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CHAPTER 1—INTRODUCTION

1.1 General

Impact, explosions, and earthquakes impose dynamic effects on concrete structures. Impact loading on a parapet can occur if it is struck accidentally by a crane. Seismic loading produces significant strain rates in concrete shear-walls and other lateral force-resisting elements. Explosive loading, due to accidental detonation of industrial vapor clouds or terrorist bombing, produces high strain rates in floor slabs and columns. These possibilities have prompted experiments on plain concrete specimens to investigate basic properties of concrete under various states and rates of loading. Under dynamic loading (rapidly applied loads of short

duration), both structural and material responses depend on the applied loading rate. Although both the geometry of the structure and the material properties control the rate of cracking, this report is concerned primarily with the material effects.

Common practice for evaluating the resistance of concrete structures to dynamic loading is to:

- a) Estimate the transient state of stress in the structure using an elastodynamic analysis; and
- b) Evaluate the resistance of the structure using strength properties for the concrete and steel that are enhanced by strain-rate-dependent factors. For the failure modes of a concrete structure controlled by yielding of the reinforcement or crushing of the concrete, common practice usually provides reliable design information. For those failure modes controlled by crack propagation, however, such as diagonal tension or splitting failures, and where resistance to fracture is of fundamental importance for computations of energy absorption and energy dissipation, common practice does not usually yield reliable information. This inadequacy is due primarily to the fact that dynamic fracture of concrete structures does not involve instantaneous fracture, but continuous dynamic crack propagation under dynamic loading. Reliable dynamic failure analyses of concrete structures requires knowledge of the dynamic fracture properties of the concrete as well as its strain-rate-dependent properties. Therefore, this report concerns not only strain rate effects but also consideration of the dynamic fracture properties of concrete in general.

As shown in Fig. 1.1, the strength of concrete in tension, flexure, and compression increases with an increase in the loading rate. The strain corresponding to the maximum strength also increases with an increase in the loading rate. The increase in strain is due to the development of multiple cracks in the failure zone, and the value of the maximum strain is strongly dependent on the width assumed for the failure zone.

The differing rates of increase in tensile, flexural, and compressive strengths with increasing loading rates, and the crack propagation effects that cause failure, can result in the mode of failure of a concrete member changing from flexure to shear with an increase in the loading rate. Consequently, a dynamically loaded beam may require more shear reinforcement to ensure ductile behavior than the same beam loaded statically. Characterization of the rate effects for the materials of the beam, its inertial effects, and how those effects combine to control crack propagation, are essential to successful designs to resist high strain-rate loadings.

Inertial effects are involved in any impact loading of a structure or in any impact testing in a laboratory. In the latter case, many efforts have been devoted to reducing this effect so that dynamic test data can be used to evaluate the dynamic strength of concrete by static analysis. Inertial effects, however, are inherent in any dynamic event of material deformation or fracture. The inertial effect of a large mass of material, such as concrete, considerably increases the impact resistance of the structure. This effect occurs because the input energy should be transformed into kinetic energy, which is directly proportional to the mass, for moving the

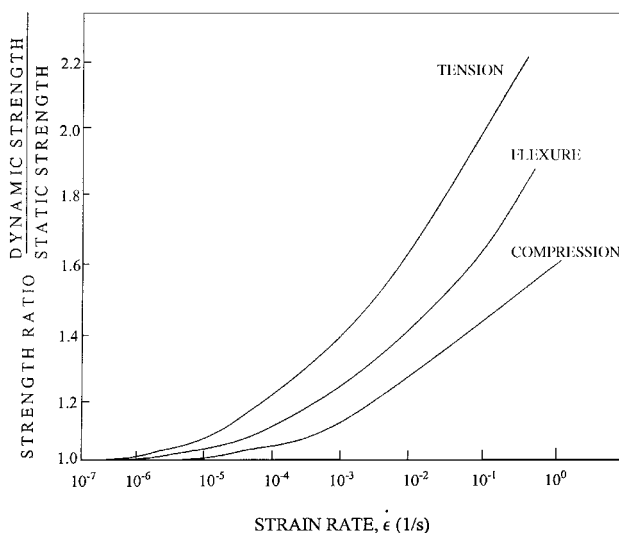


Fig. 1.1—Strain rate behaviors of plain concrete in different simple response modes (Suaris and Shah 1983).

material necessary for crack formation and propagation. Therefore, any dynamic loading analysis should incorporate inertial effects rather than avoid them. Fortunately, current dynamic finite-element computer programs can readily handle this problem.

Because of inertial and crack propagation considerations, it is not possible to directly link strain rates and loading rates. The test method used in the laboratory to investigate dynamic effects for a given type of loading is usually related to a given strain rate range. As indicated in Fig. 1.1, the lowest strain rate at which testing is performed is approximately 10^{-7} s^{-1} . That rate, which corresponds to static loading, also has creep associated with it. The next higher strain rate region, up to 10^{-6} s^{-1} , is a quasistatic loading regime and is the rate commonly involved in laboratory testing to investigate seismic effects using servo-controlled hydraulic jacks. In the third region, up to 10^{-3} or 10^{-2} s^{-1} , mechanical resonance in the specimen and testing apparatus may need to be considered to properly interpret the response of the concrete within the complex specimen-machine interaction that occurs. Such higher rates can occur in shaketable experiments and in structures dynamically loaded by earthquakes. Rates up to 1 s^{-1} can be achieved in the laboratory using special hydraulic testing machines equipped with high-capacity servo-valves. Loading rates between 10^{-2} and 1 s^{-1} correspond to those imposed by impact loadings such as vehicles hitting bridge piers or aircraft landing on airport runways. Finally, rates above 10^{-2} s^{-1} cannot be readily achieved with a hydraulic testing machine. Impact or drop weight machines should be used or wave propagation utilized (Split-Hopkinson pressure bar device) to induce rate effects in small volumes of material. Loadings in this region correspond to those that can occur in bombing adjacent to or within the building, and service system and other explosions that occur within the building.

Rate dependence is thought to have a microstructural origin in the viscoelastic character of the hardened cement paste. Rate dependence probably originates from the ability of the bonds in calcium silicate hydrates to break and reform in a process governed by their thermal activation energy. A second origin of rate dependence is thought to be the time-dependent nature of crack growth, which originates in the successive ruptures of interparticle bonds in the hardened cement paste or concrete. Those ruptures cause growth of the fracture crack, an effect that is also a thermally activated process (Bazant, Gu, and Faber 1995; Bazant and Prat 1988).

This report examines the factors that cause strain-rate effects on concrete properties such as elastic modulus and tensile strength, and on fracture properties such as crack initiation, crack propagation, critical stress-intensity factor, and fracture energy. The effects of strain rates between 10^{-6} to 10^4 s^{-1} are considered. The primary focus is on unreinforced specimens because the vast majority of the data reported in the literature deal with such specimens. Relatively little unclassified work has been reported on the dynamic fracture of reinforced concrete structures. Therefore, some interpretation is needed to apply the work summarized herein directly to reinforced structures.

1.2—Conceptual models

Any conceptual model that takes into account static and quasistatic as well as strain-rate effects in concrete depends on the scale of observation. The use of Wittmann's (1983) approach of studying concrete on three levels (macro, meso, and micro) helps to clarify the origins of rate effects.

1.2.1 Macrolevel—At the macrolevel, concrete is idealized as homogeneous and isotropic. For very large structures with dimensions measured in meters, linear elastic fracture mechanics (LEFM) may be used; a single crack can be assumed, and a critical combination of crack length and applied boundary conditions can then lead to crack growth. Growth can be locally stable (slow) or unstable (fast), depending on the stress gradients that the growing crack encounters. A critical value of K_I (the stress intensity factor) should be reached as a necessary condition for crack growth to occur. This critical value of stress intensity, also referred to as the fracture toughness, has been measured and found to be much larger under dynamic loading than under static loading, (Mindess, Banthia, and Yan 1987; John and Shah 1986). Macrolevel models regard the cause of strain-rate effects as a transfer of strain energy at finite velocity from the structure surrounding the crack to the newly formed cracked surfaces. If the velocity of the advancing crack is low, strain-energy transport from the remainder of the stressed body along the crack surfaces to the crack tip is communicated via Rayleigh waves that travel at the Rayleigh wave velocity C_r . In tests carried out by Mindess, Banthia, and Yan (1987), John and Shah (1986), and Ross, Tedesco, and Kuennen (1995), crack velocities at strain rates in the range of 0.1 to 1 s^{-1} were of the order of 100 ms^{-1} , or less than 10% of the Rayleigh wave velocity. Yon, Hawkins, and Kobayshi (1991a) measured somewhat higher crack velocities of 132 and 250 ms^{-1} but again, their values are considerably less than the Rayleigh wave velocity. On the other hand, Ross, Tedesco, and Kuennen (1995) have suggested that the crack velocity increases linearly with an increasing strain rate on a log-log plot. They report experimentally measured crack velocities well in excess of 100 ms^{-1} at strain rates greater than 1 s^{-1} . Curbach and Eibl (1989) have measured crack-tip velocities in the range of 120 to 540 ms^{-1} , and Takeda (1986) has reported crack-tip velocities as high as 1000 ms^{-1} using an extremely high loading rate. In theory, however, as the crack velocity V approaches the Rayleigh wave velocity C_r , crack faces do not move apart fast enough to provide the localized strains necessary for a high crack tip stress-intensity factor K_I . It follows that the localized stress intensity at the crack tip is less able to break bonds as the crack-tip speed increases. Thus, the strength (which might theoretically be calculated from a knowledge of the interparticle bond strengths and the crack size distribution) of a linear, isotropic material depends on V/C_r and is therefore rate-dependent.

Although this conceptual model correctly predicts the tendency for increasing strength with increasing crack velocity, it does not correctly predict the magnitude of that increase. Shah (1983) has found experimentally that crack velocities in impact tests are less than 15% of the Rayleigh wave velocity. According to the theoretical results of Freund

(1972), this crack velocity is too low to produce a strength increase effect larger than a few percentage points, as compared with static loading. Experiments show an increase in strength larger than that amount. Thus the macromodel, while providing some qualitative insight into the effects of rate, is inadequate. To gain a physical explanation, it is necessary to look at concrete at the smaller scale of the mesolevel.

1.2.2 Mesolevel—On the mesolevel (the scale of the maximum aggregate size), normal-strength concrete is idealized as a composite material comprised of a cement paste matrix with flaws, dense unflawed aggregate, and a cement-aggregate interface zone that is a rich source of flaws. It is convenient to idealize the flaws as penny-shaped and uniformly distributed throughout the concrete. The flaws are all assumed to be of a size that can be calculated from the pore size distribution of the paste (Weerheijm 1984). If LEFM is applied to such a regularly flawed body, and if inertia forces are considered to oppose a crack opening (Mott 1948), the apparent tensile strength is found to increase monotonically with strain rate. At strain rates of 10^4 s^{-1} , such an idealization can account for an increase in tensile strength relative to the static value by a factor of approximately 2.

Another rate effect that occurs at the mesolevel is due to the properties of the aggregate. Dense (unflawed) aggregate is tougher (requires more energy per unit area of new crack surface) than hydrated cement paste (HCP); therefore, if more aggregates fracture during the passage of a crack, the strength and fracture energy of the entire body will also be higher. According to Ross (1991), for low strain-rate loading, “the inherent flaws at the higher stress concentrations grow and relieve the stresses at other sites. Under the low loading rates, these activated cracks have time to grow into the weakest areas of the concrete matrix and aggregate. As the compression-load rate increases, many more of the inherent flaws are activated and forced to grow into the high-strength areas of the mortar and aggregate. This results in increased strength, a larger number of much shorter cracks and smaller fragments.” Also at high rates, cement paste cannot creep so that stress relaxation cannot occur at a crack tip. A similar explanation applies to hydrated cement paste without aggregate. Cement paste contains unhydrated cement particles and calcium hydroxide, which play a role similar to that of aggregate particles. At slow loading rates, cracks advance preferentially around unhydrated cement and calcium hydroxide in preference to the calcium silicate hydrate matrix.

A third effect, which has not been quantified, concerns the development of a process zone in front of the physical crack tip. The process zone concept is illustrated in Fig. 4.1. For static loading, it is known that a stress intensity develops around the crack tip and microcracks develop within a certain area in advance of the tip. That area is called the fracture process zone. Work by Du, Kobayashi, and Hawkins (1989) and by John and Shah (1986) has shown that for the higher stress rates considered in this report, the size of the process zone decreases relative to that observed at the slower (10^{-6} s^{-1}) rates. As a result, concrete behaves in a more brittle fashion at high loading rates than at low rates. Under creep loading conditions, the process zone is smaller than

under quasistatic conditions, Bazant and Gettu (1989, 1992) and Akutagawa et al. (1991). The process zone size seems to be a maximum for quasistatic conditions (10^{-6} s^{-1}).

1.2.3 Microlevel—Mihashi and Wittmann (1980) proposed that concrete can be considered a collection of atoms in which bonds, such as the Van der Waals forces between particles of cement gel, break and randomly re-establish themselves. If an external force is applied, more bonds may break than are re-establish in a specified time interval. This result means that less damage occurs the shorter is the time interval. Mihashi and Wittmann (1980) have used a stochastic approach based on work by Mihashi and Izumi (1977) to derive a power relation between strength and stress rate. The relation describes the variability of macroscopic mechanical behavior based on a combination of the randomness of mesolevel behavior, which is heterogeneous, and microlevel behavior, which is governed by thermal activation effects.

1.3—Scope

This report considers issues related to the fracture of concrete materials and structures subjected to dynamic loading from impact, explosions, and earthquakes. Experimental evidence on rate effects is described in Chapter 2, including mixed mode effects associated with shearing action and bond effects. Analytical modeling of strain-rate effects is discussed in Chapter 3. Models based on micromechanics considerations, damage models that incorporate microcracking phenomena, and strain-rate-dependent fracture models are described. Computational modeling of localized failure under dynamic loading is discussed in Chapter 4.

Although still the subject of speculation, rate effects on strength are considered to originate in the rate dependence of bond breakage in the mortar at a microscopic crack and in local inertia effects. Indirect evidence is presented suggesting that, at high loading rates, the cement-aggregate bond strength increases to the point that it is no longer the weakest link in the concrete. Another possible explanation, however, is that the large amount of energy per unit time available at high loading rates does not force failure at the weakest link.

No increase in bond strength between concrete and smooth wires is found at high loading rates (Yan and Mindess 1994; Vos and Reinhardt 1982). The bond between concrete and deformed bars, however, is greater at high loading rates than low rates because the bond strength depends on two effects: shear and microfracture within the concrete; and bearing between the concrete and the deformations of the bar. The strength of concrete in both the shear and bearing modes increases with increased loading rate. This effect of enhanced bond strength often also leads to reductions in failure ductility for reinforced concrete elements.

This report is one of a series of four state-of-the-art reports on fracture of concrete prepared by ACI Committee 446 over the past decade. Two reports, titled “Fracture Mechanics of Concrete: Concepts, Models and Determination of Material Properties (ACI 446.1R-91)” and “Finite Element Analysis of Fracture in Concrete Structures. State-of-the-Art (ACI

446.3R-97),” respectively, are published in the *ACI Manual of Concrete Practice*. The third report, “Fracture Mechanics Applications and the Code” was at one time published in the *ACI Manual of Concrete Practice* but was withdrawn in 1998.

1.4—Abbreviations

CCS	= crack-closing stress
CLWL-DCB	= crack-line, wedge-loaded, double cantilever beam
CMOD	= crack-mouth-opening displacement
CMSD	= crack-mouth-sliding displacement
COD	= crack-opening displacement
CTD	= crack tip displacement
CTOD	= crack-tip-opening displacement
CTSD	= crack-tip-sliding displacement
FC	= fictitious crack
FEM	= finite element model
FPZ	= fracture process zone
HCP	= hardened cement paste
LEFM	= linear elastic fracture mechanics
NS	= nonsingular
NS-FPZ	= nonsingular fracture process zone
S-FPZ	= singular fracture process zone
SHPB	= Split-Hopkinson pressure bar
SIF	= stress intensity fact

CHAPTER 2—EXPERIMENTAL EVIDENCE OF RATE EFFECTS

Testing machines have been developed for investigating the properties of concrete at high strain rates, including Split-Hopkinson pressure bar (Reinhardt 1985) and Charpy (ASTM E 23) devices. Special strain and noncontact capacity devices, designed to measure crack propagation from a pre-existing notch, are able to measure the crack extension rate as a function of load and time.

Experimental data on rate effects have been obtained primarily by subjecting concrete specimens to flexure or to unconfined direct tension or compression. The experimental apparatus needs to be developed to study rate effects under other stress states. Any study of high strain-rate effects requires careful moisture control of specimens both before and during testing. Moisture affects stress relaxation in the process zone, and loss of moisture during testing increases the effective confinement of the loading platens for compressive loading and causes surface shrinkage drying effects for tensile loadings. The experimental data presented in this chapter are grouped into four categories: Mode I failures, compression failures, mixed-mode failures, and bond failures. Both direct tension and flexural loadings cause Mode I failures. In Mode I failures, microcracking is small before a single, dominant cleavage or separation plane develops. The lumping together of direct tension and flexural failures as Mode I failures is somewhat arbitrary because there is also justification for separating those two types of failure. For compression failures, mixed-mode, and bond failures, extensive microcracking develops before failure (van Mier 1997).

The relative importance of the rate effects on strength under direct tension, flexure, and compression is illustrated in Fig. 1.1 for a wide range of strain rates.

2.1—Mode I failure: plain concrete and mortar

Mode I failure is studied by applying macroscopic tensile stress across the fracture process zone that exists ahead of the crack. Strain rates as great as 1 s^{-1} can be achieved using displacement-controlled test equipment (Yon, Hawkins, and Kobayashi 1991a, 1992b). Higher rates require the use of a Split-Hopkinson pressure bar test or an impact test using either a drop weight or a pendulum. Explosive and flyer plate testing have also been used, though quantitative data from these sources are scarce.

The strain-rate sensitivity of concrete is greater in direct tension and flexure than in compression, as shown in Fig. 1.1. Early evidence for strain-rate sensitivity in Mode I failure (Birkimer and Lindeman 1971) is illustrated in Fig. 2.1. That evidence shows an increase in the fracture strain of concrete (strain at peak stress) with an increasing strain rate. Typical experimentally measured load-deformation responses are also shown in John and Shah (1987a). Typical stress-deformation results for direct tension (Reinhardt 1986) are shown in Fig. 2.2 while results for flexure (Gopalaratnam and Shah 1985) are shown in Fig. 2.3. Reinhardt (1985) summarized data on flexural tensile strength as measured in three- and four-point bending tests. That data are presented in Fig. 2.4(a) in terms of the influence of loading rate on the modulus of rupture (flexural strength) for concretes of differing strengths. The modulus of rupture (normalized to the modulus of rupture at a rate of 10^{-6} s^{-1}) increases with the strain rate, as measured at the outermost fiber. As noted by Reinhardt, there is some scatter in the results, possibly due to some systematic difference between the four-point (two upper lines) and the three-point (three lower lines) bend test results. Also, the concrete strength has an effect, as noted by Reinhardt, but the nature of the effect is not defined clearly by the data. A clearer indication of the concrete strength effect appears in Cowell’s data (1966), shown in Fig. 2.4(b), where the strength of

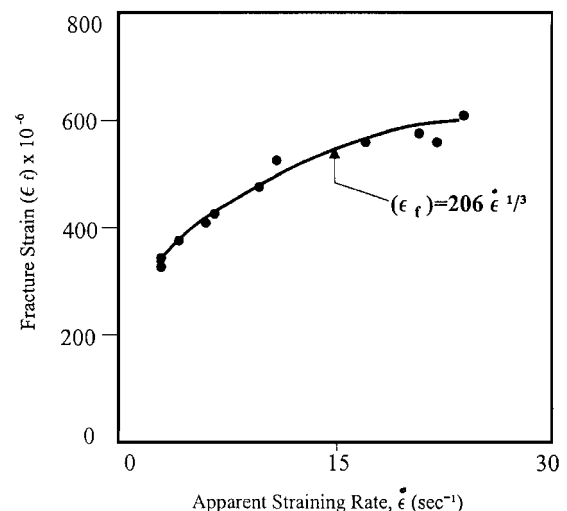


Fig. 2.1—Fracture strain in tension versus apparent strain rate (Birkimer and Lindeman 1971).

weaker concrete increases more rapidly with strain rate than that of stronger concrete. John and Shah (1987b), Weerheijm (1992), and Reinhardt and Weerheijm (1991) have successfully modeled the strength dependence of tensile strength on strain rate, and John and Shah's analytical predictions are shown in Fig. 2.4(b). The strain rates considered in the tensile tests of concrete and illustrated in Fig. 2.1, 2.4(a), and 2.4(b) range from approximately 10^{-7} s^{-1} to approximately 10 s^{-1} . Based on these data, it appears that strain-rate effects in concrete under tensile stress can occur throughout that entire range, and that such effects distinctly exceed scatter in the data (strength exceeds static strength by roughly 20%) for rates greater than 10^{-4} to 10^{-3} s^{-1} .

Figure 2.5 (Gopalaratnam and Shah 1986) shows the variation in elastic modulus for cement mortar in flexure, with strain rate. The modulus, defined as the slope of a line drawn from the origin to a point on the load-deformation curve equal to 0.4 of the peak load, shows only limited sensitivity to strain rate. The specimens for which the results are shown were reported to be saturated surface-dry. They were cured

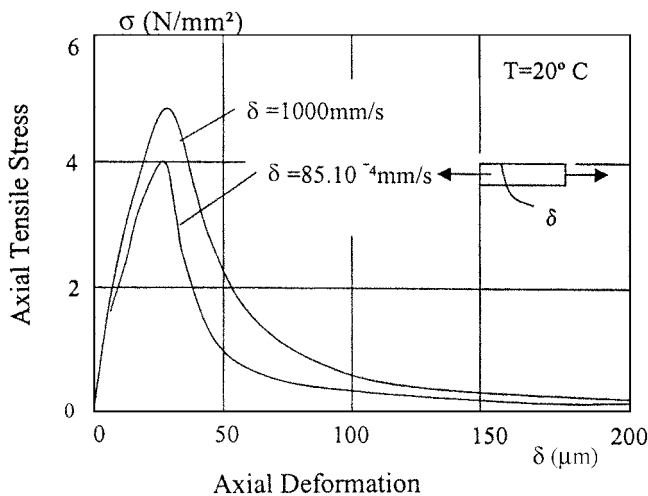


Fig. 2.2—Uniaxial tension stress-strain curves of concrete at two different rates of loading (Reinhardt 1986).

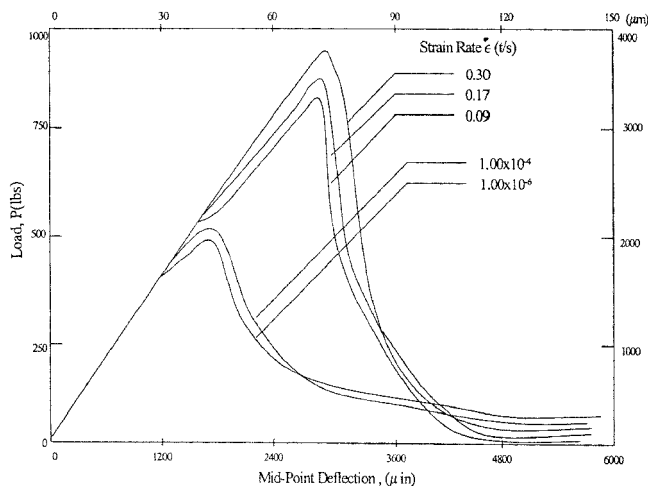


Fig. 2.3—Typical load-deflection response of concrete in bending at different rates of loading (Gopalaratnam and Shah 1985).

under 98% relative humidity for 2 days. The variation in modulus with strain rate may represent a decrease in prepeak nonlinearity or the same amount of prepeak nonlinearity with increasing strength, or both.

Brooks and Samaraie (1989) conducted direct tensile tests and three- and four-point bend tests on concrete specimens to investigate the sensitivity of prepeak stress-strain relationships, and the peak stress, to loading rate. Their results for tension confirm those of Curbach and Eibl (1989) for compression. Stiffness values for all stress levels, up to and including the peak stress, increase with increasing stress rates. Brooks and Samaraie also noted that the limit of

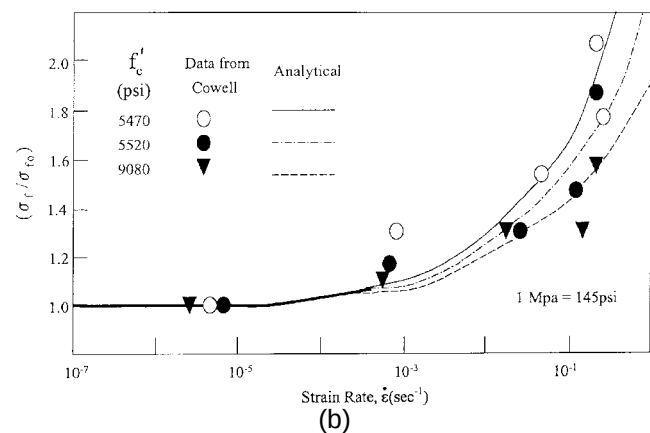
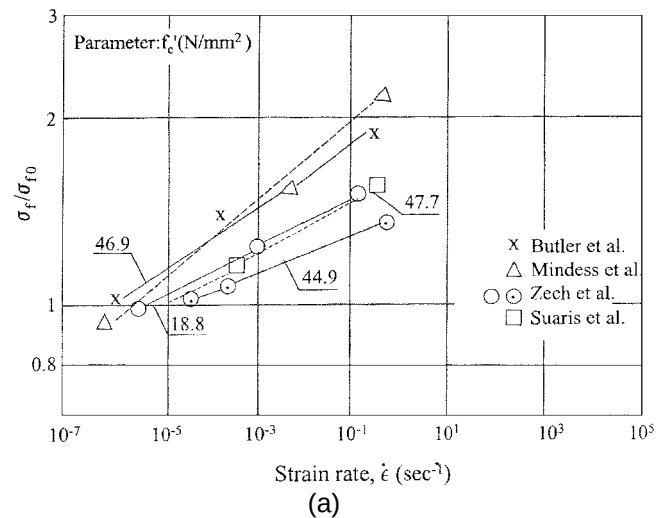


Fig. 2.4—(a) Modulus of rupture versus strain rate (Reinhardt 1985); and (b) strain rate effect on tensile strength of concrete (John and Shah 1987a).

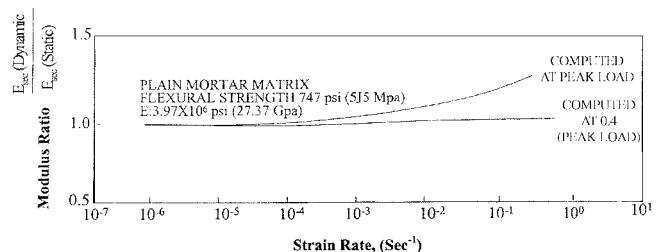


Fig. 2.5—Effect of strain rate on elastic modulus of mortar (Gopalaratnam and Shah 1986).

proportionality between stress and strain, which may be due to the onset of microcracking, increases with an increasing stress rate. The origin of this effect, which is a central factor in the rate dependence of Mode I fracture, has not been identified. Brooks and Samaraie's data for tension, however, indicate that the initial tangent modulus is insensitive to stress rate, whereas Curbach and Eibl report for their compression data that the initial tangent modulus increases with stress rate as shown in Fig. 2.6.

A method to back-calculate the dynamic modulus of elasticity from crack opening rate measurements using a dynamic two-dimensional finite element method (FEM) has been developed by Yon, Hawkins, and Kobayashi (1991a) and implemented in the computer program ANSYS. Their FEM approach provides a unique solution for the modulus of elasticity before the onset of microcrack propagation, including consideration of inertia and stress concentration effects at the notch tip. Their back-calculated moduli, for the results of displacement-controlled three-point bend tests, showed a 50% greater modulus at a strain rate of 0.24 s^{-1} than at $4.9 \times 10^{-4} \text{ s}^{-1}$.

Conversely, at the other end of the strain-rate spectrum, the rate effect during slow loading has been measured for plain concrete by Bazant and Gettu (1989, 1992), for dam concrete by Bazant et al. (1991) and He et al. (1992), and for rock by Bazant, Bai, and Gettu (1993). Bazant and his colleagues used the size-effect method to determine the dependence of the effective fracture toughness, and the effective (elastically equivalent) length of the fracture process zone, on the time to reach the peak load for constant-rate loading. The time-to-peak-load ranged from 1 to approximately 10^5 s , and the specimens were notched beams loaded in three-point bending. As expected, the fracture toughness decreased as the loading rate decreased. A surprising conclusion, however, was that the fracture process-zone length also decreased as the loading rate decreased. The response became more brittle and closer in behavior to that for linear fracture mechanics concepts (in the sense of an effective elastic modulus approximation to viscoelastic effects). This increasing brittleness appears to be due to creep or stress relaxation in the fracture process zone (Bazant and Li 1995), which supplements the creep that occurs in the bulk of the specimen.

The development of decreasing prepeak crack growth with increasing loading rates is supported by data from the tests by John and Shah (1986) on notched beams. Similar trends have also been observed by Tinic and Bruhwiler (1985) for uniaxial tension tests, by Suaris and Shah (1985) and Ahmad and Shah (1985) for uniaxial compression tests, and by John (1988) for flexure based on load versus crack-mouth opening responses of notched beams subject to impact loadings. Based on impact tensile test results, Reinhardt (1986) has also reported that the secant modulus increases with an increasing loading rate, with the increase being approximately 25% at a strain rate of 1 s^{-1} .

To address the issue of whether the velocity of crack extension significantly affects the modulus measurements, that velocity was measured by John and Shah (1986), Bhargava

and Rehnstrom (1977), Mindess et al. (1986), and Yon, Hawkins, and Kobayashi (1991b and 1992a). The velocity was found to be only 5% of the Rayleigh wave velocity in concrete and, therefore, that velocity has little effect on measured values of the modulus.

In the remainder of this section, a brief summary is provided of the experimental methods commonly used to study Mode I dynamic fracture of concrete specimens.

2.1.1 Charpy impact tests—The Charpy V-notch impact test is the most widely used test for determining the toughness of structural materials (ASTM E 23). In this test, the difference between the initial and final height of an impact hammer is interpreted as the energy of fracture of the specimen. To accommodate the larger specimens that are required for concrete testing, the standard metal testing machine has been modified. Hibbert (1979) instrumented the pendulum striker to obtain continuous load- and energy time-histories of plain and fiber-reinforced concrete beams. He observed that for both types of specimens, the peak load under impact loading (impact velocity of 2.85 ms^{-1}) was approximately 10 times that under static loading. This increase is high compared with the two-fold or less increase that is generally reported. The high load has been attributed to inertia of the specimen and a relatively stiff hammer-beam contact zone (Suaris and Shah 1982c; Gopalaratnam, Shah, and John 1984; and Gopalaratnam and Shah 1986). Gopalaratnam, Shah, and John (1984) used a modified Charpy device to test small beams with dynamic load cells built into both the pendulum striker and the anvil supporting the beam (Fig. 2.7). These measurements demonstrated that large errors in peak load result if test parameters are not properly chosen, particularly if loads are recorded only at the striker, as in Hibbert's tests. They also provided guidelines for selecting test parameters with a view to minimizing the effects of specimen inertia, which are compounded in cement matrix composites because of the relatively small strength-to-weight ratio and the shorter times to failure than in metal specimens. Gopalaratnam, Shah, and John (1984) report that tests on $75 \times 25 \times 225 \text{ mm}$ plain concrete specimens indicated a 100% increase in the flexural strength using the modified Charpy device and a strain rate of 1 s^{-1} compared with the strength at 10^{-6} s^{-1} . They also report, based on the increase in secant

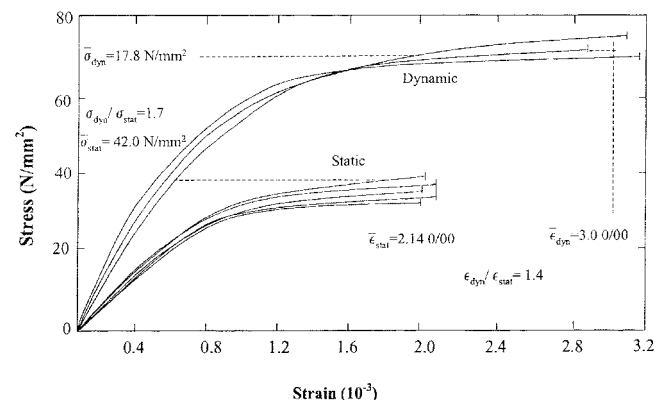


Fig. 2.6—Stress-strain curves obtained for static and dynamic compressive loading (Curbach and Eibl 1989).

modulus at peak loads, that higher rates are accompanied by a decrease in the amount of prepeak microcracking. Weaker mixtures are reported to be more rate-sensitive than stronger ones.

2.1.2 Drop-weight impact tests—In drop-weight impact tests, initially developed for metal testing, the force exerted by the striker on the specimen is monitored throughout the test by means of strain gages on the striker. Inertial effects that occur during instrumented impact testing of concrete were analyzed by Suaris and Shah (1982c). They modeled the experiment as a two-degree-of-freedom system (Fig. 2.8). In Fig. 2.8, m_h and m_b are the masses of the hammer and the beam, respectively, and k_b and k_c are the stiffnesses of the beam and of the contact between beam and hammer, respectively. The accuracy of the load measurement may be improved considerably by introducing a soft pad where the striker contacts the specimen, thereby reducing k_c . The advantage of this step, however, may be compromised by the lower strain rates produced.

Suaris and Shah (1983) showed that results by Hibbert (1979), indicating 10- to 15-fold strength enhancement due to rate effects, fail to take into account the inertial response of the specimen. In Hibbert's tests, fracture occurred during the first inertial oscillation cycle when the discrepancy between the measured impact hammer load and the beam load is a maximum. One recommended method for reducing such errors due to the effect of inertia on the measured load is to accept peak load measurements only if they occur after three inertial oscillations (Server, Wullaert, and Sheckhard 1977). Alternatively, a pad may be introduced into the contact zone to reduce inertial oscillations. The reliability of

the results can be further enhanced by instrumenting the supports. Another technique for compensating for inertial effects (Mindess, Banthia, and Yan 1987) is to measure the acceleration profile of the beam using accelerometers mounted along the length of the beam. Then, by applying the principle of virtual work, the generalized inertial load acting on the beam can be evaluated. Using one-dimensional dynamic equilibrium (Banthia et al. 1989), the system can be transformed into an equivalent static system that predicts the true midspan moments. Bending loads obtained from this procedure are compatible with the measured support reactions, although there is a time lag. Banthia, Mindess, and Bentur (1987a) showed that a major portion of the impact hammer load (60 to 70%) was contributed by the inertial load; errors in evaluating the inertial load have a considerable effect on the calculated beam bending load.

In an attempt to balance the energy during an impact, Banthia, Mindess, and Bentur (1987a) found that, at the peak load, only a small fraction of the hammer energy had been transferred to the specimen; most of it stayed in the machine as strain energy. This result is significant because a great number of investigations have used the machine instrumentation to calculate the energies consumed by a specimen.

Using machine instrumentation can increase apparent fracture energy values at high load rates (Banthia, Mindess, and Bentur 1987b; Banthia 1987). Charpy and drop-weight impact tests require crack length and crack velocity to be correlated with the load acting on the specimen. A resistive type of thin-foil gage has been used effectively on notched specimens (John and Shah 1986). The gage is glued so that the centerlines of gage and notch coincide. As the crack propagates from the root of the notch and a constant current is applied to the gage, the resistance of the gage changes and is recorded as a changing potential (Fig. 2.9).

Using linear elastic fracture mechanics, the stress intensity factor K_I may be calculated from the measured value of the crack length and the corresponding value of the load at which the crack propagates. For the three-point beam test specimen, K_I is given by

$$K_I = \frac{1.5PL}{D^2B} \sqrt{a_o} F(a_o/D) \quad (2-1)$$

where a_o = length of crack; P = applied load; $F(a_o/D)$ is a geometry correction factor (Tada, Paris, and Irwin 1973; Go and Swartz 1983); and L , B , and D are the length, width, and nominal depth of the beam, respectively.

Mindess, Banthia, and Yan (1987) interpreted the results of drop-hammer tests using the expression for critical stress intensity factor or fracture toughness, K_{ID} , given by Broek (1982). They made no corrections for prepeak crack growth, which is consistent with the findings of Rossi et al. (1986) and John and Shah (1986). Whether or not prepeak crack growth is considered, there is very little difference in computed K_{ID} values. The results summarized in Fig. 2.10 show that the dynamic value of K_{ID} is much larger than the static value K_{IC} . The sensitivity of K_{ID} to impact velocity,

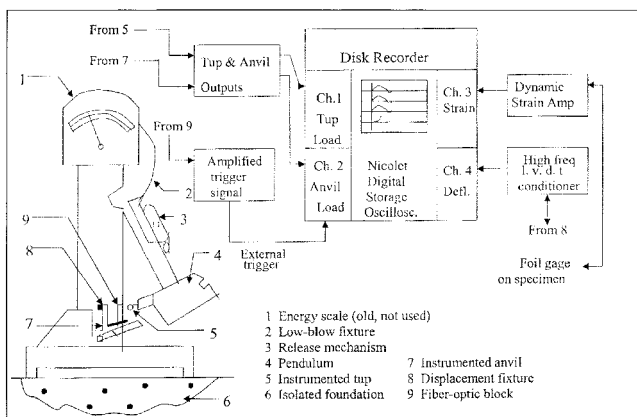


Fig. 2.7—Instrumented impact test device (Gopalaratnam, Shah, and John 1984).

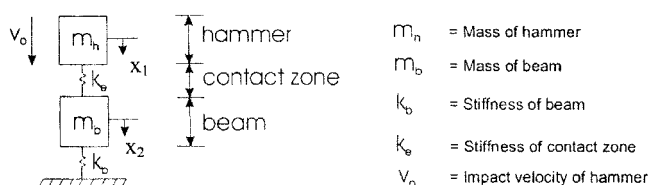


Fig. 2.8—Two-degree-of-freedom model (Gopalaratnam, Shah, and John 1984).

and the kinetic energy of the hammer are consistent with the fracture models of Kipp, Grady, and Chen (1980) and Weerheijm (1984), as reported by Reinhardt (1986). Du, Kobayashi, and Hawkins (1989) report that there is no unique relationship between K_{ID} , fracture energy dissipation, and crack velocity; however, the value of K_{ID} varied with the magnitude of the imposed strain rate.

The conventional LEFM approach of assuming precritical crack growth to be negligible is not appropriate when applied in cases of quasistatic loading (John and Shah 1986); significant precritical growth precedes fracture, and that growth needs to be reflected in a modified value of K_I in Eq. (2-1). The average values of $K_{I\text{LEFM}}$ and $K_{I\text{Modified}}$ are plotted against maximum crack velocity in Fig. 2.11. Note that K_I increases with crack velocity, and that the difference between the $K_{I\text{LEFM}}$ and $K_{I\text{Modified}}$ curves decreases at higher rates. This action is the result of reduced prepeak crack growth with increasing strain rate.

2.1.3 Displacement-controlled hydraulic testing machines—Recent advances in servo-valve technology have made it possible to apply dynamic-type loadings using hydraulic testing machines (Yon, Hawkins, and Kobayashi 1989; Oh and Chung 1989). The system developed by Yon, Hawkins, and Kobayashi (1989) consisted of a specially designed servo-controlled machine with appropriate gages and an accompanying dynamic data acquisition and reduction system. The system was stable for low frequency and high speeds of external loading and could measure the external load and load-line displacement required for numerical analysis as well as the strains at different points over the depth of the specimens. Maximum strain rates were achieved as high as 0.24 s^{-1} for single-edge notched (SEN) three-point bend tests (Yon, Hawkins, and Kobayashi 1992a) and 3.8 s^{-1} for crack-line, wedge-loaded, double cantilever beam (CLWL-DCB) tests (Yon, Hawkins, and Kobayashi 1991b). Moduli of elasticity computed by dynamic two-dimensional FEM increased 50% for the three-point bend tests and 40% for the CLWL-DCB tests, compared with those for static loading, and the load-line displacement at the peak load was not rate-sensitive for either test.

A similar test setup was developed by Oh and Chung (1989) to investigate the effect of loading rate on fracture energy. Concrete beams with various notch depths were subjected to three-point bending. Three different sizes of beams were used to investigate the dependence of fracture energy on specimen size and notch depth. Oh and Chung calculate the fracture energy as

$$G_f = \frac{W_0 + mg\delta_0}{B(H - a_0)} \quad (2-2)$$

where W_0 = the area under the load-deflection curve; δ_0 = the beam deflection when the unloading (or softening) curve returns to zero load; a_0 = the initial notch depth; H = the beam depth; B = beam width; $B(H - a_0)$ = the effective area of beam; and $mg\delta_0$ = a correction for the self-weight of the beam. The fracture energy of concrete, calculated on the

basis of Eq. (2-2), is shown in Fig. 2.12. The fracture energies of unnotched beams are much larger than those of notched beams; this is explained by a large amount of energy being consumed outside the eventual fracture process zone in unnotched beams. Oh and Chung state that the true fracture energy of concrete is the value for the notched beam, especially for relative notch depths between $0.25 < a_0/H < 0.5$. Split-Hopkinson pressure bar (SHPB) tests carried out by Weerheijm (1992) on notched and unnotched specimens led to the same conclusion.

The nominal failure stress σ_n at the root of a notch in a concrete beam may be expressed as

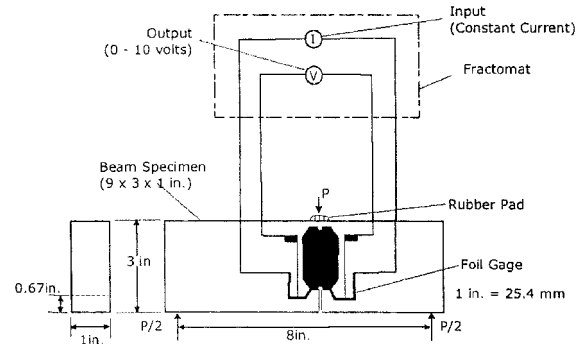


Fig. 2.9—Crack growth measurement using foil gages (John and Shah 1986).

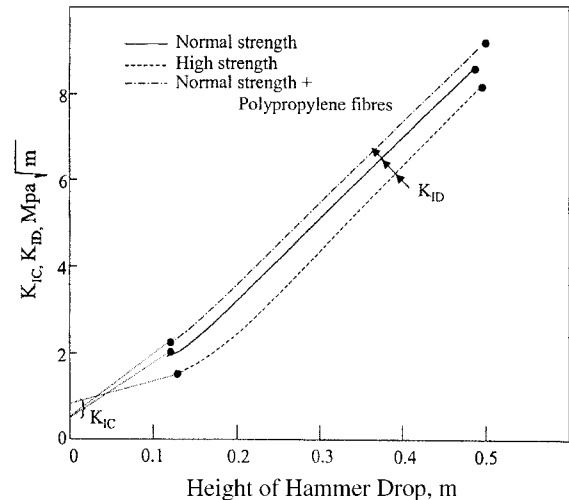


Fig. 2.10—Fracture toughness as a function of hammer drop height (Mindess, Banthia, and Yan 1987).

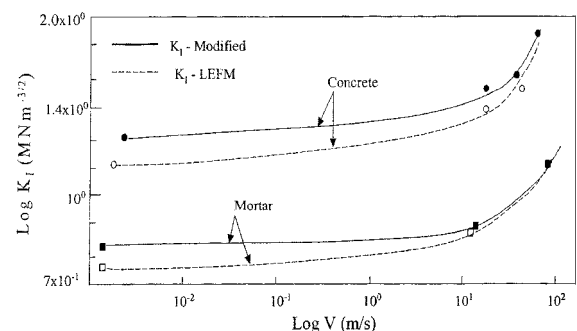


Fig. 2.11—Experimental stress intensity factor K_I (from LEFM) versus maximum crack velocity V (John and Shah 1986).