stress-strain curve in the post-peak region. Also, note that up to the ascending part, the curves, that is the stress-strain curves, are the same regardless of the gage length or the location of the strain gages. Thus, we think that it is preferable to talk about stress displacement curves for concrete rather than stress-strain curves. This definition provides a unique stress-strain curve, as shown in the next few slides.

In this slide the displacement as measured with a one-half inch gage length in abscissa are compared with the displacement as measured with 3.25 gage length in the ordinate. This comparison is for concrete specimen, mortar specimen and a paste specimen. If the strains were uniformly distributed, then all the data points should fall on a slope of 1 to 6.5. If, on the other hand, the strain were localized on a single line, then both of these displacements should be identical and they should fall on 1 to 1 slope. It can be seen, that as the displacement increases for all three materials, the slope is approximately equal to 1. However, only for concrete. The line is very close to the line equal to 1 to 1. One could conclude that at least for a concrete specimen, and to a large extent for mortar and paste specimens, in the post peak region, the displacement of the specimens are essentially the same regardless of the gage length. The fact that the large increase of nonlinear strain in the post-peak region is concentrated in a rather narrow zone, was also confirmed from optical crack measurement. This is shown in the next slide.

In the abscissa are plotted optical crack widths which were observed with the accuracy of about 25mm, are plotted against computed crack widths. Computed cracks are calculated as the difference between total measured displacement minus the elastic displacement. It can be seen that for concrete these two displacements are very close to each other. This his not true for paste and mortar. However, considering the percentage difference between the optical crack width and computed crack width, perhaps even for past and mortar, it could be assumed that these two are approximately equal.

Strain distribution across the specimen at various loads are shown in this slide. It can be seen that before the peak stress, strains are more or less uniformly distributed on both sides of the specimen, thus indicating that there was no bending. As beyond the peak stress as exptedted, the strains at the root of the crack are larger and eventually when the crack crosses a particular strain, the strain gages become inopperative, as shown by the dotted line. The vertical of the longitudinal strain distribution is shown in the next slide.

It can be seen that up to the peak stress, the longtidunal strain distribution is more or less uniform after the peak stress. Once the crack is formed the strain is localized much more near the crack while away from the crack the strain is in fact reduced.

Before the program started we had expected cement paste to behave brittley and not to exhibit significant post-peak response. This was not the case. Cement paste exhibited significant post-peak response, as shown in the next slide.

The apparent ductility of cement paste compared to mortar and concrete can clearly be seen in the next slide, where the average stresses in the post-peak region are plotted against optical crack width. It can be seen that cement paste can resist substantial stresses, even in the post-cracking range and they are quite comparable to concrete and mortar. The average tensile properties of concrete, mortar, and paste are compared in the next slide.

The important thing here is the last column where the ratio of tangent modulus versus secant modulus as peak is plotted. If this ratio was 1 then the material was linearly elastic. It can be seen that contrary to compression and contrary to prior oberservation up to the peak at least, paste and mortar are more nonlinear than concrete. A comparison between tension and compression is shown in the next slide.

Notice that the tangent model in tension is at least as gret as that in compression. Also note that the peak strain in tension is substantially less than that in compression and that the nonlinearity up to the peak as characterized by the ratio of tangent modulus to secant modulus at peak is greater in compression than in tension.

Finally, in the next slide are plotted some observations obtained from cyclic loading of concrete and mortar specimens. Plotted here are the residual displacement versus unloading displacement. That is, the displacement from the envelope curve from where the unloading commenced. A linear relationship is obtained. A similar linear relationship between residual displacement and the displacement where the unloading commences on the envelope curve, has also been obtained in compression.

In conclusion, it is possible to obtain stable post-peak response in a relative soft machine by using closed-loop testing system and by proper selection of the feed-back control. The stress-strain curve of specimens depend very much on the definition of strain that is on gage length. It is recommended that in order to obtain a unique curve it is better to use stress versus displacement relationship. If a sufficiently large gage length is used, then stress versus diaplacement relationship up to the ascending part up to the peak part are the same as the normal stress-strain curve. In the descending part stress versus displacement seems to be more or less independent of the gage length.

Conclusion #3. The nonlinear deformation in the post-peak region seems to be concentrated in a very narrow band. The width of this narrow band, also known as fracture process zone, does not seem to be related to the grain size as one might expect in a predicted manner.

HOW TO SPECIFY PERFORMANCE OF FIBER REINFORCED CONCRETE

by

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INTRODUCTION

For most structural material the most important quantity that is used for design and for which there are test specifications, is the strength of the material. For example, for steel structures, the tensile yield strength is the most routinely measured property of the steel. Similarly, for concrete structures, the most common property that is specified and used in design is its uniaxial compressive strength. Another commonly specified property for concrete is its modulus of rupture measured by testing beams. MOR is the one which is specified for the construction of concrete pavements. The addition of metallic mineral or organic fibers to concrete increases its toughness, ductility, or crack propagation resistance much more than its strength. This is explained in the next slide. Here load deflection curves of plain concrete and steel fiber reinforced concrete beams are plotted. When testing plain concrete, one generally observes that as soon as a crack propagates from the tension direction, the beam essentially fails into two. For fiber reinforced concrete beams, one also observes a tensile crack at about the same load as the maximum load observed for the corresponding plain concrete beam but, fiber reinforced concrete beams continue to resist load with increasing deflection even after the so-called first cracking. This is quantitively shown in the next slide where the deflection at first crack and deflection at the peak, or maximum load, are plotted for steel fiber reinforced concrete beams in the <u>next slide</u>.

The parameter in abscissa is a product of volume fraction of fibers and its aspect ratio. Both of these quantities have been shown to be important in defining the influence of fibers. It can be seen that the deflection at first crack is essentially independent whether you have fibers or not and in what quantity they are added or how long they are, or what are the diameters. However, the deflection at peak load increases with the addition of fibers and can be substantially greater than that at the first crack. (<u>Next slide</u>)

If the area under the load deflection curve from zero load in the ascending part to zero load in the descending part, is defined as fracture toughness then this slide shows that increasing the volume fraction of fiber substantially increases the toughness of fiber reinforced concrete beams. Note that increase in modulus of rupture is only slight, while the increase in toughness is of the order of magnitude. (<u>Next slide</u>).

The same observation regarding the performance of fibers can be made from this slide where uniaxial compressive stress-strain curves of plain concrete and concrete reinforced with different types of steel fibers are plotted. There is very little difference in compressive strength between concrete and fiber reinforced cocrete. However, the post-peak resistance is substantially influenced by the presence of fibers.

From these observations it is clear that for construction involving fiber reinforced concrete, specifying strength will not control the most important contribution of fibers. However, specifying toughness or post-cracking resistance has been difficult because it is a difficult quantity to define and measure. The problem of how to measure, define and specify this quantity called toughness, is currently being wrestled within two organizations, American Concrete Institute, and American Society for Testing and Materials. One method of specifying and measuring toughness that is being considered by both of these committees is shown in the next slide.

It is proposed that the toughness index be defined as an area under the load deflection curve measured up to a fixed deflection divided by the area under the load deflection curve up to first cracking of fiber reinforced concrete beams. The two load-deflection curves shown here are those for beams reinforced with a small amount of fiber and beams reinforced with a larger percentage of fibers. It can be seen that the toughness index as defined varies from about ten to thirty. There are several problems with this method. The post-peak load deflection curve is not independent of the size of the beam and the span of the beam. That is, the load deflection curve of the beam is not only dependent on the basic material property but also on the dimensions of the specimens, as well as on the method of loading: three point vs. four point loading. The second objection deals with the definition of first crack. It is assumed that the area labeled 1, that is, the area up to the first crack for fiber reinforced concrete beam, represents the toughness of plain concrete beams. This area, however, depends upon how one defines first crack strength. Whether it is defined as deviation from linearity or whether it is defined microscopally, gives different results. The third problem deals with the specimen testing machine interaction. Dependeing upon whether the specimens are tested in closed-loop and depending upon whether the load is imposed by controlling deflection, or load, one may get a different shape of the post-peak load deflection curve. If the loading is done carefully, one obtains a smooth, stable post-peak response even for plain concrete as shown in the next slide.

These specimens were loaded under closed-loop testing control so as to maintain a constant rate of the maximum tensile strain. It can be seen that if the toughness of plain concrete is taken simply as the area up to the peak load, then one underestimates the toughness of plain concrete. A second proposal to evaluate toughness is shown in the next slide.

Here the resistance of concrete and fiber reinforced concrete to repeatedly applied blows from a hardened steel ball is measured as a toughness. The steel ball is raised to a specified height and is dropped and this is done repeatedly until the diameter of the concrete cylindrical specimen increases by a certain amount. It has been observed that it requires 50 blows for plain concrete to reach this arbitrary state of damage, while it may require as many as 500 blows for fiber reinforced concrete. Note that the thoughness index defined by this method has the maximum value of ten, while as defined by the load deflection curve, the maximum value was around 30. This points out that both of these methods are very much dependent on the method of testing and may not be a true material property.

We are currently doing research at Northwestern University to better understand the fracture toughness behavior of fiber reinforced concrete with the goal of coming up with a more rational evaluation and specification of this important property. This research is being sponsored by a grant from the United States Air Force Office of Scientific Research to Northwestern University. The grant is being monitored by Lt. Col. Hokanson, who also happens to be in the audience. I am going to briefly summarize three different aspects of our research: 1) evaluation of toughness from uniaxial tensile tests, 2) evaluation of toughness by using modified instrumented Charpy tests, and 3) by using fracture mechanics concepts.

To better understand the post-peak response of concrete and fiber reinforced concrete subjected to uniaxial tension, we have developed a closed-loop testing arrangement. This is shown in the <u>next slide</u>.

Specimens are loaded in an MTS type servo-controlled hydrolic, closed-loop testing machine. During tests we are measuring strains with various gage length, displacement, as well as observing cracking using an optical microscope. To avoid failure within the grip, a specially designed frictional wedge type of grid is being used. <u>Next slide</u>.

To assure cracking at a predetermined location, we are using notches. These specimens are loaded so as to control average displacement as measured right across the notch. If the displacement measured from the larger gage length are used as a feed-back control of the machine, then you may not get stable post-peak response because displacement around the crack can still blow up. The measured response of the specimen is shown in the <u>next slide</u>.

Note that the displacement measured can be a different gage length than those used for the feed-back. Using this set-up, we have observed that in the post-peak region, the displacements are primarily due to widening of a single crack. As a result, there is not unique strain relationship once the cracking has occurred around the peak. During the post cracking region the value of strain would depend upon the gage length. However, a unique stress displacement relationshop has been observed. This we are using to characterize the performance of fiber reinforced concrete. Next slide.

Charpy tests have been used for metallic and polymer materials to measure indirectly toughness of the energy absorbed in breaking the specimens. Conventional Charpy impact machines are not suited for testing concrete and fiber reinforced concrete for a couple of reasons. With this conventional testing machine it is not possible to use relatively large size specimens required to test concrete and fiber reinforced concrete specimens. The second reason is the conventional Charpy test measured energy absorbed in breaking the specimen. It does not include the kinetic energy of the broken halves specimen or the energy absorbed in the testing machine. In addition, one does not get any idea of other parameters such as dynamic fracture toughness from the conventional Charpy test. As a result, we have modified a conventional Charyp test as shown in the next slide. Our modifications include the following: 1) a possibility of testing large size specimens with a span of about 10 inches, 2) we have instrumented both the striker and anvil. As a result, during the impact event it is possible to measure load versus time characteristics of the specimen. From these as well as from measurements of deflection, one can calculate load deflection curve of the specimen under impact loading.

From this one can calculate more accurately the energy absorbed in breaking the specimen. In addition, if the specimens are notched then one can also calculate stress intensity factor as well as velocity of crack propagation and other characteristics needed to identify resistance to cracking. Load deflection curves for two different steel fiber reinforced concrete specimens tested at the static rate and at the dynamic impact rate are shown in the next slide.

From this it was observed that the increase in modulus of rupture for steel fiber reinforced concrete was higher than for plain concrete. The energy as calculated from load deflection curve, is shown in the next slide.

It can be seen that compared to plain concrete, fiber reinforced concrete can have as much as about 100 times the energy absorbed to fracture. Note that in the dropping ball type impact test mentioned earlier one records an energy enhancement of only 10. This points out the need for accurately recording the toughness or the energy value.

Since the main contributions of the fiber is increasing the resistance of cracking of concrete, it would seem reasonable to measure fracture toughness of concrete using the principles of linear elastic fracture mechanics. Using these principles, the fracture toughness of metals have been measured and the methods of these measurements have been specified in the relevant ASTM specifications. Next slide

The fracture toughness in Mode I crack propagation is referred to as K_{1c} which is the critical stress intensity factor for crack propagation. Stress distribution around a singularity introduced by crack for a linear elastic material is shown in this slide. It is realized that at the tip of the crack the stresses cannot be infinite as predicted by these equations. For metals at the tip of the crack, because of local yielding, the stresses would be equal to the yield stress of the material. The zone around the crack tip where this happens is called the plastic zone. The test methods to evaluate K_{1c} for metals, are designed such that the length of this plastic zone is small compared to the length of the crack. As a result, this singularity dominated stress distribution can still be considered valid. Next slide.

One could analyze for the length of this plastic zone by calculating the stress intensity factor due to applied load away from the crack and that due to the so-called pinching or closing pressure resulting from the yielding. Attempts have been made to apply a similar model for fiber reinforced concrete where the closing pressure would result from fiber bridging forces. However, for fibers, the closing pressure is not constant as it is for yielding type of materials. In fact, the fiber bridging force depends on the crack opening or the slip, as is shown in the next slide.

In this slide using the tensile test set-up that I had described earlier, I have plotted the results for the post-peak region for a steel fiber reinforced concrete specimen. Stress versus crack opening or the slip between fiber and concrete are shown. It can be seen that the fiber bridging force decreases as the slip increases or as the crack opening displacement increases. Based on this consideration, we have developed a model to predict fracture toughness of fiber reinforced concrete. This model is shown in the next slide.

A given crack in a concrete matrix can be considered as if it is made up of three zones. Zone 1 is the traction free zone which is the classical Griffith crack. The second zone is where the fiber bridging forces are operative and the third zone is called the matrix process zone which results from nonlinearity at the tip of the matrix due to microcracking and aggregate interlock. Note that the forces in the fiber bridging zone depend on the crack opening and because of this coupling between the forces and the crack opening, one needs a nonlinear integral equation to solve this problem. In addition to the theoretical model we also have been investigating different types of fracture mechanic type of specimens. What we are aiming is to come up with a fracture mechanics evaluation, both experimentally and theoretically which is independent of the specimen type. We are testing three types of specimens. The first one is called double cantilever specimen. This type of specimen permits evaluation of fracture toughness for cracks of up to about 18 inches long. The second type of specimen is called double torsion and the third type of specimen is the notched beam specimen. During testing we measured load, deflection, crack opening, as well as the crack length, using a traveling microscope. Some results for concrete and fiber reinforced concrete are shown in the next slide.

We find out that fracture toughness cannot be expressed with a single term such as K_{1c} or G_{1c} but an R-curve, or a resistance curve. In this slide is plotted strain energy required for crack propatation with crack extension. Because of the fiber bridging forces, the energy to propagate a crack increases with crack length and when the crack is sufficiently long, it reaches a steady state value. Only for these large cracks one could apply conventional linear elastic fracture mechanics. Note also that even if R-curve were a unique material property, one may obtain different values of conventional K_{1c} or G_{1c}, depending upon the initial length of the notch and depending upon the geometry.

In the last slide are some R-curves for fiber reinforced concrete shown. The solid curves are those obtained theoretically for one type of fiber. Also shown are experimental curves. Our experimental curves and theoretical model seem to agree quite well. Note that compared to plain concrete the resistance of fiber reinforced concrete is substantially higher, as one would expect. Also, one measure of fracture toughness could be the steady state value of R-curve. If that is used as a measure for our types of fiber we observe that fracture toughness of fiber reinforced concrete is about 40 times that of plain concrete. This value seems reasonable considering other measurements of fracture toughness. In conclusion, I would like to say that it is possible to calculate fracture toughness of fiber reinforced concrete rationally and accurately, by using nonlinear fracture mechanics principles. However, the testing required for such measurements is more involved than routine compression or flexural testing. It will also be possible to obtain relatively specimen geometry independent method of evaluating fracture toughness from uniaxial tension tests. To obtain stable and reliable post-peak response one needs a closed-loop type of testing which again is more involved than more routine testing associated with cement and concrete.

The area under the load deflection curve is probably an acceptable measure of fracture toughness if the effects of specimen geometry and the interaction between specimens and testing machines are understood. For relative comparison purposes and for quality control, flexural testing methods may be quite appropriate. It should be noted that, in general, such methods will underestimate toughness of plain concrete and concrete reinforced with smaller amounts of fibers.

HIGH STRENGTH CONCRETE

by

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ABSTRACT

High strength concrete (uniaxial compressive strength, f_c' greater than 42 Mpa) is experiencing an increasing use and acceptance by designers and contractors for both reinforced and prestressed concrete construction. The first use of high strength concrete (f_c' = 7000 Psi = 49 Mpa) in buildings was in Lake Point Tower (Chicago) in 1965, and the first application in bridges was for Willow bridge (Toronto) in 1967. An excellent summary of buildings and bridges in which concretes of higher than normal strengths have been used is presented in a recent ACI committee report. The principal advantage of high strength concretes is their relatively greater compressive strength to unit cost, unit weight and unit volume ratios as compared to normal strength concretes. High strength concrete, with its greater compressive strength per unit cost, is often the least expensive means of carrying compressive forces. In addition, its greater compressive strength per unit weight and unit volume allows lighter and more slender members. Other advantages of high strength concrete include increased modulus of elasticity and increased tensile strength. Increased stiffness is advantageous when deflections or stability govern the design, and increased tensile strength is advantageous in service load design in prestressed concrete.

The current code for designing reinforced concrete structures as well as

for designing prestressed concrete structures are based on, among other things, experimental evidence of testing concrete structures of compressive strength less than 6000 psi. A logical question to ask is: "Can we use the current design code for concrete structures made with concrete with compressive strength considerably greater than 6000 psi"? In the current code, many design parameters affecting the strength and behavior of structural members are related empirically to the compressive strength of concrete. These empirical parameters have been established by both laboratory experiments and design experiments with concrete having compressive strength less than 6000 psi. Thus, one must question whether the current equations can be extrapolated to concrete with compressive strength considerably higher than 6000 psi.

The writer has been involved in a continuing investigation which has as one of its aims to answer the question that is just posed. The current ACI committee on high strength concrete is also investigating the applicability of the current ACI code to high strength concrete. Some of the current thinking of the ACI committee as perceived by the author is summarized in this report.

In this paper, experimental data on high strength concrete obtained by the author are reported. Based on these data as well as those reported by the other investigators, the author has proposed empirical relations to substitute some of the currently used relationships. Note that the details of the experiments are presented elsewhere. In this paper, the emphasis is on the results, comparison with the normal strength concrete, development of the empirical formulae and some discussion on the implication for structural design.

RATE-SENSITIVITY OF MODE I AND MODE II FRACTURE OF CONCRETE. Surendra P. Shah and Reji John, Department of Civil Engineering, Northwestern University, Evanston, Illinois 60201.

For rational and accurate analysis of concrete structures subjected to impact or impulsive loads the knowledge of crack propagation under such dynamic loads is essential. Cracks frequently propagate under Mixed-Mode (opening and sliding: Mode I and Mode II) conditions. The rate of loading effects on the mechanical properties have been attributed to the rate sensitivity of crack propagation. The strain-rate effects may be different in Mode I and Mode II fracture of concrete. Single Edge Notched Beams were subjected to varying rates of loading, to establish the Mode I Stress Intensity Factor, K_I vs. Crack Velocity, V, relationship for mortar and concrete. Impact tests were conducted using a Modified Instrumented Charpy Impact Test System. The rate of crack growth was obtained using brittle 'Krak Gages'. Beams with two notches on one edge are being tested at different rates of loading to obtain the rate-sensitivity of Mode II fracture.

Mode I test results lead to the following conclusions. (1) Slow (pre-peak) crack growth for concrete is larger than that of mortar, at a given strain rate. (2) Pre-peak crack growth decreases with increase in strain-rate. This could be the reason for decrease in pre-peak non-linearity at higher strain rates. Hence LEFM approach may be valid at high rates of loading. (3) Log K_I - Log V relationship is non-linear especially at the higher rates of loading.

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CONSTITUTIVE MODEL FOR DYNAMIC LOADING OF CONCRETE

By

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ABSTRACT

The constitutive properties of concrete under dynamic loading are necessary for the rational analysis of concrete structures subject to impact and impulsive loads. The constitutive model presented herein models microcracking through the use of a continuous damage parameter for which a vectorial representation is adopted. The rate of increase of the damage is dependent on the state of strain as well as on the time rate of strain. The constitutive equations are derived from the strain energy function which is influenced by the accumulated damage. The constitutive model is calibrated using uniaxial tension (or flexural) and uniaxial compression test data. The calibrated model is then used to predict certain other load responses of concrete.

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A MODIFIED INSTRUMENTED CHARPY TEST FOR CEMENT BASED COMPOSITES

BY

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and Reji John*

Abstract

A description is given of a modified instrumented Charpy test that is designed to enable impact-testing of cementitious composites. Problems encountered in instrumented impact testing of such composites and solutions to overcome them are discussed. Results of tests on concrete specimens at four different impact velocities are reported and are used to evaluate the performance of the test set-up. A simple spring-mass model is used to verify the test results. This model is capable of providing suitable guidelines for the apriori selection of the basic test parameters with a veiw to minimize parasitic effects of inertial loading.

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Test Methods For Impact Resistance of Fiber Reinforced Concrete*

By W. Suaris[1] and S. P. Shah[2]

Synopsis: It is well accepted that fiber-reinforced concrete exhibits superior impact resistance than does plain concrete and numerous tests have been employed to evaluate its impact resistance. These include explosive tests and impact tests using projectiles and drop weights. Some of these tests and their results are described in this paper. Attempts to obtain more basic material parameters, by conducting instrumented impact tests, are described next and problems associated with the interpretation of their results are discussed. Finally a testing method developed by the authors, which appears to yield basic mechanical properties of fiber-reinforced concrete subjected to impact is presented.

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PROPERTIES OF STEEL FIBER REINFORCED CONCRETE SUBJECTED TO IMPACT LOADING

By

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ABSTRACT

The effect of strain-rate on the flexural behavior of unreinforced matrix and 3 different fiber reinforced concrete (FRC) mixes are discussed. Results obtained from the modified instrumented Charpy tests on cement composites compare well with results from several similar investigations that use an instrumented drop-weight set-up.

FRC mixes are more rate-sensitive than their respective unreinforced matrices, showing increases in dynamic (strain-rate of 0.3/s) strength of up to 111% and energy absorption (up to a deflection of 0.1 in., 2.5 mm) of up to 70% (V_f = 1.5%) over comparable values at the static (strain-rate of 1 x 10^{-6}/s) rates. Composites made with weaker matrices, higher fiber contents and larger fiber aspect ratios are more rate sensitive than those made with stronger matrices, lower fiber contents and smaller fiber aspect ratios. Several observations made in the study suggest that the rate sensitivity exhibited by such composites is primarily due to a change in the cracking process at the different rates of loading.

Relative improvements in performance due to the addition of fibers as observed in the instrumented tests are also compared to those from the conventional impact and static tests. Resulting from this comparison, it is observed that static flexural toughness tests may be used to approximately estimate the dynamic performance of FRC.

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MECHANICAL PROPERTIES OF MATERIALS SUBJECTED TO IMPACT

An Introductory Report for the Interassociation Symposium on

CONCRETE STRUCTURES UNDER IMPACT AND IMPULSIVE LOADING

Berlin (West), June 1982

Prepared by

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Summary

The development of computational models for predicting the behavior of structures subjected to impact loading requires the knowledge of constitutive relationships and failure criteria (i.e., mechanical properties) of the constituent materials at the high strain-rates caused by impact. In particular for concrete structures one needs to know the effects of varying strain-rate on the properties of steel, concrete and the interface between them.

Firstly, the report deals briefly with the strain-rate effects on the tensile behavior of reinforcing and prestressing steels. Then, available experimental results of strain-rate effects on concrete under tension, flexure compression and multiaxial stresses are presented; various material compositions and environmental effects are dealt with in detail. Some models proposed for the prediction of fracture strength dependence on strain-rate is presented next. Here, models with relevance to concrete such as the chemical reaction rate-process theory, stochastic theories and material inertia models are addressed. In addition, some proposed constitutive models for concrete, such as rheological models, porous media models and visco-plastic models are discussed. Finally, the properties of the concrete-reinforcement interface at high strain rates are summarized.

A STRAIN RATE SENSITIVE DAMAGE MODEL FOR THE BIAXIAL BEHAVIOUR OF CONCRETE

UN MODELE D'ENDOMMAGEMENT SENSIBLE AU TAUX DE DEFORMATION, POUR L'ETUDE DU COMPORTEMENT BIAXIAL DU BETON

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SUMMARY

A constitutive model, based on the continuous damage concept is presented. The continuous damage concept is well adapted to model the behaviour of concrete as the failure of concrete is accompanied by the growth of microcracks. The proposed continuous damage theory uses a vectorial representation for damage which is motivated by the flat nature of the microcracks. The constitutive equations and the damage evolution equations are derived from the strain energy function, in a thermodynamically consistent manner. The proposed model is calibrated by using experimental results in flexure and uniaxial compression. The results of the model are then compared with available experimental results on the biaxial behaviour of concrete.

RESUME

Nous présentous un modèle de comportement, basé sur le concept d'endommagement continu. Le concept d'endommagement continu est approprié pour modéliser le comportement du béton, étant donné que la rupture du béton donne lieu à la croissance de micro-fissures. La théorie d'endommagement continu qui est proposée, fait appel à une représentation vectorielle de l'endommagement, représentation qui est motivée par l'aspect plat des micro-fissures. Les équations de comportement et les équations d'évolution de l'endommagement sont démontrées a partir de la fonction énergie de déformation, et ceci, en accord avec la thermodynamique. Le modéle proposé est calibré à l'aide de résultats expérimentaux en flexion et en compression uniaxiale. Enfin, les résultats du modéle sant comparés avec les données expérimentales disponibles sur le comportement biaxial du béton.

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ABSTRACT

CONCRETE AND FIBER REINFORCED CONCRETE SUBJECTED TO IMPACT LOADING: Surendra P. Shah, Professor, Dept. of Civil Engineering, Northwestern University, Evanston, IL 60201.

Despite its extensive use, low tensile strength has been recognized as one of the major drawbacks of concrete. Although one has learned to avoid exposing concrete structures to adverse static tensile load, these cannot be shielded from short duration dynamic tensile stresses. Such loads originate from sources such as impact from missiles and projectiles, wind gusts, earthquakes and machine vibrations. The need to accurately predict the structural response and reserve capacity under such loading has led to an interest in the mechanical properties of the component materials at high rates of straining.

One method to improve the resistance of concrete when subjected to impact and/or impulsive loading is by the incorporation of randomly distributed short fibers. Concrete (or Mortar) so reinforced is termed fiber reinforced concrete (FRC). Moderate increase in tensile strength and significant increases in energy absorption (toughness or impact-resistance) have been reported by several investigators in static tests on concrete reinforced with randomly distributed short steel fibers. Studies on the dynamic behavior of FRC are rather limited in comparison. This, despite the fact that the most important property of such composites is its superior impact resistance.

As yet no standard test methods are available to quantify the impact resistance of such composites, although several investigators have employed a variety of tests including drop weight, swinging pendulums and the detonation of explosives. These tests though useful in ascertaining the relative merits of different composites do not yield basic material characteristics which can be used for design.

The author has recently developed an instrumented Charpy type of impact test to obtain basic information such as load-deflection relationship, fracture toughness, crack velocity and load-strain history during an impact event. From this information, a damage based constitutive model was proposed. Relative improvements in performance due to the addition of fibers as observed in the instrumented tests are also compared with other conventional methods.

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APPLICATION OF FRACTURE MECHANICS TO CEMENTITIOUS COMPOSITES

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Portland cement concrete is a relatively brittle material. As a result, mechanical behavior of concrete, conventionally reinforced concrete, prestressed concrete, and fiber reinforced concrete is critically influenced by crack propagation. It is, thus, not surprising that attempts are being made to apply the concepts of fracture mechanics to quantitify the resistnce to cracking in cementitious composites.

Many attempts have been made, in the last two decades or so, to apply the fracture mechanics concepts to cement, mortar, concrete and reinforced concrete. So far, these attempts have not led to a unique set of material parameters which can quantify the resistance of these cementitious composites to fracture. No standard testing methods or a generally accepted theoretical analysis is established for concrete as for metals.

One of the primary reasons for this lack of success is that most of the past work is based on the concept of linear elastic fracture mechanics. However, it is increasingly being realized that because of the large-scale heterogeneity inherent in the microstructure of concrete, strain softening, microcracking and large scale process zone, the classical linear elastic (or the classical elastic-plastic) concepts must be significantly modified to predict crack propagation in concrete. More recently, researchers in many countries are beginning to explore, theoretically, numerically and experimentally these hitherto unexplored aspects of nonlinearity associated with crack growth in cementitious composites as well as in ceramics and rocks.

As a result of this increased understanding of fracture processes and better numerical and theoretical modeling, it is also beginning to be recognized that use of nonlinear fracture mechanics can be advantageous in rational analysis of the behavior of concrete structures. Situations where fracture mechanics can be a useful tool include: impact and impulsive loading, dynamic shear fracture, some aspects of bond between reinforcement and concrete, and predictions of deflections and ductility.

Crack growth in cement composites can be divided into subcritical and post-critical. Subcritical crack growth occurs prior to reaching the maximum (or critical) load and is often termed slow crack growth since it occurs with increasing loads. Partly a result of aggregates and fiber-bridging effects, crack growth is associated with nonlinear deformation. Because of these nonlinear effects, it is not possible to use a single parameter (for example, K_{Ic}) LEFM based criteria for cement based composites. In this paper, it is observed that the occurrence of the critical load can be uniquely determined by two parameters: K_{Ic} and $CTOD_c$ are size independent.

The effective crack model can be applied to either precracked or uncracked structures. It is shown that this model can correctly predict many heretofore observed phenomena such as size dependency on conventional K_{Ic}, modulus of rupture and uniaxial tensile strength.

PARAMETERS FOR CONSTITUTIVE MODELING

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The basic information required in design (based on finite element calculations) for concrete structures is the multiaxial stress-strain relations of concrete, tensile stress-strain relation of steel (reinforcing rods or prestressing tendons) and the bond-slip behavior of the steel-concrete interface. The subject of this paper is to summarize some of the current attempt at predicting multiaxial constitutive relationship of plain concrete. The paper is substantially based on the two extensive and excellent committee reports prepared by ASCE and CEB.

The following topics will be discussed:

1) Uniaxial Behavior
   - Compression
   - Tension
   - Cyclic Loading
2) Strength of Concrete Subjected to Multiaxial Stresses
   - Some Classical Failure Theories
   - Some Recent Theories
3) Constitutive Models
   - Macroscopic Isotropic Postulates
   - Secant vs. Incremental Laws
   - Isotropic Nonlinear Elastic Models
   - Orthotropic Models
   - Plasticity Based Models
   - Physically Motivated Models
4) Continuous Damage Based Models

An orthotropic model to predict the ascending and descending parts of the stress-strain curves of concrete subjected to biaxial or triaxial compressive stresses is presented. The stress-induced orthotropic material properties are expressed in terms of six constants in the compliance matrix. The six material constants are expressed as functions of stress invariants at ultimate strength. The proposed orthotropic constitutive model depends on the knowledge of the three principal stresses at the maximum strength, which are obtained through the use of a strength criterion (which is sensitive to all the three stress invariants). For predicting the ascending and descending parts of the multiaxial stress-strain curves, the six orthotropic material constants in the compliance matrix are continuously changed with the help of a "fracturing index" parameter. The proposed constitutive model compares favorably with the available experimental data under multiaxial stresses.

ORTHOTROPIC MODEL FOR COMPLETE STRESS-STRAIN CURVES OF CONCRETE UNDER MULTIAXIAL STRESSES

UN MODELE ORTHOTROPIQUE DE COURBES COMPLETES DE CONTRAINTES-DEFORMATIONS DU BETON SOUS CONTRAINTES MULTIAXIALES

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ABSTRACT

An orthotropic model to predict the ascending and descending parts of the stress-strain curves of concrete subjected to biaxial or triaxial compressive stresses is presented. The stress-induced orthotropic material properties are expressed in terms of six constants in the compliance matrix. The six material constants are expressed as functions of stress invariants at ultimate strength. The proposed orthotropic constitutive model depends on the knowledge of the three principal stresses at the maximum strength, which are obtained through the use of a strength criterion (which is sensitive to all the three stress invariants.). For predicting the ascending and descending parts of the multiaxial stress-strain curves, the six orthotropic material constants in the compliance matrix are continuously changed with the help of a "fracturing index" parameter. The proposed constitutive model compares favorably with the available experimental data under multiaxial stresses.

Résumé

Nous présentons un modèle orthotropique pour prédire les parties ascendante et descendante des courbes de contraintes-déformations du béton soumis à des contraintes biaxiales ou triaxiales en compression. Les propriétes orthotropiques du matériau, induites par les contraintes, sont exprimeés par 6 constantes dans la matrice de déformation unitaire. Les 6 constantes caractéristiques du matériau sont des fonctions des invariants de contraintes au niveau de contrainte maximum. Le modéle de constitution orthotropique proposé dépend de la connaissance des 3 contraintes maximales au niveau de résistance maximale; elles sont obtenues à l'aide d'un critére de résistance du matériau, sensible aux 3 invariants de contraintes. Pour prédire les parties ascendante et descendante des courbes de contraintes multiaxiales et déformations, les 6 constantes orthotropiques du matériau dans la matrice de déformation unitaire sont modifiées de facon continue à l'aide d'un paramètre "index de rupture". Le modèle constitutif propose est en bon accord avec les résultats expérimentaux disponibles en contraintes multiaxiales.

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[redacted]

CRACK PROPAGATION RESISTANCE OF FIBER REINFORCED CONCRETE

By

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ABSTRACT

A fracture mechanics based theoretical model is presented to predict the crack propagation resistance of fiber reinforced cement based composites. Mode I crack propagation and steel fibers are treated in the proposed model. The mechanism of fracture resistance for FRC can be separated as: subcritical crack growth in matrix and beginning of fiber bridging effect; post critical crack growth in matrix such that the net stress intensity factor due to the applied load and the fiber bridging closing stresses remain constant; and a final stage where the resistance to crack separation is provided exclusively by fibers. The response of FRC during all these stages was successfully predicted from the knowledge of matrix fracture properties and the pull-out load vs. slip relationship of single fiber. The model was verified with the results of experiments conducted on notched-beams reported here as well as by other researchers. Beams were loaded in a closed-loop testing machine so as to maintain a constant rate of crack mouth opening displacement.

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A TWO PARAMETER FRACTURE MODEL FOR CONCRETE

by

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ABSTRACT

Attempts to apply linear elastic fracture mechanics (LEFM) to concrete have been made for several years. Several investigators have reported that when fracture toughness (K_{Ic}) is evaluated from notched specimens using conventional LEFM (measured peak load and initial notch length) a significant size effect is observed. This size effect has been attributed to nonlinear slow crack growth occuring prior to the peak load. In this paper a two parameter fracture model is proposed to include this nonlinear slow crack growth. Critical stress intensity factor (K_{Ic}^{S}) is calcualted at the tip of the effective crack. The critical effective crack extension is dictated by the elastic critical crack tip opening displacement ($CTOD_c$). Tests on notched beam specimens showed that the proposed fracture criteria to be size-independent.

The proposed model can be used to calculate the maximum load (for Mode I failure) of a structure of an arbitrary geometry. The validity of the model is demonstrated by an accurate simulation of the experimentally observed results of tension and beam tests.

Key Words: LEFM, critical stress intensity factor, critical crack tip opening displacement, effective crack, slow crack growth, size effect, uniaxial tensile strength, modulus of rupture.