Concrete

SECOND EDITION

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anything abnormal has occurred). The specimen is then capped and tested at an age of 49 h ± 15 min.

4. *High temperature and pressure method.* This procedure involves the simultaneous application of elevated temperature (150°C or 300°F) and pressure (10.3 MPa or 1500 psi) using special containers. The total curing period is 5 h. The specimens are then tested within 15 min of the end of the curing period.

Interpretation of Accelerated Curing Results The purpose of these tests is to provide a very early indication of the potential strength of the concrete. These tests are generally used to predict the 28-day strengths, on which most design procedures are based. They have about the same variability as conventional tests. Thus, there is no reason why the results cannot be used directly for design purposes, although this would require suitable changes in the strength values now used for design. It must be emphasized that the values obtained from these four accelerated tests are not equal to each other. Neither are they equal to the standard 28-day strength, being generally lower. They are merely different numbers that may be used to evaluate the concrete quality. It should be noted also that there is really no "universal" curve that can be used to obtain 28-day strengths from any one of these accelerated methods. If accelerated tests are to be used as predictors of the 28-day strength, they must be carried out on the same materials. It has been found that the ratio of accelerated strength to 28-day strength increases as the cement content increases and as the initial mixing temperature increases.

14.5 ASSESSMENT OF CONCRETE QUALITY

Core Tests

So far, we have discussed only tests carried out on companion samples of concrete that purport to represent the concrete in the structure, even though we have seen that this relationship can be a tenuous one. However, situations arise where it is desirable to have some measure of the strength of the concrete actually in the structure. This is the case particularly when it is suspected that low cylinder strengths are due to improper specimen preparation. The appearance of cracking or other signs of distress may also warrant an investigation of concrete strength. In addition, if it is desired to use a concrete structure for a higher stress situation than the one it was originally designed for (i.e., trying to increase allowable loads on the structure), a study of both the concrete strength and the position and size of the reinforcing steel may be necessary.

The common way of measuring the strength of the concrete in the structure is to cut cores using a rotary diamond drill (ASTM C 42). These cores (which may contain some embedded steel) are then conditioned for moisture content as described in the standard, capped, and tested in the usual way. As mentioned earlier, if the l/d ratio of the cores is less than 2.0, the core strengths must be corrected by the appropriate factor. Although this seems like a perfectly straightforward way of assessing concrete quality, there are a number of problems in interpreting the strength values obtained:

1. The strengths of cores are generally lower than those of standard cylinders, because the curing of concrete on site (where it is allowed to dry out and is subject to temperature variations) is not as favorable to strength development as curing in a moist room. Also, it is possible that some damage to the concrete occurs due to vibration of the core drill. However, the design considerations are based on standard cylinders rather than on the true strength that the concrete develops in the structure, so it is far from clear what the significance of low core strength is. (Of course, there is considered to be no problem if core strengths are higher than the specified concrete strength.)
2. The ratio of core strength to cylinder strength is not constant. It decreases as the strength of the concrete increases, from a ratio of about 1.0 for cylinder strengths of 20 MPa (3000 lb/in.^{2}) to 0.7 for cylinder strengths of 60 MPa (9000 lb/in.^{2}).
3. The strength of the core will depend on its position in the structure. Generally, cores taken near the top surface of a structural element (or near the top of a lift) are weaker than those at the bottom, simply because of the effects of bleeding and of the settlement of the coarse aggregate.
4. Concrete is anisotropic, since bleeding can cause the creation of a weak cement–aggregate bond under aggregate particles (Figure 9.7, Section 9.1). As may be seen from Figure 14.11, these planes of weakness are always horizontal in the concrete as cast, because of the influence of gravity. Therefore, they will tend to be perpendicular to the applied load (Case I) for specimens cast with the axis of loading vertical, and parallel to the applied load (Case II) for specimens cast with the axis of loading horizontal. Cracks perpendicular to the applied load affect tensile

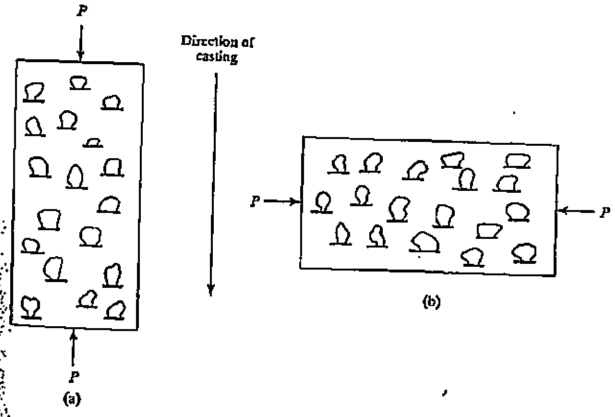

FIGURE 14.11
Planes of weakness due to bleeding: (a) axis of specimen vertical; (b) axis of specimen horizontal.

loads much more than compressive loads; the opposite is true for cracks parallel to the direction of loading. Therefore, planes of weakness due to bleeding will lead to a greater reduction in tensile strength for Case I and a greater reduction in compressive strength for Case II. As discussed in Section 13.4 (see Figure 13.26), it has been found experimentally that the strength of concrete cast with the axis of loading vertical may be as much as 10% higher in compression, with an average of 4 to 8% higher than the strength of concrete cast with the axis of loading horizontal. These values seem to be independent of the mix parameters. Therefore, when trying to evaluate the concrete strength from drilled cores, this effect should be considered when cores are drilled horizontally, as in walls or columns.

5. The core strength is affected by both the moisture content and by moisture content gradients between the surface of the specimen and the interior. The usual conditioning periods recommended in ASTM C 42 (48 hours to 7 days) are generally too short for a uniform moisture content to be established throughout the specimen. It would appear that cores dried in air for seven days are from 5–9% stronger than "as drilled" cores.

To get a proper statistical sample of concrete from the structure under consideration, many cores may have to be obtained. Thus, in addition to the problems of interpretation, cores are expensive and time-consuming, besides leaving holes in the structure that must be repaired.

Nondestructive Tests

A great deal of work has been done since the late 1940s to try to develop rapid, nondestructive tests that would provide a reproducible measure of the quality of concrete in a structure. Many such tests have been proposed, but they all still lead to difficulties in interpretation of the results. While the development of a highly reliable in situ test to monitor concrete quality remains an elusive goal, some of the tests are useful for several reasons: (1) They can be used as a quality control measure, (2) they can help determine the time for form removal, and (3) they can help in the assessment of the soundness of the concrete in existing structures (e.g., after a structure has been damaged by fire). However, it must be remembered that the tests described subsequently do *not* measure concrete strength; rather, they attempt to provide an *estimate* of the concrete strength through correlation with some other property. Unfortunately, as is usually the case in concrete testing, all of these nondestructive tests give results that are affected by a number of parameters—aggregate type and size, age, moisture content, mix proportions, and other variables. Therefore, the correlation between the measured property and strength is different for different concretes and must be determined for the particular concrete in question. The tests are useful primarily for indicating differences in concrete quality from one part of a structure to another. They may thus indicate those portions of the structure that require much closer examination, which will usually involve the drilling of some cores and possibly petrographic or other examinations.

The available nondestructive in situ tests can be classified in a number of different categories. Only a few tests have been adopted as ASTM standards, but this does not imply that the tests listed in ASTM are in any way better or more accurate than the

other tests available; it simply indicates the new tests are being developed more rapidly than the rate at which standards can be proposed, written, and adopted.

Rebound Hardness Probably the most common nondestructive test is the rebound test, using a Schmidt rebound hammer. This device (Figure 14.12a) was developed in 1948 and is universally used because of its simplicity. The test measures the *rebound* of a hardened steel hammer impacted on the concrete by a spring. Although there is no theoretical relationship, empirical correlations between rebound hardness and strength can be obtained (Figure 14.12b). This method is described in detail in ASTM C 805. The results will be affected by the following parameters:

1. *Surface finish of the concrete being tested.* Troweled surfaces give higher values than formed surfaces, and ground and unground surfaces cannot be compared.
2. *Moisture content of the concrete.* Dry concrete gives higher values than does wet concrete.
3. *Temperature.* Frozen concrete will give very high values and must be thawed before testing; the temperature of the hammer will also affect the rebound number.
4. *Rigidity of the member.* Stiffer (more rigid) members will give higher readings.
5. *Carbonation.* Carbonation of the surface can increase the hardness values by as much as 50%.
6. *Direction of impact.* The orientation of the hammer (upward, downward, horizontally, or at an angle) affects the reading since gravity affects how far the hammer mass will move.

The general view held by many users of the Schmidt rebound hammer is that it is useful in checking the uniformity of concrete and in comparing one concrete against another, but that it can only be used to obtain a rough indication of the concrete strength in absolute terms.

Penetration Resistance This type of test involves measurement of the resistance of concrete to penetration by a steel probe driven by a given amount of energy, as described in ASTM C 803. The most common device of this type is the Windsor probe (Figure 14.13). This consists of a powder-activated driving unit that "fires" a probe into the concrete; the depth of penetration (or operationally, the exposed probe length) is measured, and this can be correlated with strength. Since in this technique there is considerable penetration into the concrete, surface texture and carbonation have less effect than in the rebound test previously described. However, mix proportions and material properties are still important, and the device must be calibrated for the material in question. Harder aggregates tend to give higher apparent compressive strengths.

Pull-Out Tests Pull-out tests involve the determination of the force required to pull a steel insert out of the concrete in which it was either embedded *during casting* (Figure 14.14) or installed *after casting* (Figure 14.15). A suitable apparatus for this purpose is described in ASTM C 900. Assuming that the failure surface is a frustum, the nominal normal stress at pullout, f_n, can be calculated using the equations:

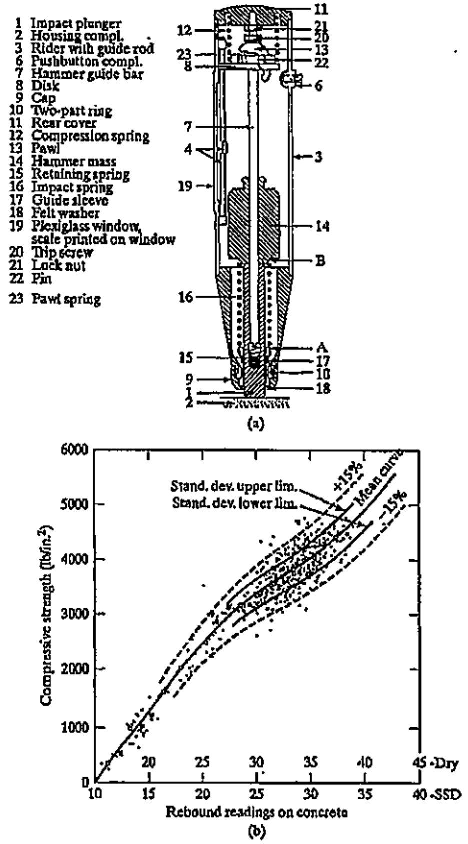

FIGURE 14.12

Schmidt rebound hammer: (a) Longitudinal section of the Type N concrete test hammer (condition on impact). (Reproduced courtesy of Proceq, S.A.); (b) calibration chart for concrete made with crushed limestone and natural sand aggregates. Five hundred standard 6 × 12-in. (150 × 300 mm) cylinders tested SSD at 28 days. Test hammer was calibrated in the horizontal position. Add five points for downward direction; deduct five points for upward direction. [From N. Zoldners, *Journal of the American Concrete Institute*, Vol. 54, No. 2, pp. 161–165 (1957).]

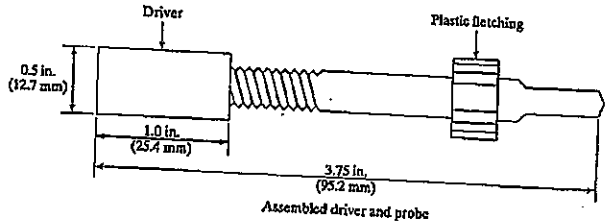

FIGURE 14.13

Windsor probe: assembled driver and probe. [From V. M. Malhotra, Department of Energy, Mines and Resources, Mines Branch Investigative Report IR-71-3 (1971). Reproduced by permission of the Minister of Supply and Services, Canada.]

$$f_n = (P/A)\sin\alpha \tag{14.12a}$$

$$\sin\alpha = (d_3 - d_2)/2s \tag{14.12b}$$

$$A = \pi S(d_3 + d_2)/2 \tag{14.12c}$$

$$S = \sqrt{h^2 + ((d_3 - d_2)/2)^2} \tag{14.12d}$$

where f_n = nominal normal stress, MPa;
P = pullout force, N;
α = ½ the frustum apex angle = $\tan^{-1}(d_3 - d_2)/2h$;
A = fracture surface area, mm²;
d_2 = diameter of pullout insert head, mm;
d_3 = inside diameter of bearing ring, mm;
h = height of conic frustum, mm;
S = slant height of the frustum, mm.

Essentially, this provides some measure of the *shear strength* of the concrete, and this can, for any particular system, be correlated with compressive strength.

The test is economical and rapid, although it does leave a hole in the concrete that must be repaired. It is probably better than those discussed previously, because a greater depth and volume of concrete are tested. The cast-in-place test, of course, must be planned in advance so that the assembly can be embedded in the concrete during casting. In contrast, the post-installed test, can inserted later and used to evaluate existing structures.

Ultrasonic Pulse Velocity This method is based on the fact that the pulse velocity, V, of compressional waves in a concrete body may be related to the elastic properties by the expression

$$V = \sqrt{\frac{E_d(1 - \nu_d)}{\rho(1 + \nu_d)(1 - 2\nu_d)}} \tag{14.13}$$

where E_d = dynamic modulus of elasticity, ν_d = dynamic Poisson's ratio, and ρ = density.

Thermal Diffusivity

The thermal diffusivity measures the rate at which temperature changes take place in the concrete and is a function of both thermal conductivity and specific heat.

$$D = \frac{k}{c\rho} \tag{17.2}$$

where D is the diffusivity constant, k the thermal conductivity, c the specific heat, and ρ the density. Diffusivity is often measured experimentally for the determination of thermal conductivity. Obviously, factors that affect conductivity and specific heat will also affect diffusivity. Typical values for concrete range from 0.002 to 0.007 m²/h (0.02 to 0.08 ft²/h).

Exposure to High Temperatures

From the foregoing discussions, it can be concluded that within the normal environmental temperature range, the thermal properties of a concrete can be considered to be constant, *provided that there is no change in moisture content.* However, at elevated temperatures these properties change because of changes in the moisture content of the concrete components and because of progressive deterioration of the paste and in some cases of the aggregate. These processes depend on the conditions of exposure: the rate of temperature rise, the maximum temperature, and the time at elevated temperatures. The response of concrete will also depend on its initial properties and those of its constituents. Therefore, prediction of concrete behavior at elevated temperatures is a difficult problem.

Strength Unless large temperature differentials are allowed to develop (as in rapid heating), the compressive strength of concrete at elevated temperatures is usually maintained up to about 300°C (570°F). However, above this temperature, significant decreases can be anticipated (Figure 17.6). The magnitude of the decreases depends on the nature of the aggregate and the initial moisture content of the specimen. The changes in both strength and modulus have been attributed to a combination of decomposition of the hydrated pastes, deterioration of the aggregates, and thermal incompatibilities between paste and aggregate leading to stress concentrations and microcracking. The effect on flexural strengths is more marked, and this would be anticipated since flexural strength is more sensitive to the internal microcracking that would be expected to occur. When concretes are cooled back to room temperature before testing, the strength is less than that found if the concrete is tested while hot. This may be due in part to the imposition of additional thermal stresses and also to the rehydration of those hydration products which have been partially dehydrated, thereby causing in situ expansions.

If concretes are heated in a sealed condition to prevent loss of moisture, or when loss of moisture is slow, the cement paste is effectively subjected to high-temperature autoclaving conditions. Over short periods of time, additional reaction of the cement may result in a slight increase in strength, but eventually considerable loss of strength is observed due to the formation of crystalline calcium silicate hydrates, which is accompanied by an increase in porosity.

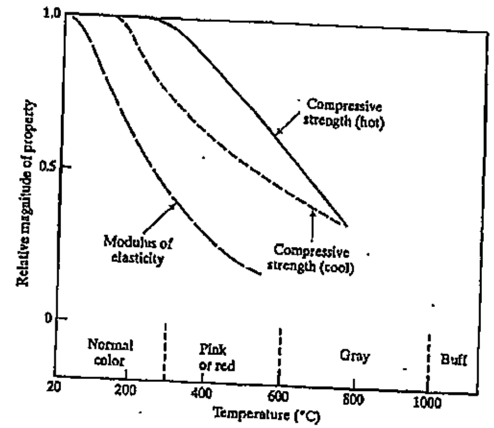

FIGURE 17.6
Effect of heating on strength and modulus of concrete.

Deformations The modulus of elasticity also decreases with temperature (Figure 17.6), and the change is more marked than in the case of compressive strength. The decrease is greater than would be expected from a change in bonding energies, and it is believed that internal microcracking at the paste–aggregate interface contributes to the change in modulus.

As mentioned earlier, drying shrinkage occurs when concrete is heated to elevated temperatures due to additional loss of moisture from the paste (Figure 17.2). The drying shrinkage at 100°C (212°F) is four to five times higher than that at 21°C (70°F) under comparable conditions. Shrinkage continues to increase at higher temperatures (Figure 17.7) as structural decomposition of the hydration products continues, and much of this is irreversible. The rate of shrinkage depends on the rate of moisture loss from the concrete and thus depends on such factors as water content, *w/c* ratio, aggregate content, specimen geometry, and drying conditions at the surface.

Creep will also increase with increasing temperature. Between 50 and 140°C (120 and 285°F), there are conflicting data concerning the magnitude of specific creep and the rate of creep (see Chapter 16). At higher temperatures, the rate of creep can be expected to increase, but the amount of creep should depend on the extent of moisture loss from the concrete.

Fire Resistance Compared to structural steel, concrete has excellent fire-resistant properties and is often used to protect steel from the effects of fire. Concrete has a lower thermal conductivity and a higher specific heat than metals; indeed, its properties