Core Tests: Easy to Perform, Not Easy to Interpret

Many engineers have the experience of ordering the taking of cores. The operation is not difficult, usually undertaken by skilled specialist personnel. Once cut out of the concrete structure, the cores are sent to a testing laboratory for determination of compressive strength, and sometimes for other tests, generally petrographic in nature. The determination of strength is quite similar to the determination of compressive strength of standard test cylinders, and yields in the first place the value of the load in Newtons (or pounds), under which failure by crushing occurs, which is then to be divided by the cross-sectional area of the core in square millimeters (or square inches). Dividing the first of these by the second gives a number in megapascals (or psi); but does this number represent the compressive strength of concrete in the structure from which the core was cut?

The answer is no. Not only must the number be processed, but the resulting value of strength also must be carefully interpreted. Because cores are generally taken when there is a problem, or suspected problem, with concrete, the situation usually involves two or more parties, and they may have different views of what is an appropriate interpretation of the core test results.

Why take cores?

The taking of cores most commonly occurs when the results of tests on standard test cylinders to determine the 28-day compressive strength indicate noncompliance with the specification. Such noncompliance may be due to the fact that the concrete that was placed in a given part of the structure, as well as in the test cylinders, is indeed noncompliant because its 28-day strength is lower than specified. But there may be other reasons: the cylinders may have been incorrectly consolidated (compacted); they may have been damaged in transit, subjected to freezing at a very early age, badly cured, or incorrectly tested; or the resulting compressive strength may have been incorrectly calculated or recorded.

The contractor has reasons to suggest that it is the cylinders that are unsatisfactory, while the concrete in the structure is as specified. On the other hand, the engineer has a professional responsibility to ensure the structural adequacy of the concrete, as well as a responsibility to the client (or owner) to ascertain that the quality of concrete corresponds to the price paid.

It may also happen that while the strength of test cylinders is satisfactory, there are suspicions that the concrete being placed in the actual structure has segregated or has been inadequately consolidated. This can be resolved by the inspection and testing of cores.

There are other situations where taking cores may be desirable, or even essential. For example, it may be required to subject an existing structure to heavier loads than hitherto; or a change of use may be proposed, and the load-carrying capacity of the structure needs to
be verified; or it may be necessary to ascertain that the strength of concrete has not been impaired by overloading or by fatigue, fire, explosion, chemical attack, or some other deleterious agent.

**Where to take the cores?**

The decision on this depends on the question to which the engineer wishes to have an answer. Several questions are possible. The two most common are: does the concrete comply with the specification? and, what is the compressive strength of concrete in the structure in general? To answer this, cores should be taken at a number of locations spread over the whole structure. But there is a caveat: concrete in the uppermost part of any member or concrete lift is nearly always weakest and, therefore, nonrepresentative.

There may also be this question: is the strength of the particularly highly-stressed parts of the structure adequate? In such a case, the locations for core-taking should be carefully chosen so that the load-carrying capacity of the structure is not impaired by removing some concrete from the place where it is most needed. It follows that only an engineer familiar with structural action can identify the appropriate locations.

An engineer would also be aware of where to test to answer the question: is the strength adequate for the actual loading system, even if the strength is not adequate for the designed loading system. In other words, if the structure is not adequate for the original design situation, is it strong enough for more modest loading? The owner may be willing to compromise in such a manner, perhaps by being recompensed by a lower price for the work done than stated in contract. Such a solution may well be acceptable to the contractor in preference to the wholesale removal of the concrete already placed and replacement by new concrete. There is a benefit to the owner, too: the considerable delay due to removal and replacement is avoided. It can be noted in passing that a contractual provision for penalty payment for concrete with a strength lower than specified, but accepted by the engineer, is likely to have a salutary effect on the contractor’s efforts with respect to the quality of concrete.

**How to decide on a testing plan?**

I have already referred to the need for a clear decision on the location of the cores to be taken. In my opinion, this has to be taken by a structural engineer, and not left to the person on the job. This requirement is obvious, perhaps so obvious that it is not considered to be worthy of explicit instructions in codes of practice. The Commentary on ACI 318, Section R5.6.5 wisely says: “The building official should apply judgment as to the significance of low test results and whether they indicate need for concern.” The same section says: “Lower strength may, of course, be tolerated under many circumstances, but this again becomes a matter of judgment on the part of the building official and design engineer.”

This approach is right and proper, but ACI 318 is not normally consulted by a testing contractor who is more concerned with ASTM C 42-99, “Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.” Quite rightly, the ASTM standard does not concern itself with the testing location and plan, or with the interpretation of test results. It is in these respects that I have seen problems arise.

I have encountered problems with testing plans in countries other than the U.S., and, because Concrete International has a large worldwide readership, I feel that they should be aired. Accordingly, I would like to refer to European Standard (which is also a British Standard) BS EN 12504-1:2000. Because this standard was first published in 2000, it is not yet well-known. It is likely, however, that the standard will be used either as is or in a modified form not only in Europe but also elsewhere, mainly in Commonwealth countries.

An important feature of BS EN 12504-1 is that it says explicitly what it does not cover. The Foreword states: “The standard ... does not consider a sampling plan.” Notes to the section headed “Scope” say: “The standard does not give guidance on the decision to drill the cores or on the locations for drilling” and “This standard does not provide procedures for interpreting the core strength results.” These deliberate omissions mean that it is the structural engineer involved in the core exercise who should decide on such matters. I believe this is important because I have encountered cases where the testing laboratory interprets the results of core tests vis-à-vis the specification for construction and vis-à-vis structural safety. The demarcation between testing and structural interpretation should be carefully maintained.

With respect to the choice of the location of the cores, ACI 318 says that cores may be drilled from the area suspected of having a low strength, and expects the core tests to represent that area. British Standard BS 6089:1981, while giving no guidance on the selection of locations of cores, proffers important advice in clause 4.2: “Before any programme (of testing) is commenced, it is desirable that...”
there is complete agreement between the interested parties on the validity of the proposed testing procedure, the criteria for acceptance...”

Following this wise advice obviates subsequent hassle over the interpretation of the core test results. The “agreement” referred to above should not be simply a compromise between the engineer and the contractor; it must take into account the questions that the engineer wishes to have answered. European Standard BS EN 12504-1 states explicitly in clause 5.1: “It is essential that full consideration is given to the aims of the testing and the interpretation of the data, before deciding to drill the cores.”

The word before should be noted.

Why should length-diameter ratio be 2?
The simple answer to this question is: because this is the value of the length-diameter ratio (L/D) of a standard cylinder. However, more should be said. In particular, it is worth giving the reasons for the choice of L/D = 2 as a standard. It is also worthwhile to discuss the situation in cores, because, unlike a test cylinder where the mold controls the value of L/D, the choice of the value of L/D lies in the hands of those involved in testing.

Test programs that led to the choice of L/D = 2 go back a long time. In 1925, Gonnerman wrote: “The 6 by 12-in. cylinder is now generally used for compression tests.” Supporting arguments for the choice of L/D = 2 were given by Murdock and Kesler in 1957. In essence, they showed that the variations in the calculated compressive strength of a cylinder were small for L/D values in the vicinity of 2, and were more pronounced for values between about 1.0 and 1.6 (see Fig. 1). The reasons for this situation lie in the mode of failure of the test specimen: in a squat specimen, the restraining effect of the platens of the testing machine is much more significant than in a more slender specimen. In other words, the use of correction factors necessary to normalize the test results to the value of strength of a standard specimen is greater at smaller values of L/D; it is clearly preferable to minimize the need for correction factors.

The use of cores with the value of L/D = 2 is appropriate only when standard molded test cylinders have that value. In countries using cubes, the core should preferably have L/D = 1; this is recommended in European Standard BS EN 12504-1:2000, which caters both to countries using cylinders and to those using cubes. In the United Kingdom, which uses cubes, BS 6089:1981 requires the value of L/D to be not less than 0.95 before capping, and not more than 1.3.

Although it has not been unequivocally demonstrated to be the case, I am of the opinion that a cylinder (molded or a core) with the value of L/D = 1 has approximately the same strength as a cube whose edge is equal to the diameter of the cylinder.

What are the correction factors for L/D?
The use of correction factors to “convert” the strength determined on a test specimen with a given value L/D to the strength that would be obtained in a test on a specimen with L/D = 2 is well known. But how good are those factors?

Murdock and Kesler tell us that the factors are a function of the level of strength of the concrete. Specifically, stronger concretes are less affected by the value of L/D than concretes of lower strength; this is shown in Fig. 1. According to ASTM C 42-99, concretes with strengths above 70 MPa (10,000 psi) are even less affected by the value of L/D. It follows that the correction factors tabulated in ASTM C 42 and by other standards
In My Judgment

Table 1: Correction factors for L/D according to ASTM C 42

<table>
<thead>
<tr>
<th>Year of ASTM C 42</th>
<th>Value of L/D</th>
<th>Strength correction factor^*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1927</td>
<td>2.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>1.75</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>1.25</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>0.85</td>
</tr>
<tr>
<td>1949</td>
<td>2.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>1.75</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>1.25</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>0.85</td>
</tr>
<tr>
<td>1961</td>
<td>2.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>1.75</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>1.50</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>1.25</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>0.89</td>
</tr>
<tr>
<td>1968</td>
<td>2.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>1.75</td>
<td>0.99</td>
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<tr>
<td></td>
<td>1.50</td>
<td>0.97</td>
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<tr>
<td></td>
<td>1.25</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>0.91</td>
</tr>
<tr>
<td>1977</td>
<td>2.00</td>
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<tr>
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<td>0.93</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>0.87</td>
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</table>

^* The strength of a core with L/D = 2 is equal to the strength determined on a core with the actual L/D multiplied by the correction factor.

are in reality typical values or average values. Using a single set of correction factors, such as that given by ASTM C 42, may overcorrect the test results on cores with a small value of L/D drilled from low-strength concrete; and yet, it is for this type of concrete that an estimate of strength may be particularly important. We can see thus that the use of correction factors increases the uncertainty about the strength of the concrete being tested, as compared with a situation where all the cores have L/D = 2. In other words, if at all possible, tests should be performed on cores with L/D = 2.

It is interesting to note that even the same standard, namely ASTM C 42, in its various editions, gives somewhat different sets of correction factors. These are shown in Table 1, based on a paper by Bartlett and MacGregor. As they pointed out, the "true" value of the correction factor may be a function of the moisture condition of the core at the time of testing. All this militates against the use of cores with varying values of L/D coupled with reliance on correction factors.

There is one other important observation in the paper by Murdock and Kesler, derived from earlier tests: "the data on the specimens whose L/D was less than 0.5 were erratic." This situation is apparent in Fig. 1. Other tests showed that the "correction factor" for L/D = 0.5 was 1.98 for 21 MPa (3000 psi) concrete and 1.68 for 31 MPa (4500 psi) concrete. A value as high as 2.09 is quoted by Murdock and Kesler. It is not surprising, therefore, that BS 6089:1981 states: "Little reliance can be placed on results obtained on cores having length/diameter ratio of less than 0.5." My conclusion from the preceding discussion is that cores with L/D smaller than 1 should never be used, and yet I have seen the use of cores with L/D = 0.5; cores with a diameter of 6 in. (150 mm) were extracted from slabs 4-in. (100 mm) thick or even thinner. Testing such cores could be easily avoided either by taking cores with a smaller diameter or by subsequently subcoring the larger cores. Admittedly, there remains the issue of the minimum diameter of the core to be tested in compression. Of course, there is no problem with cores that are too long: they can be cut so as to achieve the value of the ratio of the capped length to diameter of between 1.9 and 2.1, described as preferred by ASTM C 42. It is only when L/D is less than 1.8 that a correction factor needs to be applied.

Although it is not always stated explicitly, the various correction factors for L/D apply to normal-weight concrete only. According to ASTM C 42-99, they apply also to lightweight concrete with a density between 100 and 120 lb/ft^3 (1600 and 1999 kg/m^3). However, there is some uncertainty about correction factors for concretes of lower density.

What size of cores?

It is considered preferable to use cores with a diameter of 6 in. (150 mm), with the alternative of 4 in. (100 mm) being acceptable. ASTM C 42-99 prescribes a minimum diameter of 3.75 in. (95 mm). It allows the use of smaller diameters when it is impossible to obtain a core with L/D of at least 1, but only for "cases other than load-bearing situations." According to ASTM C 42, the minimum diameter of the cores is governed by the maximum aggregate size: it "should preferably be at least three times the nominal maximum size of the coarse aggregate and must be at least twice the nominal maximum size..." European Standard BS EN 12504-1:2000...
simply says that, when the core diameter approaches a value that is less than 3 times the maximum aggregate size, there is “a significant influence on the measured strength.” An Annex to this Standard gives the relevant data, from which Table 2 has been derived.

The reason for the limitations on the core size is that, unlike a molded cylinder, in a core some coarse aggregate particles are cut in the drilling process and are, therefore, not wholly bonded to the cement paste matrix. The adverse effect of incomplete bond is aggravated by the difference in the modulus of elasticity between the aggregate and the cement paste. When a significant proportion of coarse particles is in that state, some of them may become partially loosened during the test and cease to carry their share of the applied load. An indirect evidence of such behavior is offered by Malhotra, who found that cores with a smaller diameter exhibit a higher variability. This situation is recognized in the assessment of precision of tests on cores of various sizes.

Nonetheless, there are some proponents of the use of cores with a diameter as small as 1.2 in. (30 mm), even when the maximum aggregate size is 3/4 in. (20 mm). The confidence limits of the predicted strength of test cubes from the strength of such small cores are very wide, and I, for one, remain skeptical about the interpretation of the test results of compressive strength of such small cores. It has also been reported that the correction factor for the value of L/D is dependent on the diameter of the core.

**Which moisture condition?**

Cores are usually obtained using water-cooled drills with diamond bits attached to a core barrel so that, when extracted from the parent concrete, the cores are superficially wet. The interior of the core may be wet or dry, depending on the situation of the concrete in service. The question to consider is: in which moisture condition should the cores be tested to ascertain their compressive strength? Different standards and guides offer differing views. ASTM C 42-99 states that the cores “shall be tested in a moisture condition representative of the in-place concrete” or alternatively “as directed by the specifying authority.” There is a section in ASTM C 42 that allows this authority freely to choose any moisture condition. I would like to emphasize the provision for the decision by the “specifying authority,” which usually is the project engineer.

In the absence of a specially chosen moisture condition, ASTM C 42 provides for two conditions. One of these is the “as received condition,” which requires drying for 12 to 24 hours at a temperature of 60 to 80 F (16 to 17 C) and a relative humidity below 50%. The second condition is the “dry condition,” which requires drying at the same temperature but at a relative humidity below 60% for seven days.

The significance of the careful control of the moisture condition of the core at the time of testing lies in the fact that this condition influences the apparent strength of the core; broadly speaking, wet cores record a lower strength than dry cores. The problem is that the difference is variable and uncertain.

This article does not purport to present a literature review, and I shall limit myself to the work by Bartlett and MacGregor. They found that, on average, the strength of cores dried in the air for 7 days is 14% higher than the strength of cores soaked in water for at least 40 hours. The value of 14% appears to apply over the range of strengths from 15 to 92 MPa (2200 to 13,400 psi). The crucial words are “on average” because the scatter about the 14% difference is considerable.

Bartlett and MacGregor point out that the actual situation is more complex than “dry” and “wet”: what influences the magnitude of the difference in strength of cores in the two conditions is the presence of a moisture gradient.

### Table 2:

**Effect of maximum aggregate size (m.a.s) and core diameter on measured strength (based on ref. 3)**

<table>
<thead>
<tr>
<th>Core diameter</th>
<th>Relative strength for m.a.s. (mm)</th>
</tr>
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<tbody>
<tr>
<td>mm</td>
<td>20</td>
</tr>
<tr>
<td>100</td>
<td>1.00</td>
</tr>
<tr>
<td>50</td>
<td>0.94</td>
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<th>mm</th>
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<td>100</td>
<td>1.00</td>
<td></td>
</tr>
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<td>50</td>
<td>0.86</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>0.72</td>
<td></td>
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</table>
between the exterior and the interior of the core.\textsuperscript{13} In the absence of such a gradient, the difference is not 14\% but only 9\%. They state “that the effect of 7 days of air drying is to cause a moisture gradient that artificially increases the strength of the specimen by about 5\% above the true in situ strength.”\textsuperscript{13} Personally, I do not accept that any core test gives us the value of “the true in situ strength.” In a later paper, Bartlett and MacGregor reported that the apparent reduction in strength of wet cores is greater in cores with diameters of 2 in. (50 mm) than in larger cores.\textsuperscript{14}

For these reasons, I am of the opinion that mimicking the moisture conditions in service is too elusive to be useful. In any case, the strength of the core as determined by the compression test does not represent the actual strength of the concrete in the structure in a meaningful way. What the test result gives us is a value that has to be interpreted by the engineer who has to take into consideration a number of factors relevant to the core as well as to the history of the concrete in service. I shall discuss this topic more fully later.

Overall, then, I favor testing cores after immersion in water for a period long enough to prevent the presence of moisture gradients. The advantage of this condition is that it is much more reproducible than a so-called dry condition, where the degree of dryness is uncertain, so that accidental variations in conditions can arise. In this connection, we can note that European Standard BS EN 12504-1 says: “If it is required to test the specimen in a saturated condition, soak in water at 20 ± 2°C (± 4°F) for at least 40 hours before testing.”\textsuperscript{13}

### What else affects core strength?

It has been observed that when drilling is performed in the horizontal direction the resulting strength is lower than for cores drilled vertically downward. A likely reason for this is that, whereas drilling downward allows the drill to be fixed and held firmly in position, drilling horizontally almost inevitably permits a slight movement of the drilling barrel, even when holding-down bolts locate the drill. The consequences of such movement during drilling may be seen as shallow corrugations on the surface of the core.

Another possible reason for the difference between cores drilled vertically and horizontally can be sought in the presence of bleed water. Such water would have no effect when the load applied in the compression test is normal to the lenses of water trapped underneath coarse aggregate particles, but the lenses of bleed water parallel to the axis of the core might result in some weakening.

A reduction in strength of 8\% of the value for vertical cores has been suggested;\textsuperscript{9} this is probably the best correction factor that we have, but it cannot be assumed to be universally valid. In fact, Bartlett and MacGregor found that the direction of the core axis relative to the direction of casting has no effect on the strength of cores taken from high-strength concrete.\textsuperscript{14} Some support of my doubts about the general use of a correction factor for horizontally-drilled cores is given by the recent European Standard BS EN 12504-1:2000, which makes no reference whatsoever to the direction of drilling.\textsuperscript{2} This is in contrast to British Guide BS 6089:1981, which enhances the strength of horizontal cores by 8.7\%.\textsuperscript{4}

Drilling cores in a horizontal direction introduces another factor influencing the strength of the core. This factor is not related to coring but (as already mentioned) is the consequence of the inherent variation in strength of concrete in the vertical direction of a concrete member or a lift, caused by an increase in the actual water-cement ratio in the uppermost layers of concrete. Such differences should not be “corrected” but should be recognized as a fact of life. A usual way to avoid this effect is not to take cores in the uppermost part of the concrete member.

### What happens in slipformed concrete?

Slipformed concrete presents a special case. In my opinion, in such concrete there is no increase in the water-cement ratio with height because the apparent upward movement of water occurs continuously and steadily, owing to the slow progress of placing of concrete. For this reason, I would not expect the formation of lenses of trapped bleed water. A consequence of this situation is that there would be no lowering of the apparent strength of concrete in cores drilled horizontally.

This direction of drilling of cores is inevitable in a silo in service. However, I encountered an exception when a slipformed silo was demolished, and parts of the silo walls were laid horizontally on the ground. Cores were then taken by drilling horizontally, that is, in the direction of placing of concrete. In consequence, although the core is drilled horizontally, the core axis is parallel to the direction of placing of concrete. It is therefore difficult to say whether such a core should be classified as horizontal or vertical.
What is the effect of steel?

Another potential factor influencing the testing of cores and their apparent strength is the presence of reinforcing steel within the core. It is obvious that reinforcement should be avoided wherever possible; on the other hand, in some structures, the density of reinforcement is so great that it may be impossible to avoid it. If this is the case, there is a serious risk of adversely affecting the structural integrity of the given structural member by coring through the steel. I believe that, in such a situation, coring should not be resorted to, and nondestructive methods of testing should be used.

If, for whatever reason, cores cut through steel bars, then the cores can be tested for compressive strength, but only if there are no steel bars parallel to the axis of the core; this requirement is laid down in European Standard BS EN 12504-1:2000.3 British Guide BS 6089:1981 and also the Concrete Society Report No. 54 give correction factors for the presence of one or more reinforcing bars in the core. The British Guide adds that “the in-situ strengths estimated from the [given expressions] cannot be equated to standard cube strengths.”4 I view all this as militating against the use of cores containing reinforcing steel.

How to test for splitting tensile strength?

ASTM C 42-99 also provides for testing cores to determine the tensile splitting strength. The requirements with respect to the cores themselves are the same as for compression testing, with the exception that capping is not to be applied. It follows thus that the value of L/D must not be greater than 2 and must not be smaller than 1. However, no correction for L/D is required. I am stating these requirements because I have seen results on tests on cores with a value of L/D as low as 0.5. Clearly such tests for the splitting tensile strength cannot be considered to conform to ASTM C 42-99. I have discussed the broader question of arbitrary modification of ASTM standard tests in an earlier article.16

Why is it not easy to interpret core tests?

This assertion is included in the title of this article, but, at this stage, the reader might be surprised: after all, I have discussed the various factors influencing the strength of cores and the relevant correction factors. It could, therefore, be thought that all that is needed is to apply the various correction factors and, presto, we arrive at the compressive strength of concrete.

What is the variability of strength?

First of all, there is a scatter in the strength of cores even when taken in close vicinity to one another. Admittedly, there is also scatter in the strength of molded cylinders, but the scatter of strength of cores is larger because coring itself introduces variability. It may be worth repeating that, however carefully performed, the coring operation causes some damage to the core: there may be a weakening of the bond of large particles of aggregate near the surface of the core, and there may be a dynamic effect of drilling, which induces some fine cracks. It is, therefore, best to test more than one core, but this may not be desirable or even possible. The coring operation is expensive, and it may disturb the structure or mar its appearance.

According to Concrete Society Report No. 54,17 the 95% confidence limits on the estimate of actual strength of the cores are as follows: ±12% of the value determined on a single core, and ±6% of the average value of four cores. Overall, the confidence limits are inversely proportional to the square root of the number of cores tested. These figures mean that, when the value determined by test is X, there is a 95% probability that the “true” value lies within 12 and 6%, respectively, of the value determined in the test. This situation should be borne in mind when someone tries to argue that, say, a value of 27 MPa determined on a single core indicates that the expected value of 30 MPa has not been satisfied.

The precision statement in ASTM C 42 says that the results of two tests (each being an average of two adjacent cores) performed by two laboratories should not differ from each other by more than 13% of their average. European Standard BS EN 12504-1 gives no precision statement but acknowledges a greater variability of cores than of molded specimens.3

Which strength?

The first question is: what is meant by strength? It is sometimes forgotten that the intrinsic or “true" strength of concrete cannot be measured. By “intrinsic" I mean the strength of concrete that is independent of the characteristics of the test specimen.

We know that the strength of molded test cylinders cured, treated, and tested under standard conditions (such as those of ASTM C 31) represents, at best, the
I am, therefore, firmly of the opinion that a wholesale application of correction factors is highly undesirable.

The possibility of steam curing or the application of pressure in making the cylinder.

On the other hand, the actual strength is the strength of concrete in place. Consequently, the consolidation has been less good than in a test cylinder: there are bound to be some excess voids. The curing has been less good because the concrete in the interior of the structure could not be kept continuously wet. Likewise, the temperature was not kept constant. It follows that, at the age of 28 days, the actual strength is lower than the potential strength; under some circumstances, the situation may later on be reversed.

We can thus see that the potential strength is of interest when we want to know how good the concrete in the structure is. This difference is crucial to the interpretation of the value of strength determined on the cores.

How not to use the correction factors?

It could be expected that the discussion of various factors influencing the strength of cores earlier on in this article would make it possible to apply various correction factors so as to “convert” the test result into a value of either potential or actual strength. Alas, this is not so.

First, let me deal with the various correction factors. I have tried to point out that the factors are, at best, average values; sometimes, they are a compromise value. It follows that a given correction factor may not be entirely appropriate in the case of the actual mixture used.

More than that: even when the factor is appropriate, it is an average value around which there is some scatter. Thus, it is possible that in a particular case the correction factor used is excessive. It is possible that another correction factor is also excessive. As I see it, applying several factors, one after another, is a risky operation that reduces the accuracy of the prediction. I am saying this because I have seen a situation where the value of compressive strength on a core was sequentially corrected for the value of L/D, then for the diameter of the core, and finally for the moisture condition; and yet, we know that the effect of moisture condition on strength is greater in smaller cores than in larger ones. Therefore, the two corrections should not be applied independently.

I am, therefore, firmly of the opinion that a wholesale application of correction factors is highly undesirable. If the objective of the
In My Judgment

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test were to determine, say, the strength of a core with L/D = 2, a diameter of 4 in. (100 mm), and in a moist condition, then this is how the core should have been taken and tested. If this was physically impossible, then the outcome of the test should have been compared with a core from concrete with known properties and with characteristics similar to those of the test core. On occasion, this may not be possible, in which case cores are not an appropriate method of resolving the given problem.

Can we determine the potential strength?

In seeking to “convert” the strength established by testing the cores into the potential strength of the given mixture, attempts are sometimes made to allow for the age and curing history of the concrete in the structure. I believe that this is an uncertain operation because this history is rarely known reliably. Moreover, any possible incomplete compaction requires a correction as well, and this is not easy. I realize that excess voidage can be assessed visually and that the density of a concrete specimen is affected by the presence of voids, but correcting for these may induce a large error because 1% of voids reduces the compressive strength by 5%.

Because of these uncertainties, a rough-and-ready approach is sometimes used; this may be no less reliable than fine tinkering with corrections for individual factors. Concrete Society Report No. 11 simply states that the estimated potential strength is 1.3 times larger than the estimated actual strength. This is unlikely to be universally correct because, as ASTM C 42 says, “There is no universal relationship between the compressive strength of a core and the corresponding compressive strength of standard-cured molded specimens.” To my way of thinking, it follows that there is no such universal relationship between the estimated strength of molded specimens and the core strength.

I find the approach of ACI 318-99 and of ACI 301-99 to be preferable. They both say that concrete shall be considered adequate (which I understand to mean “as specified”) when “the average of three cores is equal to at least 85% of f’ and no single core is less than 75% of f’.” No allowance for age or curing history is mentioned. This approach is robust and has the merit of simplicity. The Commentary on ACI 318-99 is persuasive; it says, “Core tests having an average of 85% of the specified strength are entirely realistic. To expect core tests to be equal to f’ is not realistic, since differences in the size of specimens, conditions of obtaining samples, and procedures for curing, do not permit equal values to be obtained.” A difficulty that sometimes occurs is that fewer than three cores are available.

The value of 85% in ACI 318-99 is not very different from the value used in BS 6089:1981, which requires the “estimated in-situ cube strength” to be 83% of the specified characteristic strength of concrete, to which the partial safety factor for design strength is applied.

ACI 318-99 applies the “85% correction” to the compressive strength of cores but remains silent on the interpretation of tests on the splitting tensile strength of cores. In my opinion, the “85% correction” should be applied to the splitting tensile strength as well because the factors responsible for this correction apply equally in splitting tension and in compression. Moreover, without a parity of application of the correction, the ratio of splitting tensile strength to the compressive strength applicable to molded cylinders would become distorted in the case of cores; there is no rational reason for such a distortion.

Who decides?

The merits of this simple approach cannot be overemphasized, but a party to a dispute may seek to manipulate the test results to its advantage. Before it does so, it would do well to read Malhotra’s paper, in which he says: “The available test data on cores are full of contradiction and confusion.” He also cites Bloem: “Core tests made to check adequacy of strength in place must be interpreted with judgment. They cannot be translated to terms of standard cylinders strength with any degree of confidence, nor should they be expected necessarily to exceed the specified strength f’.”

Bloem’s reference to “judgment” and the reference in the Commentary in ACI 318-99 to “judgment on the part of the building official and design engineer” lead me to emphasize that such judgment is not the province of the testing laboratory. Perhaps this approach is observed in the U.S., but elsewhere I have seen technicians who, after reporting the results of core tests, yield to the temptation to express views about...
In My Judgment

structural adequacy; to do so, the expertise of the structural engineer is essential.

It must be re-emphasized that testing and interpretation of test results are distinct operations. My view is supported by ACI 301-99, which states: “Core test results will be evaluated by Architect/Engineer and will be valid only if tests have been conducted in accordance with specified procedures.”

As for the next step, if necessary, BS 6089:1981 says, “Action to be taken in respect of a structural member in which the in situ concrete is considered to fall below the level required has to be determined by the engineer.”

Can we reduce the use of cores?

It is well known that all testing involves errors due to chance; indeed, it is chance that is responsible for the stochastic distribution of strength of concrete test specimens about the mean value. This distribution is reflected in the coefficient of variation of test results. Errors are distinct from mistakes caused by action, or absence of action, by the operator. So, chance — or in common parlance, luck — plays a role in testing.

Careful performance of all tests, strict adherence to standard test methods, and soundly-based judgment minimize the element of luck. As Jack Nicklaus (who has never played on a concrete golf course like the one on Das Island in the Arabian Gulf) is supposed to have said: “People tell me I am lucky. The funny thing is the harder I practice the luckier I get.” When we make concrete we should try harder, so that the need for tests on cores is rare.

Nevertheless, perfection is unachievable, and this article has considered the use of cores in cases where there are doubts about the quality of concrete in a structure. Such a situation arises from time to time, and testing cores may be an effective way of resolving the problem. Nevertheless, as we strive to make better concrete and to ensure that it complies with the specification, we should reduce the need for core tests and, hopefully, reach the situation in Mark Twain’s Tom Sawyer Abroad (admittedly, concerned not with concrete but with apples) in which “there ain’t-a-going to be no core.”

References


Selected for reader interest by the editors.

ACI Honorary Member Adam Neville has recently been awarded the ACI Henry C. Turner Medal “in recognition of his contributions to the concrete industry through extensive research and comprehensive experience as a teacher, author, and consultant.” At about the same time, he was elected an Honorary Member of the Concrete Society in the United Kingdom.