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Field Manual for Maturity and Pullout Testing on Highway Structures

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Abstract

This report provides guidance on the use of maturity testing and pullout testing on highway construction projects. The background on the use of these procedures is described, together with advice on the selection, and the correct and safe use of testing equipment. Site Testing and correlation with standard cured cylinders are described, as is the evaluation of the data obtained. Guidance is given on the use of ACI and ASTM documents, and a list of recommended publications is provided for further reading.

Executive Summary

In-place testing of concrete has been used mainly on private sector contracts for buildings and other structures. Maturity testing ensured the safety of the CN Tower as it was slipformed during the winter of 1973. A combination of maturity and pullout testing on that 68-story building made it safe to remove forms from vertical components as early as 11 hours after the concrete was placed, even in cold weather. A critical fast-track program was maintained with these procedures.

On many multistory building projects the use of flying-form systems has been optimized by using in-place testing to complete two to three floors per week, with slabs stripped as early as 24 hours after casting. On three such projects the savings attributed to fast-track programs were $143,000, $1,000,000, and $1,665,000. On one project the need for an extra floor of forms was eliminated with consequent savings of $200,000.

In-place testing has come into more widespread use on transportation projects. It was used to determine compliance with end-result specifications on the Storebaelt bridges in Denmark and on the Channel Tunnel precast concrete tunnel segments.

Demonstration fast-track highway projects by the Federal Highway Administration have used in-place testing in Virginia (1989), Michigan (1989), Iowa (1989), and Oregon (1991).

In the context of highway pavements and structures, in-place testing can be used to make fast-track construction safe and economical. Cost factors, that are pertinent to the contract can be identified. Prompt reopening of pavements after repair, overlay, or replacement, results in reduced overhead and financing costs for all parties to a contract. Earlier reuse or release of formwork, which is a major portion of the total cost of a bridge (50 percent to 60 percent), can produce significant savings, as can the earlier application of post-tensioning in prestressed structures. In cold areas, shorter protection and curing periods can result in lower heating costs.

Because achieving acceptable test results at an early age is critical to the success of the fast-track concept, all parties have an interest in ensuring that the test results will meet the specification. Better compliance with curing and protection requirements is in everyone's interest.

The concrete formulations used in fast-track construction typically contain state-of-the-art cementitious and admixture systems. Some of these were developed and proven on other Strategic Highway Research Program projects. In most cases, the result is a concrete superior to that used traditionally.

Given the trend to end-result specifications, in-place testing provides the agency or the contractor a reliable way to prove compliance at the ages critical to the contract's progress.

This approach to testing is a powerful tool to win bonuses when incentives are offered to contractors. In one case where contractors were given the opportunity to fast-track with in-place testing, the successful contractor offered the owner a discount if allowed to fast-track.

Since there is some innovation involved, and in the interest of fostering a team approach, a pre-bid meeting is a good way to address the use of in-place testing. Questions about the proposed test procedures can be discussed prior to their implementation in the field.

The amount of later-age conventional testing can be reduced with confirmation of the compliance of a concrete mix at early ages.

The maturity and pullout test procedures described in this manual are economical and do not interrupt the contractor's operations in any way. The required equipment is portable, and the results are available on site as soon as the tests are complete.

1. Introduction

Historically hardened concrete has been tested primarily to determine whether the potential strength of the concrete delivered to a site will meet or exceed a value used in the design of the pavement or structure.

The test procedures use beams or cylinders that are made, cured and tested according to standardized procedures. These specimens, while representative of the concrete delivered to the site, are not necessarily representative of the concrete strength in-place because of differences in compaction and curing. Standard-cured test specimens are usually tested at an age of 28 days, while in-place tests can be done at any desired age and are usually carried out at early ages.

In recent years, fast-track construction has created a need for early-age estimates of the strength of concrete in place. Fast-track construction can have considerable economic benefits. Accelerated construction schedules that put a new, repaired, or overlaid pavement into service require adequate concrete strength to withstand traffic loads. Typical applications include localized repairs, replacement of busy intersections, and major slipform paving. Similarly, in structural components, the early removal of forms or the application of post-tensioning, the removal of shores, and, during cold weather, the termination of curing, can have safety implications as well as result in major cost savings.

Test procedures used to determine the in-place strength of the concrete must be reliable. The tests must also be simple, practical, economical, and capable of implementation without disrupting the construction process.

SHRP Contract C-204, Non-Destructive Testing for Quality Control/Condition Analysis of Concrete, evaluated all the available in-place test procedures covered at the time by North American standards, plus several innovative tests. The evaluation included simulated field trials on highway structures and pavement. It was concluded that the maturity method and pullout testing are the two preferred test procedures for estimating the in-place strength of concrete in a highway structure. The factors used to compare test procedures included the accuracy of strength estimation, the within-test variability, the cost of testing, and the practical aspects of use on construction sites.

Maturity testing has been used on North American construction sites since 1970 and pullout testing since 1978. Both test procedures have been standardized by ASTM and are included in the ACI report on in-place testing. Both procedures are simple to use and can be implemented without any interference with construction operations. In addition to saving money and increasing safety, these procedures can lead to a reduction in the numbers of more traditional tests required.

In-place testing may be required by the specifier and carried out by either the owner's forces or agents, or by the contractor as part of quality control to facilitate operations. Who performs these tests depends on the type of specification used — prescription or end — result. Where a prescription-type specification is used, the testing will usually be performed by, or on behalf of, the highway agency. Where an end-result specification is used, it will be in the contractor's interest to make whatever in-place tests are required to confirm the achievement of early strength, avoid penalties, and allow the earliest opening of a pavement or structure. The tests described in this manual can estimate the in-place strength of the concrete beginning a few hours after casting in any form of highway construction.

These procedures are used primarily to determine strength at early ages, usually up to seven days. The tests can provide reliable data to determine whether there is sufficient strength so that formwork and shores can be removed, post-tensioning can be applied, cold weather protection can be removed, or a pavement or structure can be put into service.

This manual is intended for field personnel use and provides practical guidance on using the test procedures. It is based on current practice. Improvements and refinements are constantly being reviewed by the appropriate ACI and ASTM committees. For those who want more information on the background of these tests or the latest published improvements, references are provided in Chapter 6.

Generally, the faster a structure or pavement is completed, the lower the cost. Formwork is a major construction cost. The sooner it can be removed and reused, the sooner post-tensioning can be applied and shoring removed. The structure or pavement can then be put into service, and the overall construction costs are lower.

The safety of the public and the workforce is paramount. The quality of the construction has to be checked to be sure that the owner receives what is paid for. In-place testing, rationally applied and properly carried out, can help achieve these objectives of safety, economy, and quality.

Since in-place tests for construction purposes are usually made at early ages, the factors that affect strength at early ages must be considered. These factors include the mix composition, such as the effects of cement type and pozzolan addition, and the use of retarding or accelerating admixtures. At high temperatures, strength is gained more rapidly, and at low temperatures, more slowly. If the temperature is too low, strength gain will cease.

The use of maturity and pullout testing equipment is explained in detail in the manufacturers' literature. Where the recommendations of this manual differ from those provided by the manufacturers, follow those in this manual. The available equipment at the time of this writing is listed.

Maturity and pullout testing equipment is portable, simple, and can be used at sites remote from testing laboratories. Construction operations that depend on the attainment of adequate strength can be implemented as soon as the test measurements have been made.

2. The Principle of In-Place Testing

The design of reinforced concrete structures is based on the strength of standard-cured cylinders. Thus it is essential that the result of any in-place test is correlated to the strength of standard-cured cylinders.

The principle of in-place testing is shown in Figure 2.1. The correlation between values obtained by the in-place test and the strength of standard-cured cylinders has to be determined before the in-place test is used to estimate strength in the field. Correlations provided by the manufacturers of the testing equipment should not be used unless valid data are available to confirm their validity for the concrete mix being used.

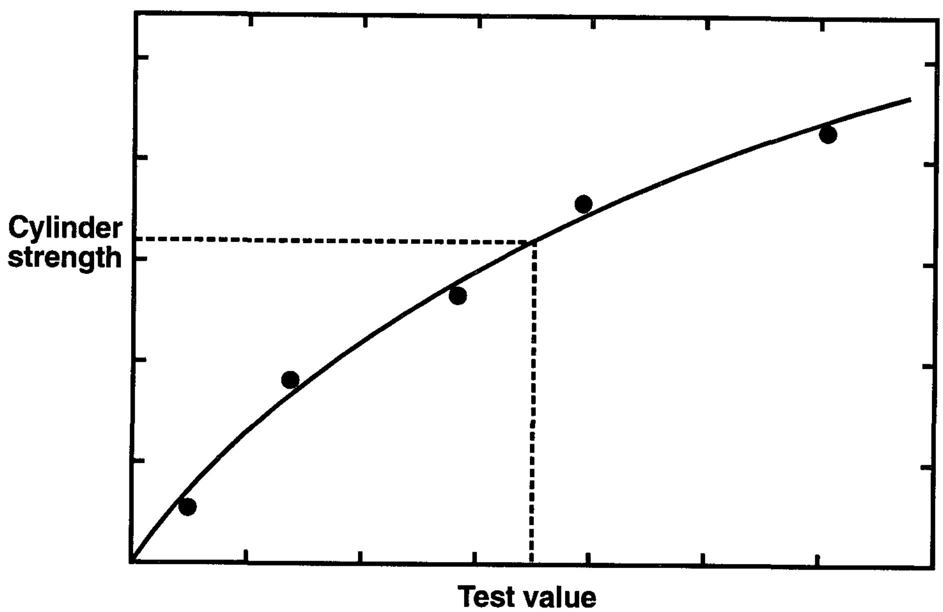

Figure 2.1 Principle of in-place tests

3. Maturity Testing

3.1 General

After initial setting, concrete gains strength over time. The higher the temperature during the early life of the concrete, the faster it gains strength; the lower the temperature, the slower it gains strength. At a very low temperature, generally thought to be in the range of 10°F to 14°F (-12°C to -10°C), hydration, and thus strength gain, ceases. The exact temperature at which strength gain ceases for each concrete mix depends on its composition and the properties of the cementitious materials and chemical admixtures used.

The maturity method is a technique to account for the combined effects of time and temperature on the strength development of concrete. By measuring the temperature of concrete during the curing period, it is possible to estimate the strength at any particular age. The temperature history is used to calculate a maturity index which can be related to compressive strength by a curve such as that shown in Figure 2.1.

The maturity index is calculated from the temperature history by a maturity function. One function, for example, simply computes the product of time and temperature, and is thus expressed in degree-hours. Another function computes the equivalent age, which is the age at a standard temperature that results in the same strength as under the nonstandard condition. The maturity indices can be computed by hand from a recorded temperature history, or they can be computed automatically by a maturity meter. The second method is recommended as a matter of practicality.

3.2 Options for Use

A wide range of maturity meters is available (see Section 7). The hardware available, combined when appropriate with computer hardware, offers a number of options (see Figure 3.1).

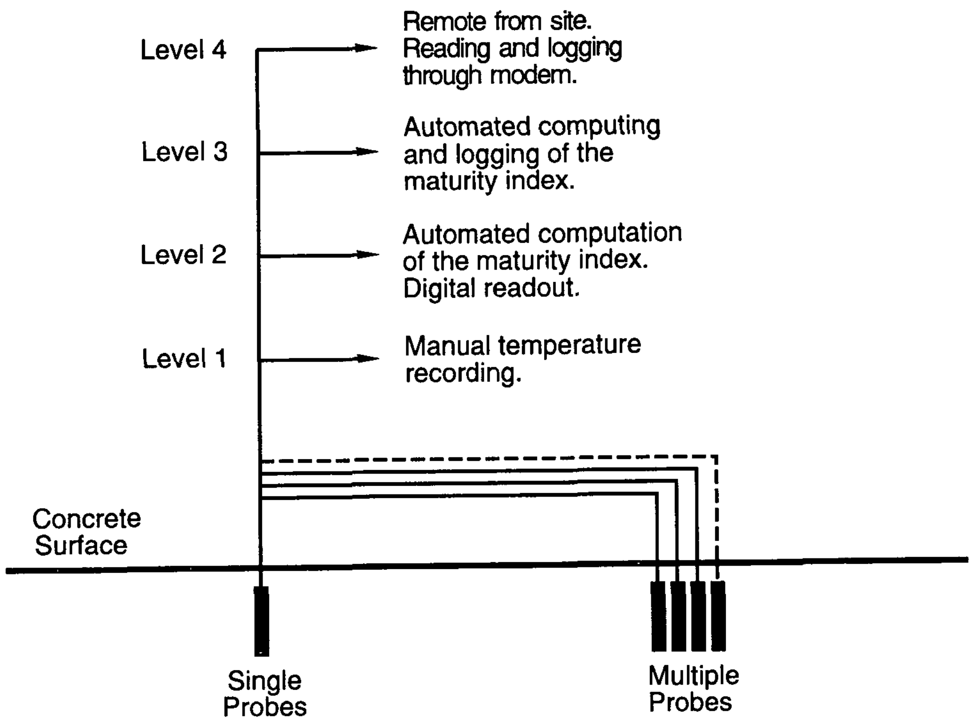

Figure 3.1 Maturity testing options of usage

At the lowest level of sophistication, the maturity index can be determined using a thermocouple wire, recording temperature manually at regular intervals and calculating the maturity index from the recorded data. At the highest level, meters with multiple channels have been remotely connected to off-site locations where the strength gain can be monitored. General practice is to use one of the meters listed in Section 7.

3.3 Limitations

Maturity testing does not measure the strength of the concrete. When structurally critical operations are involved, such as formwork removal or post-tensioning, maturity testing should be used only as a guide to the probable in-place strength. There must be assurance that the in-place concrete is of the correct composition. The actual in-place strength should be estimated by physical tests on pullout inserts cast into the structure. The mixture composition can be determined by early-age testing of standard-cured cylinders cast from the same concrete as the placement being tested (see also Section 5.2).

When pullout tests are intended to confirm the strength estimated from maturity readings, the tests can be performed as soon as the maturity readings indicate adequate strength.

Strength estimates based on maturity readings are valid only if the concrete structure being tested has received adequate hydration throughout the test period. The concrete must not have frozen during the test period.

Commercially available maturity meters use one of two functions to compute the maturity index (Equations 1 and 2 in ASTM C 1074). One of these is the Nurse-Saul function, which computes the product of time and temperature and produces a result expressed in degree-hours (or degree-days). The Nurse-Saul function is based on the assumption that the rate of strength development is a linear function of the curing temperature. The other function is based on the so-called Arrhenuis equation, which means that the rate of strength development is assumed to be a nonlinear function of the curing temperature. Meters based on the Arrhenius function report the number of equivalent hours or days at the standard temperature. In most instruments the standard temperature is assumed to be 20°C.

It is strongly recommended that only meters using the Arrhenius function be used. This is because the Arrhenius function is better able to represent the effects of temperature on strength development than the Nurse-Saul function. Meters using the Nurse-Saul function will increasingly produce less accurate estimates of strength gain as the temperature deviates from the standard temperature used to establish the strength-maturity relationship (see Section 3.8).

3.4 Testing Standards

The use of the maturity method to estimate the in-place strength of concrete is covered by ASTM C 1074. A copy of the latest edition is in Section 8 of this manual. Further guidance is given in ACI 228.1R, a copy of which is in Section 10 of this manual.

3.5 Selection of Hardware

Equipment sources are listed in Section 7. The equipment varies from a disposable one-use meter, to reusable meters with one recording channel, to one meter with 12 recording channels. Selection will be based on site needs. All manufacturers provide after-sales service.

3.6 Calibration

All maturity meters are precalibrated. Checking their calibration on a regular basis does not appear to be necessary. A partial calibration check can be made using the data obtained during a correlation test by comparing the strength and calculated maturity of standard-cured cylinders with meter readings. The operation of the electronics can be checked by putting a sensor in a temperature-controlled water bath for a period of time and checking whether the output agrees with the known result.

The accuracy of the maturity index in representing the true effects of temperature and time on strength development depends on the in-place temperature variation and the constants used in the maturity function. Some maturity meters allow the user to choose the desired constants, while in others the constants are "hard-wired" in the electronic circuits. If the meter allows a choice, use the following values, which are now thought to be generally appropriate.

- datum temperature for Nurse-Saul equation = 0°C (32°F)
- activation energy for Arrhenius function = 40,000 J/mol

When the utmost accuracy is desired, the most precise values for the datum temperature or activation energy can be determined from tests following the procedure in ASTM C 1074.

3.7 Maintenance

See manufacturers' manuals for guidance.

3.8 Correlation with Standard-Cured Cylinders

Before the maturity method can be used, a correlation must be established between strength and maturity index. Follow the procedure detailed in ASTM C 1074 (Section 8). Only the type and make of meter used in the correlation tests should be used on the project for which the correlation is made. After correlation, the datum temperature or activation energy settings on the meters must not be changed.

3.9 Installation on Site

With the exception of the disposable one-use meter, all the maturity meters listed in Section 7 use thermocouple--or thermistor--type sensors. The active end of each sensor can be inserted in the fresh concrete at any location, in an open surface or through very small holes in the forms. The meter can be remote from the test location(s). A sensor should be placed at each location where a strength estimation is required. In cold weather, place sensors at locations expected to experience the lowest temperatures. Generally it is recommended that the sensor be within 2 to 4 inches of the exposed or formed surface of a placement. The SHRP C 204 research project found that if the concrete surface is protected from a high rate of heat loss, the difference in maturity index between the center and surface of a pavement or bridge deck, or the center and surface of a 1-ft (300 mm) thick wall, was negligible. In an overlay, place sensors at mid-depth.

Tie the sensor wire to reinforcement to avoid displacement during concrete placing. In running the wire back to the meter location, consider the possibilities for breakage by construction personnel during concrete placing and subsequent construction operations. In critical placements, use duplicate sensors in different locations with similar curing conditions.

3.10 Number of Measurement Points

Measurement points in Table 3.1 are suggested for guidance. Judgment must be applied on a site-specific basis. Much will depend on such factors as the number of columns concreted in one day, the perceived effectiveness of protection in cold weather, and the sequence of concrete placement. In a group of columns, those exposed to the most adverse curing conditions should be instrumented. In a large pavement or overlay placement, sensors should be located in concrete placed late in the placement, since it will be the weakest.

Table 3.1 Suggested measurement points for pullout tests

Structure Component	Size of Concrete Placement	Number of Sensors
Slabs, beams and abutment walls	100 yd^3	5
Columns	2 - 10 yd^3 >10 yd^3	1 per column 2 per column
Pavement, pavement overlays Pavement repairs	1000 yd^2 per repair	1 1 -2

Note: 1 yd^3 = 0.7645 m^3, 1 yd^2 = 0.8361 m^2

3.11 Operation on Site

Read each meter channel at intervals near the time that the concrete is expected to reach the strength required.

3.12 Evaluation of Data

Use the in-place maturity readings and the strength-maturity relationship to estimate the in-place strength at the location of each sensor.

3.13 Application of Data and Safety

The in-place maturity readings will determine whether enough strength has developed to permit the start of significant operations or to open pavements to traffic. Because the in-place measurement is only of temperature, there is no assurance that the concrete has attained the predicted strength. This is because the maturity method cannot detect errors in batching or curing. In the case of operations such as form removal and the application of post-tensioning, public and workforce safety is involved. It must be re-emphasized that decisions to proceed with these operations should not be made on the basis of maturity tests alone. Supporting physical tests of pullouts or standard-cured cylinders are required on the concrete in or from the placement being tested. See Section 5.

4. Pullout Testing

4.1 General

Figure 4.1 Testing pullout in slab soffit

Pullout testing can be performed using metal discs (called inserts) installed within the formwork prior to concreting, or by inserting an expandable metal disc into an under-reamed hole drilled into hardened concrete. The former is by far the more commonly used pullout test procedure. In early-age testing, the latter procedure would be used only when the preplaced inserts had not been installed prior to concreting.

A schematic representation of the essential features of a pullout test appears in Figure 4.2. The inserts presently in use are 1 in. in diameter, and are fixed 1 in. (25 mm) from the concrete surface. To measure the in-place strength, a loading apparatus is used to measure the force required to extract the insert from the concrete mass. The measured force is used to estimate the in-place strength by means of a previously established correlation relationship.

4.2 Limitations

Pullout testing can be used for testing in the range of 700 to 19,000 psi (5 to 130 MPa). For specified strengths in excess of 6,000 psi (40 MPa), a special high-strength pullbolt is required to apply the failure load to the insert.

4.3 Testing Standards

Pullout testing procedures and requirements using preplaced inserts are covered by ASTM C 900. A copy of the latest edition is in Section 9 of this manual. Pullout testing using expanding discs inserted in hardened concrete is not yet standardized in North America but is in some European countries. Pullout testing of expanding discs is performed in a manner similar to the C 900 procedure and produces similar results. Further guidance on the standard pullout test is given in ACI 228.1R, a copy of which is in Section 10 of this manual.

To date, only one manufacturer produces equipment for drilling into hardened concrete and inserting an expandable pullout insert. This manufacturer provides detailed instructions on the correct procedure, which takes significantly longer to carry out than a test on a preplaced inserts. In early age testing this procedure would be used only if preplaced inserts had been inadvertently omitted prior to concreting.

4.4 Selection of Hardware

Sources of pullout testing equipment are listed in Section 7. The equipment produced by all the suppliers listed complies with the requirements of ASTM C 900. Selection can be made on the basis of price, delivery, and personal experience. Both manufacturers provide after-sale recalibration and repair services.

4.5 Calibration

ASTM C 900 requires that all instruments be recalibrated at least once a year and after any repairs or adjustments to assure that the pullout force is measured accurately. Recalibration is also appropriate any time there is reason to doubt the accuracy of an instrument.

A procedure for recalibrating is given in an appendix to C 900. This procedure does not apply to the pullout testing equipment listed in Section 7. When possible, the owner of the equipment should recalibrate, but if suitable equipment is not available, the suppliers listed in Section 7 provide calibration services.

4.6 Maintenance

Routine maintenance includes keeping the testing machines clean and periodically oiling moving parts. The testing machines should not be subjected to shock. The equipment should be kept in its carrying case when not in use. Manufacturers' manuals supplied with the equipment provide advice on field maintenance. Major maintenance and repairs should be carried out by the manufacturer.

4.7 Correlation with Standard-Cured Cylinders

For each concrete mix to be tested in-place, a correlation must be established before testing starts between the force required to load the pullout insert to failure and the compressive strength (ASTM C 39) of standard-cured cylinders.

A correlation should include tests using at least six strength levels with a strength range in excess of 3,000 psi (20 MPa). This can be achieved by casting twelve standard-cured cylinders and testing pairs at ages of 1, 2, 3, 7, 14, and 28 days. At the same ages, sets of eight replicate pullout tests are made. The pullout inserts are cast in 8-in.(200 mm) cube molds with the same concrete used to cast the standard-cured cylinders. Four pullout inserts are cast in each of two cubes for each test age.

The test cubes must be cured under conditions identical to the standard cured cylinders so that both types of test specimens have the same maturity.

Pullout inserts should be placed at mid-height in the four sides of the cube mold. Each insert is installed with a circular metal plate 3-in. in diameter to provide a flat surface normal to the main axis of the insert, which will ensure axial loading during testing. The concrete in the 8-in.(200 mm) cube molds must be compacted to the same degree as that in the standard cured cylinders. Because the circular metal plates provide a flat surface normal to the loading direction, the 8-in. (200 mm) molds can be made from plywood, as shown in Figure 4.3.

Linear regression is commonly used to determine the relationship between the averages of the sets of cylinder tests and sets of pullout tests at the six test ages. The analysis results in values for a slope and an intercept for subsequent use to estimate the in-place strength (See Figure 4.4). This analysis can be made using either a hand-held calculator with linear regression capabilities or a computer spreadsheet.

The linear regression procedure is common practice but is not as accurate as more sophisticated procedures that take into account the variability of both X and Y values.

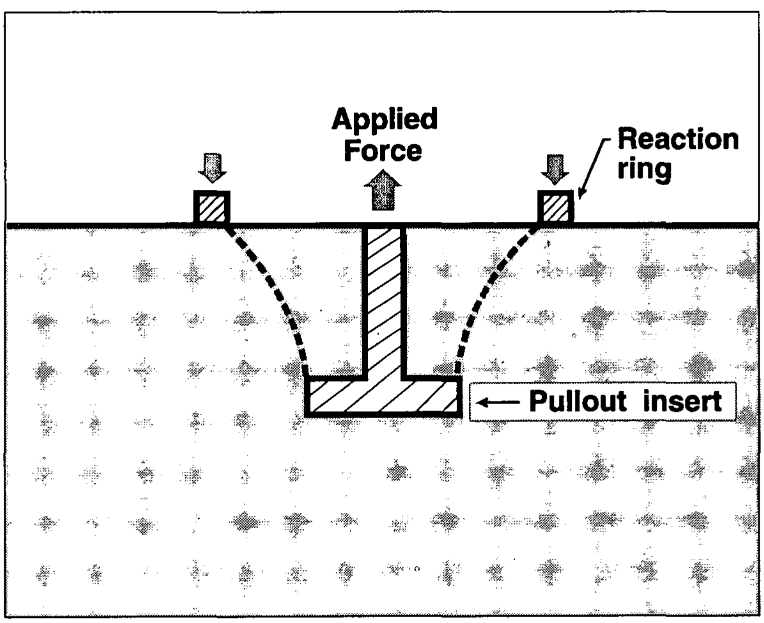

Figure 4.2 *Schematic of pullout test*

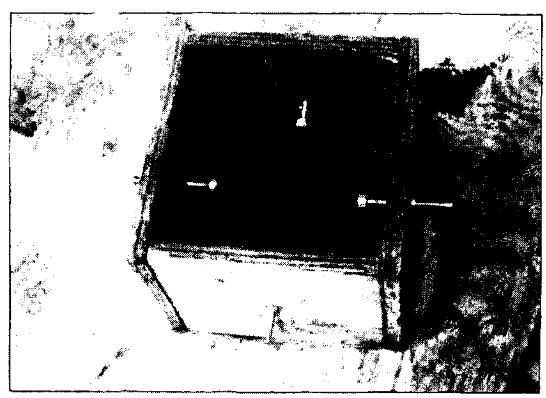

Figure 4.3 Mold for pullout tests

On the basis of a regression analysis, estimated strengths can be calculated for a range of pullout loads and a table prepared similar to Table 4.1. Note that Table 4.1 is given only as an example of how to set up a table. A similar table has to be produced for each pullout tester used and for each concrete mix.

Each correlation is valid only for the mix tested and the pullout machine used in the correlation tests. If more than one machine is to be used, the correlation data must be converted using the latest calibration data for each machine to be used.

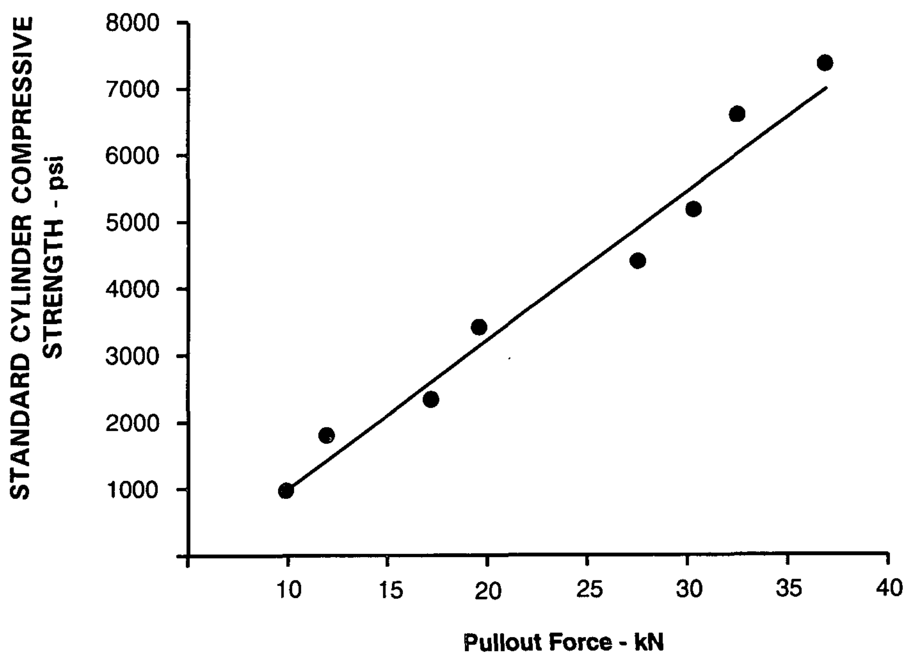

Figure 4.4 Typical relationship between pullout force and the compressive strength of standard cured cylinders

Table 4.1 Sample Pullout Conversions
Date ____
Machine Number ____

Dial Reading (kN)	Compressive Strength (psi)	Dial Reading (kN)	Compressive Strength (psi)	Dial Reading (kN)	Compressive Strength (psi)
5	410	25	3260	45	6720
6	570	26	3440	46	6900
7	710	27	3630	47	7080
8	860	28	3790	48	7240
9	1000	29	3960	49	7430
10	1130	30	4130	50	7600
11	1290	31	4310		
12	1440	32	4480		
13	1580	33	4660		
14	1730	34	4830		
15	1860	35	4990		
16	2020	36	5180		
17	2160	37	5350		
18	2290	38	5510		
19	2450	39	5700		
20	2580	40	5860		
21	2740	41	6030		
22	2890	42	6220		
23	3020	43	6380		
24	3100	44	6560		

Note: The above values are examples only, and are not to be used in practice; 1000 psi = 6.89 MPa

4.8 Installation on Site

The use of pullout tests is limited to structures. Inserting pullouts manually into the horizontal surfaces of unset concrete of low workability, such as freshly placed pavements, can produce unreliable rest results and is not recommended.

Pullouts can be installed in the soffits of beams and slabs, and in the sides of beams, columns, piers, and abutment walls. Typical installations are shown in Figures 4.5 and 4.6.

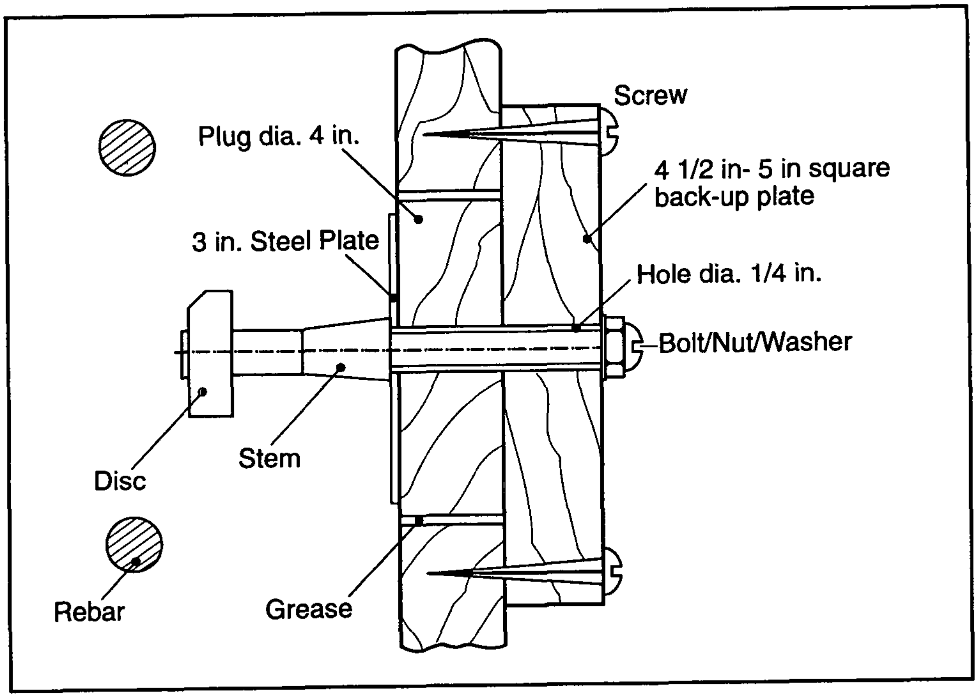

Figure 4.5 Installation of pullout insert in wooden formwork with removable access plug

(Note: 1 in = 25.4 mm)

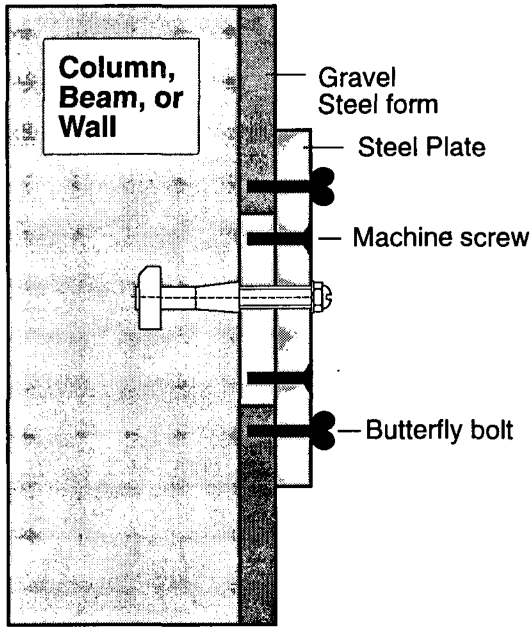

Figure 4.6 Installation on vertical surfaces with steel forms

Inserts should not be placed closer than 1 ft (300 mm) from the top or bottom of a beam, column, or wall placement. The minimum clear spacing for inserts is given in Clause 6.1 of ASTM C 900. The operation of a currently available pullout testing machine requires a clear space at least 18 in. (460 mm) in diameter.

In a soffit installation in which timber forms are used, an access hole is cut by using a circular saw bit (Figure 4.7). The circular plug (Figure 4.8) is then fixed to a square piece of plywood about 4 1/2 in. (110 mm) square (Figure 4.9). The assembly is drilled in the center to take the bolt that holds the pullout insert and the circular metal plate in position. The corners of the backup plywood are drilled for four screws, which are used to fix the assembly to the bottom of the form. The assembly fits into the hole in the form (Figure 4.10). This is attached to the form as shown in Figures 4.11 and 4.12.

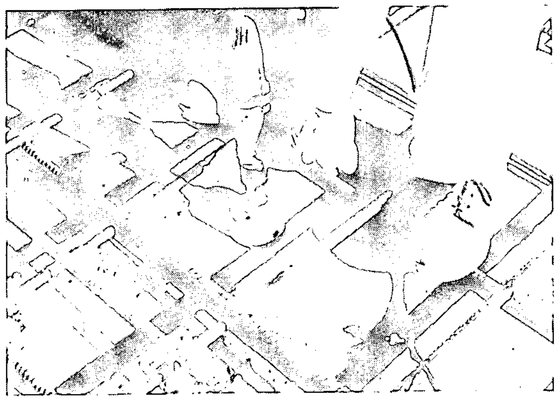

Figure 4.7 Use of a circular bit saw to cut an access hole in a soffit form

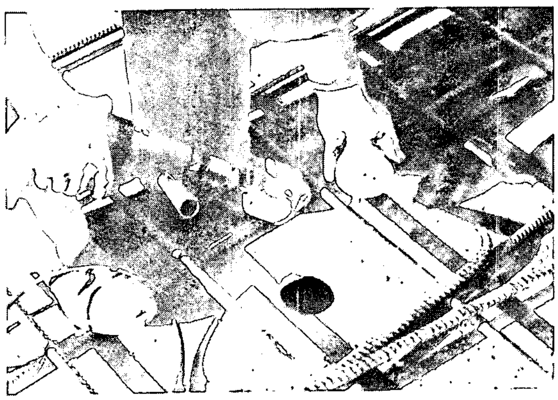

Figure 4.8 Circular plug obtained by use of a circular saw

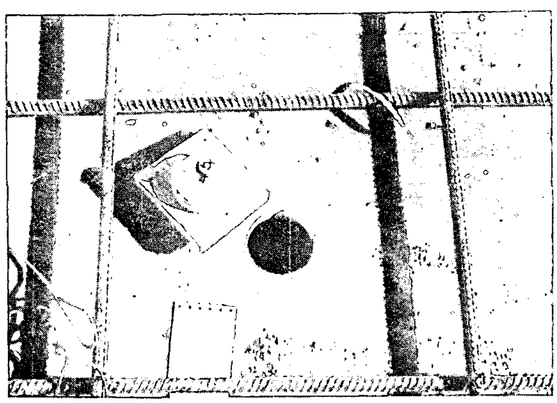

Figure 4.9 Circular plug affixed to plywood square with bolt, pullout insert, and circular metal plate

Figures 4.7 - 4.12 Installation sequence for pullout insert in the soffit of a slab

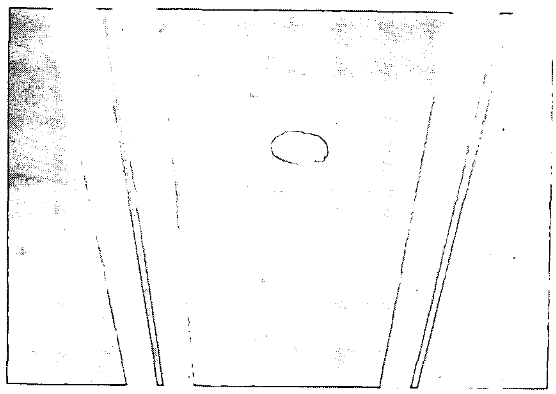

Figure 4.10 Reverse side of timber form

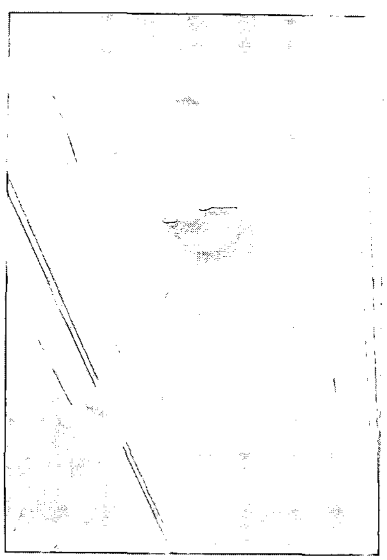

Figure 4.11 Attachments of pullout assembly to reverse side of form

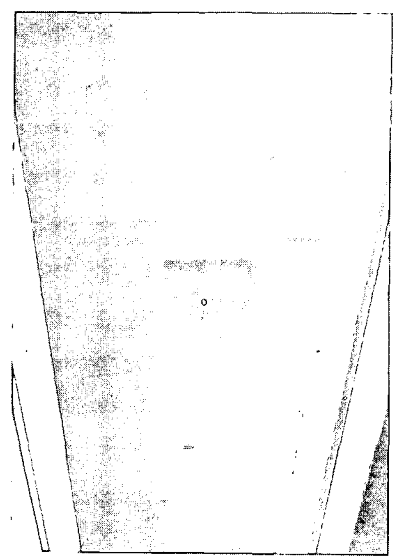

Figure 4.12 Pullout asembly screwed into place

Figure 4.13 Application of grease to fill the gap between the pullout assembly and the timber form

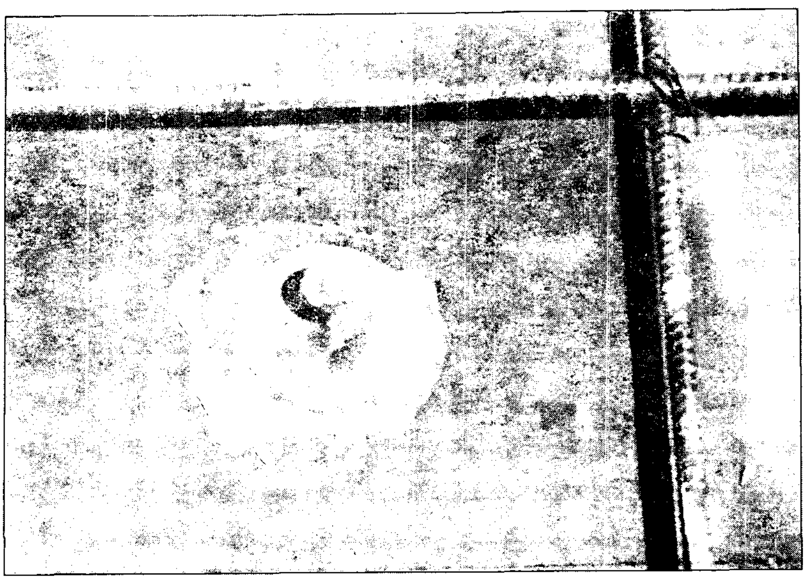

Figure 4.14 Excess grease has been removed from the gap

In the soffit form installation, the plug is a loose fit. Before concreting, the gap around the plug should be filled with grease (Figures 4.13 and 4.14). The surplus grease is then removed. As concreting proceeds, inserts are buried in the concrete (Figure 4.15).

Where the forms are to be removed before the pullout inserts are tested, an access hole is not required. The pullout insert and circular plate can be fixed to the form face with a 1/4 inch (6 mm) bolt through a 1/4 inch (6 mm) hole drilled in the form (Figure 4.16).

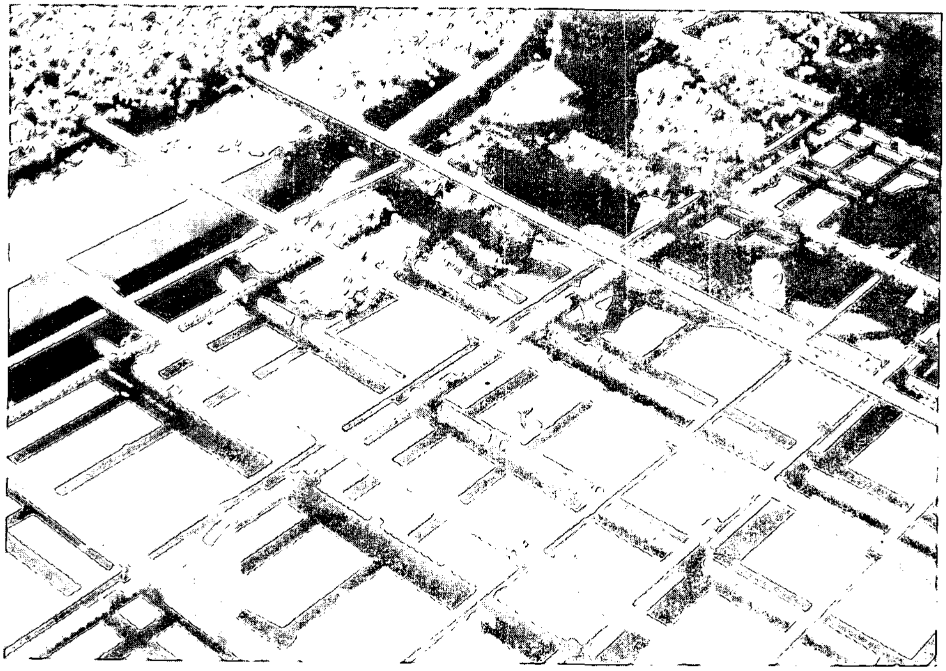

Figure 4.15 Concrete is poured over the installed pullout assembly

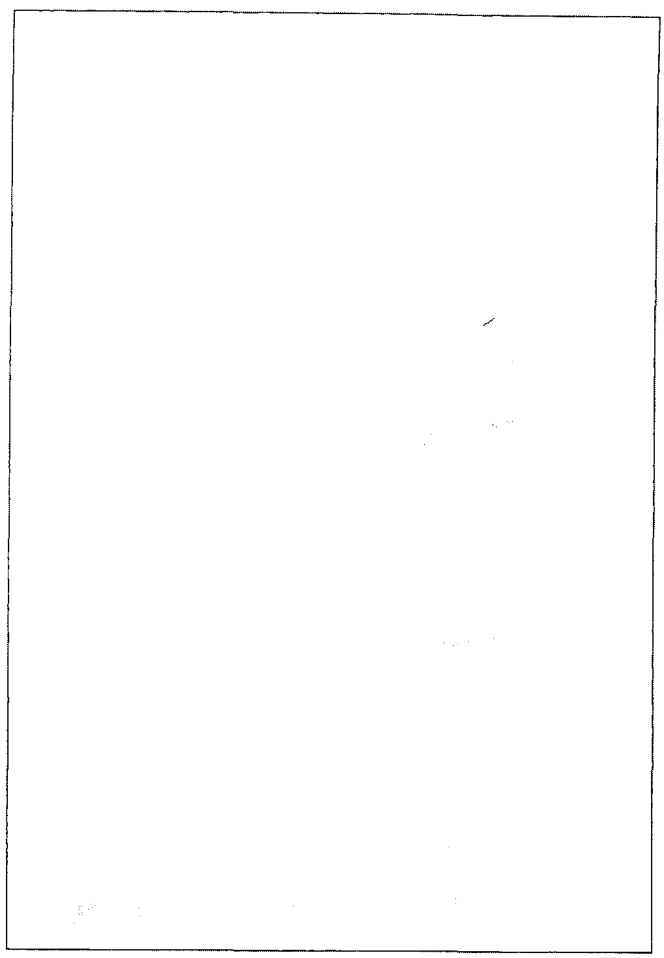

Figure 4.16 A pullout asssembly installation where forms are to be removed before pullout evaluations are to be performed

4.9 Number of Tests

The following are the minimum number of tests that should be made. Pullout inserts are inexpensive. The more that are preplaced in a concrete placement, the more flexibility there is as to when and how many tests can be made. If unforeseen cold weather or the failure of curing and protection produces unacceptably low test results at the preplanned testing time, repeat tests are possible if extra inserts have been installed.

Table 4.2 Minimum number of pullout tests for various structural components

Structural Component	Size of Concrete Placement cu.yd.	Number of Pullout Inserts Provided	Minimum Number of Inserts Pulled per Determination of Strength
Slabs and beams	100	15	10
Abutment walls and pier caps	100	5-10	3-6
Columns	2-10 >10	2-4 4-6	2 2-4

(Note: 1 $yd^3 = 0.7645\ m^3$)

4.10 Operation on Site

See appropriate manufacturer's manual.

4.11 Evaluation of Data

The ACI 228.1R.89 report, "In-place Methods for Determination of Strength of Concrete," describes three procedures for the analysis of test results. The most commonly used procedure is recommended. The minimum strength to allow construction operations to proceed will be established by the project engineer.

The following simple method of evaluation produces a slightly conservative but economic value for the minimum strength of concrete in a placement. It is thus a safe method. First, the individual pullout test results are converted to equivalent values of compressive strength using the previously established correlation. The mean and standard deviation of the set of pullout test results are determined with a handheld calculator. The following is a sample calculation.

	Equivalent Compressive Strength (psi)
	4250
	3570
	3700
	3840
	4230
	4410
	3950
	4640
	3260
	4050
Mean (X) =	3990

Standard deviation = 414 psi

Note: 1000 psi = 6.89 MPa

The minimum strength in the concrete placement is then calculated as follows:

Minimum Strength = X minus Ks psi
= 3990 minus (1.67 x 414 psi)
= 3300 psi

The constant K varies with the number of pullout tests made. The greater the number of tests, the smaller the constant and the higher the calculated minimum strength. It is therefore advisable pays to make a larger number of tests. The risk to the contractor and the project owner is reduced, and cost savings are increased.

The value of K for the range of numbers of pullout tests likely to be used in one concrete placement is as follows. Five is the minimum number required by ASTM C 900.

Number of tests	(n)	3	4	5	6	7	8
	K	2.50	2.13	1.96	1.86	1.79	1.74
	(n)	9	10	11	12	13	14
	K	1.70	1.67	1.65	1.62	1.61	1.59
	(n)	15	16	17	18	19	20
	K	1.58	1.57	1.55	1.54	1.54	1.53

For pavement contracts the project engineer may elect to use the average of the test results instead of the minimum strength calculated as above.

4.12 Application of Data and Safety

If the value of minimum strength determined by the procedure in 4.11 equals or exceeds the value specified by the project engineer then it is safe to proceed with the construction operation controlled by this testing. However, some precautions must be taken to ensure the validity of the test results.

First, it is important to be sure that the concrete has not been frozen at any time prior to the test and is not frozen at the time of the test.

Second, the minimum value calculated must be checked independently. This can be done by phoning an authorized engineer or technician at the head office, repeating the individual test results, and having the calculation of minimum strength checked. The construction operation can then proceed using documentation described in Section 4.13.

4.13 Sample Forms

Model forms are given in Section 3.5.5 of ACI 228.1R (Section 10 of this manual).

5. Combined Maturity-Pullout Testing

5.1 For non-structural applications such as pavement overlays or repairs, the use of maturity measurements alone is adequate to give an estimate of in-place strength, provided an initial correlation has been made with the concrete mix in use.

5.2 Where structural safety considerations apply, as when forms are removed or post-tensioning is applied, maturity tests should not be used alone. It is necessary to verify that the in-place concrete has the required strength potential. Separate standard-cured cylinders made from the same concrete placement can be tested before an in-place strength determination is required. The results can then be used to interpret the maturity data.

The standard-cured cylinders are used to ensure that the concrete in the placement has the correct composition. When the correlation tests are done, the early-age strength of the target mixture is established. Early-age compressive strengths of standard-cured cylinders made from the same concrete placed in the structure will indicate whether the in-place concrete is similar to the target mixture. The ratio of the early-age strengths of the field and laboratory concretes can be used to adjust the strength estimated by the maturity method.

A recommended technique is to combine maturity tests with pullout tests. Maturity readings can be taken a number of times without any loss of testing locations. When maturity readings indicate that the required strength has been reached, pullout tests can be made on the same placement to confirm that the required strength has been reached. This is a safe and efficient procedure and can reduce the number of pullout inserts required. Because pullout tests are not made until maturity tests have establishedthat the required strength has been achieved, there is no wastage of pullout inserts resulting from premature testing.

The recommended interrelationship of maturity and pullout tests is shown in Figures 5.1 - 5.3.

If the combined method is used, both test procedures can be correlated with standard-cured cylinders in one correlation test program.

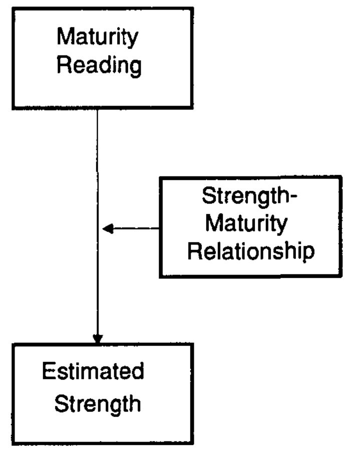

Figure 5.1 Non-structural members

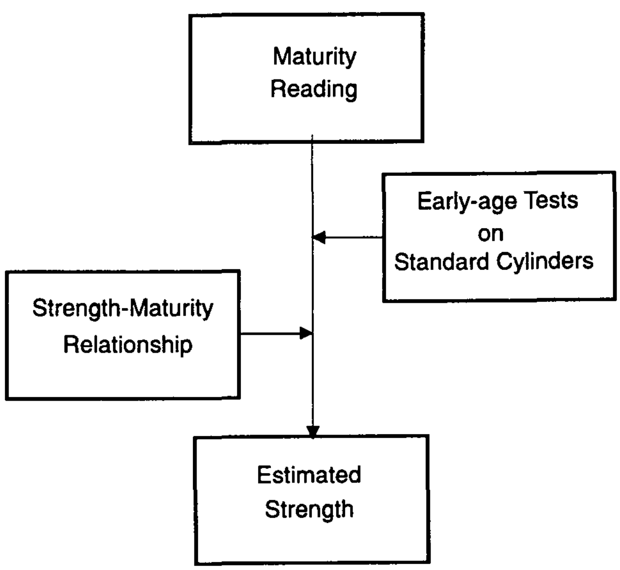

Figure 5.2 Structural members

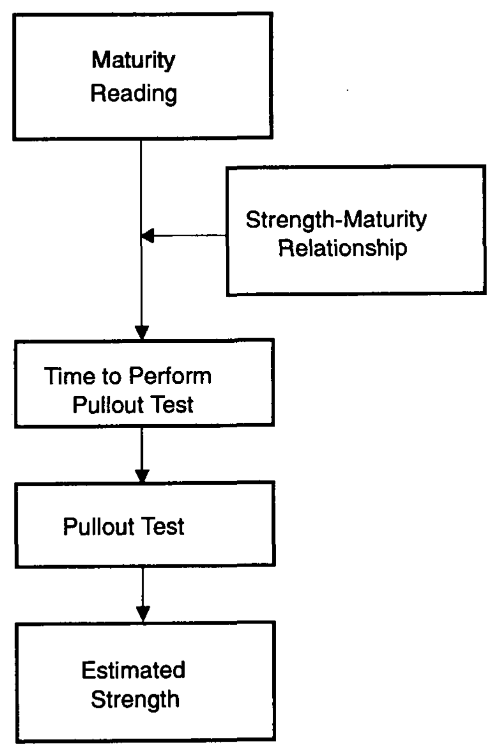

Figure 5.3 Structural members

6. Recommended Further Reading and Bibliography

This manual includes copies of ASTM C 1074, ASTM C 900, and ACI 228.1R, and provides all the information necessary to perform and interpret maturity and pullout tests. For those who wish a better understanding of these procedures, the following documents are recommended.

Handbook on Nondestructive Testing of Concrete. V.M. Malhotra and N.J. Carino, eds. CRC Press, Boca Raton, 1991

This is the most up-to-date text book on the subject.

In Situ/Nondestructive Testing of Concrete. V.M. Malhotra, ed. ACI SP82, Detroit, 1984

These are the proceedings of an international conference in Ottawa, and contain some important papers on maturity and pullout testing.

Four annotated bibliographies of papers on the nondestructive testing of concrete have been prepared by the Canada Centre for Mineral and Energy Technology (CANMET) and can be obtained from

CANMET
Energy Mines and Resources Canada
405 Rochester Street
Ottawa, Ontario K1A 0G1 Canada

The four documents cover papers from 1934 to 1991 as follows:

Part I: *1975-1984 CANMET Special Publication* SP 85-5E, 1985

Part II: *1934-1974 CANMET Mineral Sciences* Division Report MSL 89-127(R)

Part III: *1984-1989 CANMET Division Report* MSL 91-25(R)

Part IV: *1989-1991 CANMET Minerals Sciences Laboratories*
June 1992

These documents include a brief summary of many of the papers published on maturity and pullout testing during the period 1934-1991.

The following important paper has been published since the compilations by CANMET.

Carino, N.J., and Tank R.C. "Maturity Functions for Concretes Made with Various Cement and Admixtures," ACI Materials Journal, Vol 89, No. 2, March-April 1992, pp 188-196.

7. Sources of Hardware, Product Sheets and Manufacturers' Manuals

7.1 Sources of Maturity Meters

7.1.1 Reusable Meters

Supplier	Meter	No. of Channels	Maturity Function
James Instruments Inc. 3727 North Kedzie Avenue Chicago IL 60618 Phone 1-800-426-6500 312-463-6565 Fax 312-463-0009 Telex 206729 # U.D.	M3004	6	Saul (-10°C datum)
	M3056	6	Arrhenius (adjustable) 22,000-56,000 J/mol
Sartell Instruments Ltd. 225 Traders Blvd. East, Unit #3 Mississauga ON L4Z 3E4 Canada	4101	4	Arrhenius or Saul (-10°C but datum adjustable)
	1101	1	Saul (-10°C but other values on request)
Humboldt Mfg. Co. 7300 W. Agatite Ave. Norridge IL 60656 Phone 708-456-6300 Fax 708-456-0137 Telex 28-3455	4101	4	Saul (-10°C but datum adjustable) a
	1101	1	Saul (-10°C)

Supplier	Meter	No. of Channels	Maturity Function
Gilson Company, Inc. P.O. Box 677 Worthington OH 43085-0677 Phone 1-800-431-5935 614-548-7298 Fax 614-548-5314 Telex 241211	HM136 HM135	4 1	Saul (-10°C but datum adjustable) a Saul (-10°C)
SDS Company P.O. Box 844 Paso Robles CA 93447 Phone 805-238-3229	Control Box	4	Arrhenius (activation energy not stated)
M & L Testing Equipment Co. Ltd. 27 Dundas Street East Hamilton ON L9J 1B1 Canada Phone 416-689-7327 Fax 416-689-3978	Control Box	4	Arrhenius (activation energy not stated)
Digital Site Systems Inc. 4516 Henry St., Suite 305 Pittsburgh PA 15213 Phone 412-687-2475 Fax 412-687-7517	CIMS	12	Arrhenius (adjustable activation energy values)

Note: °F = °C x 9/5 + 32

- • See Standard Scientific Inc. brochures in Section 7.3 for details of these meters
- a Can be obtained with Arrhenius function on request
- ° See Skanska brochure in Section 7.3 for details of these meters

7.1.2. Disposable Meters

Supplier	Meter	No. of Channels	Maturity Function
Germann Instruments, Inc. 8845 Forest View Road Evanston IL 60203 Phone 708-329-999 Fax 708-329-8888	COMA Meter	1	Arrhenius
M & L Testing Equipment Co., Ltd. 27 Dundas Street East Hamilton ON L9J 1B1 Canada Phone 416-689-7327 Fax 416-689-3978	COMA Meter	1	Arrhenius
Pullout Testing Ltd. 97 Lamar Street Maple ON L6A 1A7 Canada Phone 416-832-3524 Fax 416-832-3524	COMA Meter	1	Arrhenius

7.2 Sources of Pullout Testing Equipment

7.2.1. Using Pre-placed Inserts

James Pullout Test System

James Instruments Inc.
3727 North Kedzie Avenue
Chicago IL 60618
Phone 1-800-426-6500
312-463-6565
Fax 312-463-0009
Telex 206729 U.D.

Sartell Instrumentation Ltd.
225 Traders Blvd. East, Unit #3
Mississauga ON L4Z 3E4
Canada
Phone 416-890-1090
Fax 416-890-1744

Lok-test

Germann Instruments Inc.
8845 Forest View Road
Evanston IL 60203
Phone 708-329-9999
Fax 708-329-8888

M & L Testing Equipment Co. Ltd.
27 Dundas Street East
Hamilton ON L9J 1B1
Canada
Phone 416-689-7327
Fax 416-689-3978

Pullout Testing Ltd.
97 Lamar Street
Maple ON L6A 1A7
Canada
Phone 416-832-3524
Fax 416-832-3524

7.2.2. *Inserts Inserted in Drilled Holes in Hardened Concrete*

Capo-Test

Germann Instruments Inc.
8845 Forest View Road
Evanston IL 60203
Phone 708-329-9999
Fax 708-329-8888

M & L Testing Equipment Co. Ltd.
27 Dundas Street East
Hamilton ON L9J 1B1
Canada
Phone 416-689-7327
Fax 416-689-3978

Pullout Testing Ltd.
97 Lamar Street
Maple ON L6A 1A7
Canada
Phone 416-832-3524
Fax 416-832-3524

Appendix A

ASTM C 1074

Designation: C 1074 – 87$^{\epsilon 1}$

AMERICAN SOCIETY FOR TESTING AND MATERIALS
1916 Race St., Philadelphia, Pa. 19103
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If not listed in the current combined index, will appear in the next edition.

Standard Practice for Estimating Concrete Strength by the Maturity Method[1]

This standard is issued under the fixed designation C 1074; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

$^{\epsilon 1}$ NOTE—Editorial corrections were made throughout in April 1990.

1. Scope

1.1 This practice provides a procedure for estimating concrete strength by means of the maturity method. Maturity is expressed either in terms of the temperature-time factor or in terms of the equivalent age at a specified temperature.

1.2 This practice requires establishing the strength-maturity relationship of the concrete mixture in the laboratory and recording the temperature history of the concrete for which strength is to be estimated.

1.3 The values stated in SI units are to be regarded as the standard.

2. Referenced Documents

C 39 Test Method for Compressive Strength of Cylindrical Concrete Specimens[2]
C 192 Practice for Making and Curing Concrete Test Specimens in the Laboratory[2]
C 109 Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens)[3]
C 403 Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance[2]
C 511 Specification for Moist Cabinets, Moist Rooms, and Water Storage Tanks Used in the Testing of Hydraulic Cements and Concretes[3]
C 684 Test Method for Making, Accelerated Curing, and Testing of Concrete Compression Test Specimens[2]
C 803 Test Method for Penetration Resistance of Hardened Concrete[2]
C 900 Test Method for Pullout Strength of Hardened Concrete[2]

3. Descriptions of Terms Specific to This Standard

3.1 *datum temperature*—the temperature that is subtracted from the measured concrete temperature for calculating the temperature-time factor according to Eq 1.

3.2 *equivalent age*—the number of days or hours at a specified temperature required to produce a maturity value equal to the value achieved by a curing period at temperatures different from the specified temperature.

3.3 *maturity*—the extent of cement hydration in a concrete mixture. Provided there is sufficient moisture, maturity at a given age is primarily a function of temperature history. Maturity is evaluated from the recorded temperature history of the concrete by computing either the temperature-time factor or the equivalent age at a specified temperature.

3.4 *maturity function*—the mathematical expression for evaluating maturity from the recorded temperature history of the concrete. Refer to Appendix X1 for additional discussion of this subject.

3.5 *maturity method*—a technique for estimating concrete strength that is based on the assumption that samples of a given concrete mixture attain equal strengths if they attain equal maturity values **(1,2)**.[4]

3.6 *temperature-time factor*—the maturity value computed according to Eq 1.

4. Summary of Practice

4.1 A strength-maturity relationship is developed by laboratory tests on the concrete mixture to be used.

4.2 The temperature history of the concrete sample, for which strength is to be estimated, is recorded from the time of concrete placement to the time when the strength estimation is desired.

4.3 The recorded temperature history is used to calculate the maturity of the concrete sample.

4.4 Using the calculated maturity and the strength-maturity relationship, the strength of the concrete sample is estimated.

5. Significance and Use

5.1 This procedure can be used to estimate the in-place strength of concrete to allow the start of critical construction activities such as: (*1*) removal of formwork and reshoring; (*2*) post-tensioning of tendons; and (*3*) termination of cold weather protection.

5.2 This procedure can be used to estimate strength of laboratory specimens cured under non-standard temperature conditions.

5.3 The major limitations of the maturity method are: (*1*) the concrete must be maintained in a condition that permits cement hydration; (*2*) the method does not take into account the effects of early-age concrete temperature on the long-term ultimate strength; and (*3*) this method needs to be

[1] This practice is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.02.05 on Nondestructive Testing of Concrete.
Current edition approved Feb. 10, 1987. Published April 1987.
[2] *Annual Book of ASTM Standards*, Vol 04.02.
[3] *Annual Book of ASTM Standards*, Vol 04.01.
[4] The boldface numbers in parentheses refer to the list of references at the end of this method.

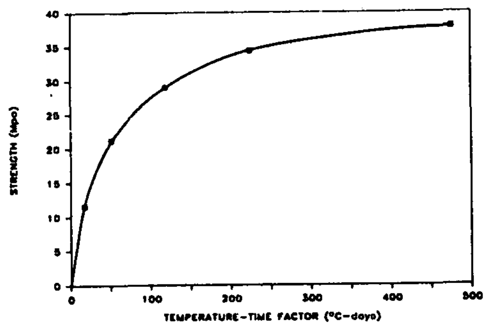

FIG. 1 Example of a Relationship Between Compressive Strength and Temperature-Time Factor

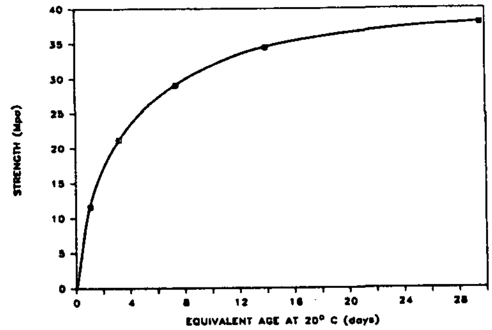

FIG. 2 Example of a Relationship Between Compressive Strength and Equivalent Age at 20°C

supplemented by other indications of the potential strength of the concrete mixture.

5.4 The accuracy of the estimated strength depends on properly determining the maturity function for the particular materials used.

6. Maturity Functions

6.1 There are two alternative functions for computing the maturity value from the measured temperature history of the concrete.

6.2 One maturity function is used to compute the temperature-time factor as follows:

$$M(t) = = \Sigma(T_a - T_o)\,\Delta t \qquad (1)$$

where:

$M(t)$ = the temperature-time factor at age t, degree-days or degree-hours,
Δt = a time interval, days or hours,
T_a = average concrete temperature during time interval, Δt, °C, and
T_o = datum temperature, °C.

6.3 The other maturity function is used to compute equivalent age at a specified temperature as follows (3):

$$t_e = \Sigma e^{-[Q((1/T_a) - (1/T_s))]}\Delta t \qquad (2)$$

where:

t_e = equivalent age at a specified temperature T_s, days or h,
Q = activation energy divided by the gas constant, °K,
T_a = average temperature of concrete during time interval Δt, °K,
T_s = specified temperature, °K, and
Δt = time interval, days or h.

6.4 Suggested approximate values of the datum temperature T_o and the activation energy divided by the gas constant, Q, are given in Appendix X1. Where maximum accuracy of strength prediction is desired, the appropriate values of T_o or Q can be determined according to the procedures given in Annex A1.

7. Apparatus

7.1 A device is required to monitor and record the concrete temperature as a function of time. Acceptable devices include thermocouples or thermistors connected to strip-chart recorders or digital data loggers. The recording time interval shall be ½ h or less for the first 48 h and 1 h or less thereafter. The temperature recording device shall be accurate to within ±1°C.

7.2 Alternative devices include commercial maturity instruments, that automatically compute and display either temperature-time factor or equivalent age.

NOTE 1—Commercial maturity instruments use specific values of datum temperature or activation energy in evaluating maturity; thus the displayed maturity value may not be indicative of the true value for the concrete mixture being used. Refer to Appendix XI for information on correcting the displayed values.

8. Procedure to Develop Strength-Maturity Relationship

8.1 Prepare cylindrical specimens according to Practice C 192 using the mixture proportions and constituents of the concrete whose strength-maturity relationship is to be developed.

8.2 Embed temperature sensors at the centers of at least two specimens. Connect the sensors to maturity instruments or to temperature-recording devices such as data-loggers or strip-chart recorders.

8.3 Moist cure the specimens in a water bath or in a moist room meeting the requirements of Specification C 511.

8.4 Perform compression tests at the ages of 1, 3, 7, 14, and 28 days in accordance with Test Method C 39. Test at least three specimens at each age.

8.5 At each test age, record the average maturity value for the instrumented specimens.

8.5.1 If maturity instruments are used, record the average of the displayed values.

8.5.2 If temperature recorders are used, evaluate the maturity according to Eq 1 or Eq 2. Use a time interval (Δt) of ½ h or less for the first 48 h of the temperature record. Larger time intervals may be used for the relatively constant portion of the subsequent temperature record.

NOTE 2—Appendix X2 gives an example of how to evaluate the temperature-time factor or equivalent age from the recorded temperature history of the concrete.

8.6 On graph paper, plot the average compressive strength as a function of the average maturity value. Draw a best-fit curve through the data. The resulting curve is the strength-maturity relationship to be used for estimating the strength of the concrete mixture cured under other temperature conditions. Fig. 1 is an example of a relationship between compressive strength and temperature-time factor, and Fig. 2 is an example of a relationship between compressive strength and equivalent age at 20°C.

NOTE 3—If regression analysis of the data is desired, see Refs (4 and 5) for possible regression models to use for the strength-maturity relationship.

9. Procedure to Estimate In-Place Strength

9.1 As-soon-as is practicable after concrete placement, embed temperature sensors into the fresh concrete. When using this practice to allow critical construction operations to begin, install sensors at locations in the structure that are critical in terms of exposure conditions and structural requirements.

NOTE 4—In building construction, exposed portions of slabs and the slab-column connections are typically critical locations. The advice of the engineer should be sought for the critical locations in the particular structure under construction.

9.2 Connect the sensors to maturity instruments or temperature-recording devices and activate the recording devices as soon as is practicable.

9.3 When it is desirable to estimate the strength at the location of the sensors, read the maturity value from the maturity instrument or evaluate the maturity from the temperature record.

9.4 Using the strength-maturity relationship developed in Section 8, read off the value of compressive strength corresponding to the measured maturity.

9.5 Prior to performing critical operations, such as formwork removal or post-tensioning, supplement determination of the concrete maturity with other tests to ensure that the concrete in the structure has a potential strength that is similar to that of the concrete used to develop the strength-maturity relationship. Appropriate techniques include:

9.5.1 In-place tests that give indications of strength, such as Test Method C 803 or Test Method C 900,

9.5.2 Early-age compressive strength tests of control specimens molded from samples of the concrete as-delivered, or

9.5.3 Compressive strength tests on specimens molded from samples of the concrete as-delivered and subjected to accelerated curing in accordance with Test Method C 684.

ANNEX

(Mandatory Information)

A1. DETERMINATION OF DATUM TEMPERATURE OR ACTIVATION ENERGY

A1.1 Procedure

A1.1.1 The testing required to experimentally determine datum temperature can be performed with mortar specimens, and the results are applicable to concrete made with the same mortar (6). The activation energy is most accurately determined from calorimetric measurements of heat of hydration of cement paste under different curing temperatures. However, it has been reported that the activation energy can also be determined from strength tests of mortar specimens (7), and this is the approach adopted here.

A1.1.2 Proportion a mortar mixture similar to the mortar in the concrete that is to be used. The mortar shall include the appropriate quantities of admixtures that will be used in the concrete.

A1.1.3 Prepare three mortar specimens using the containers specified in Test Method C 403. Carefully submerge each specimen into temperature-controlled water baths. Two baths shall be at the maximum and minimum concrete temperatures expected for the in-place concrete during the time the strength predictions will be made. The third bath temperature shall be midway between the two extremes.

A1.1.4 Using Test Method C 403, determine the time of final setting for each temperature. The specimens are removed from the water baths and the excess water is removed prior to making penetration measurements.

A1.1.5 Prepare three sets of 50-mm mortar cubes, with 18 cubes per set. Mold the cubes in accordance with Test Method C 109 and carefully submerge each set into the temperature-controlled baths used in A1.1.3. For each set, remove the molds and return the specimens to their respective baths 1 h before the first series of compression tests.

A1.1.6 For each set of cubes, determine the compressive strength of three cubes in accordance with Test Method C 109 at an age that is approximately twice the age to reach final setting. Perform subsequent tests on three cubes from each set at ages that are approximately twice the age of the previous tests. For example, if the time of final setting were 12 h, then compressive tests would be performed at 1, 2, 4, 8, 16 and 32 days.

A1.1.7 For each curing temperature, plot the reciprocal of the average cube strength along the y-axis and the reciprocal of the age beyond the time of final setting along the x-axis. An example of such a plot is shown in Fig. A1.1.

A1.1.8 Determine the slope and intercept of the best-fitting straight line through the data for each curing temperature.

A1.1.9 For each straight line, divide the value of the intercept by the value of the slope. These quotients, or K-values, are used to calculate the datum temperature or activation energy.

A1.2 Determination of Datum Temperature

A1.2.1 Plot the quotients (K-values) from A1.1.9 as a function of the waterbath temperatures (Fig. A1.2). Determine the best-fitting straight line through the three points and determine the intercept of the line with the temperature axis. This intercept is the datum temperature, T_o, that is to be used in computing the temperature-time factor according to Eq 1.

A1.3 Determination of Activation Energy

A1.3.1 Calculate the natural logarithm of the quotients (K-values) in A1.1.9, and determine the absolute temperatures (in Kelvin) of the water baths.

A1.3.2 Plot the natural logarithm of the quotients (K-values) as a function of the reciprocal absolute temperature (Fig. A1.3). Determine the best-fitting straight line through the three points. The slope of the line is the value of the activation energy divided by the gas constant, Q, that is to be used in computing equivalent age according to Eq 2.

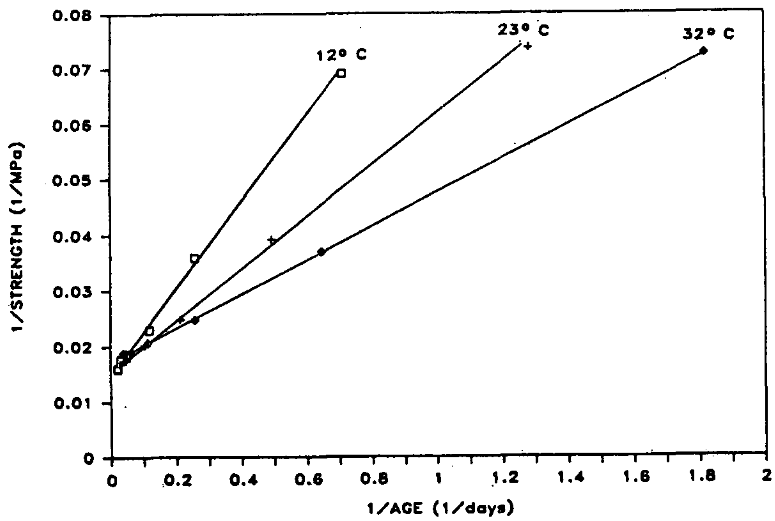

FIG. A1.1 Reciprocal of Strength Versus Reciprocal of Age Beyond Time of Final Setting

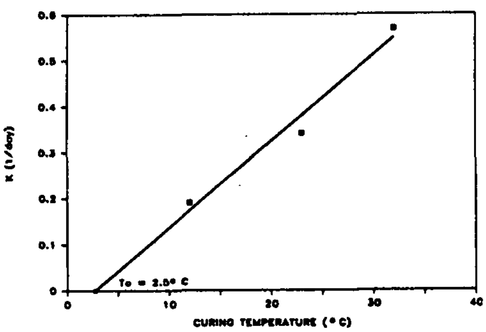

FIG. A1.2 Example of Plot of *K*-Values Versus Curing Temperature for Determining the Datum Temperature

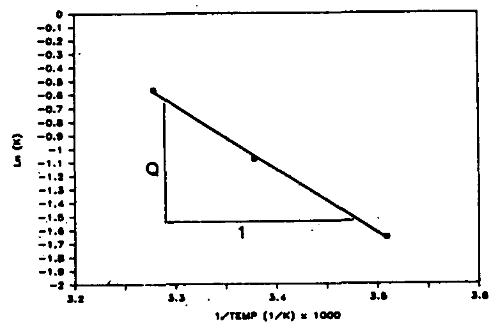

FIG. A1.3 Example of Plot of the Natural Logarithm of *K*-Values Versus the Inverse Absolute Temperature for Determining the Value of *Q* used in Calculating Equivalent Age

APPENDIXES

(Nonmandatory Information)

X1. MATURITY FUNCTIONS

X1.1 General

X1.1.1 A maturity function is a mathematical expression to account for the combined effects of time and temperature on the hydration of cement. The strength of concrete is, in turn, directly related to the extent of hydration. The key feature of a maturity function is the representation of how temperature affects the rate of hydration. There are two widely-used approaches; one assumes that the hydration rate is a linear function of temperature, and the other assumes that the hydration rate obeys the exponential Arrhenius equation. Further information on how these two approaches are related may be found in Ref **(6)**.

X1.2 Temperature-Time Factor

X1.2.1 The assumption that hydration rate is a linear function of temperature leads to the maturity function given in Eq 1, that is used to compute the temperature-time factor. To compute the temperature-time factor, it is necessary to know the appropriate value of the datum temperature for the specific materials and conditions. The datum temperature may depend on the type of cement, on the type and the dosage of admixtures or other additives that affect hydration rate, and on the temperature range that the concrete will experience while hardening **(6)**. For Type I cement without admixtures and a curing temperature range from 0 to 40°C, the recommended datum temperature is 0°C **(6)**. For other conditions and when maximum accuracy of strength prediction is desired, the appropriate datum temperature can be determined experimentally according to the procedures in Annex A1.

X1.2.2 Current models of maturity instruments that compute temperature-time factors may not employ the appropriate datum temperature, and therefore may not indicate the true factor. The value of the temperature-time factor displayed by the instrument can be corrected for the datum temperature as follows:

$$M_c = M_d - (T_o - T_d)\,t \tag{X1.1}$$

where:

M_c = the corrected temperature-time factor, degree-days or degree-hours,
M_d = the temperature-time factor displayed by the instrument, degree-days or degree-hours,
T_o = the appropriate datum temperature for the concrete, °C,
T_d = the datum temperature incorporated into the instrument, °C, and
t = the elapsed time from when the instrument was turned on to when a reading was taken, days or h.

X1.3 Equivalent Age

X1.3.1 The assumption that hydration rate obeys the Arrhenius equation leads to the maturity function given in Eq 2, that is used to compute equivalent age at a specified temperature. Note that in using Eq 2, the temperature must be in Kelvin (Kelvin = Celsius + 273). To compute equivalent age it is necessary to know the activation energy for the specific materials and conditions. It has been suggested that the activation energy depends on the type of cement and on the type and the dosage of admixtures that affect hydration rate **(6)**. For Type I cement without admixtures or additions, values of activation energy in the range of 40 000 to 45 000 J/mol have been reported. Thus an approximate value of Q, the activation energy divided by the gas constant for use in Eq 2, is 5000°K. (The value of the gas constant is 8.31 J/(K-mol)). For other conditions and when maximum accuracy of strength prediction is desired, the appropriate value for Q can be determined experimentally according to the procedures in Annex A1.

X1.3.2 The calculation of equivalent age also requires a specified temperature, T_s. Traditionally, a value of 20°C has been used **(3)**, but any other convenient temperature, such as 23°C (73.4°F), is permissible provided that it is reported along with the value of the equivalent age.

X1.3.3 Maturity instruments that compute equivalent age according to Eq 2, are based on specific values of activation energy. The displayed readings cannot be corrected for the appropriate activation energy value of the concrete being used. The user should recognize this limitation when the in-place concrete has an activation energy that is widely different from that incorporated into the instrument.

X2. EXAMPLE MATURITY CALCULATIONS

X2.1 Temperature Record

X2.1.1 Fig. X2.1 shows a hypothetical temperature history for concrete that will be used to illustrate the calculations of temperature-time factor and equivalent age. The temperature values at half-hour intervals are tabulated in column 2 of Table X2.1.

X2.2 Calculation of Temperature-time Factor

X2.2.1 The value of the datum temperature, T_o, is required to compute the temperature-time factor according to Eq 1. For this example, a value of 2.5°C is assumed as indicated in Fig. A1.2.

X2.2.2 The average temperature during each half-hour

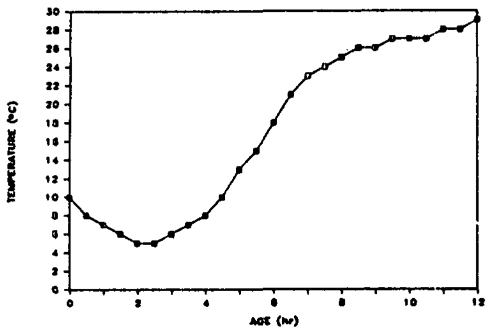

FIG. X2.1 Hypothetical Temperature History used to Illustrate Computations of Temperature-Time Factor and Equivalent Age

interval is computed and the results are given in column 4 of Table X2.1. The datum temperature is subtracted from the average temperatures, and the difference is multiplied by the age interval, which in this example is 0.5 h. The products give the incremental values of the temperature-time factor for the age intervals. The incremental values are shown in column 5 of Table X2.1.

X2.2.3 The summation of the incremental temperature-time factors gives the cumulative temperature-time factor at each age. For example, at an age of 12 h the temperature-time factor is 175° C-hours.

X2.3 Calculation of Equivalent Age

X2.3.1 The value of Q and the value of the specified temperature, T_s, are required to compute the equivalent age according to Eq 2. For this example, the value of Q is assumed to be 4700°K, and the specified temperature is assumed to be 20°C.

X2.3.2 Using the average temperature during each age interval, the values of the exponential function in Eq 2 are calculated. These values are given in column 7 of Table X2.1 under the heading Age Factor. The product of each of the age factors and the age interval (0.5 h) gives the incremental equivalent ages at 20°C; the incremental equivalent ages are shown in column 8 of Table X2.1.

X2.3.3 The summation of the incremental equivalent ages gives the cumulative equivalent age at 20°C (column 9 of Table X2.1). For example, at an age of 12 h the equivalent age at 20°C is 11.3 h.

TABLE X2.1 Example Maturity Calculations

(1) Age, h	(2) Temperature, °C	(3) Age Increment, h	(4) Average Temperature, °C	(5) Temp-Time Factor, Increment °C-h	(6) Temp-Time Factor, Cumulative °C-h	(7) Age Factor	(8) Eq. Age at 20°C, Increment h	(9) Eq. Age at 20°C, Cumulative h
0	10	...	...	...	0	...	...	0.0
0.5	8	0.5	9	3.3	3	0.53	0.27	0.3
1	7	0.5	7.5	2.5	6	0.49	0.24	0.5
1.5	6	0.5	6.5	2.0	8	0.46	0.23	0.7
2	5	0.5	5.5	1.5	9	0.43	0.22	1.0
2.5	5	0.5	5	1.3	11	0.42	0.21	1.2
3	6	0.5	5.5	1.5	12	0.43	0.22	1.4
3.5	7	0.5	6.5	2.0	14	0.46	0.23	1.6
4	8	0.5	7.5	2.5	17	0.49	0.24	1.9
4.5	10	0.5	9	3.3	20	0.53	0.27	2.1
5	13	0.5	11.5	4.5	24	0.62	0.31	2.4
5.5	15	0.5	14	5.8	30	0.72	0.36	2.8
6	18	0.5	16.5	7.0	37	0.82	0.41	3.2
6.5	21	0.5	19.5	8.5	46	0.97	0.49	3.7
7	23	0.5	22	9.8	55	1.11	0.56	4.3
7.5	24	0.5	23.5	10.5	66	1.21	0.60	4.9
8	25	0.5	24.5	11.0	77	1.27	0.64	5.5
8.5	26	0.5	25.5	11.5	88	1.34	0.67	6.2
9	26	0.5	26	11.8	100	1.38	0.69	6.9
9.5	27	0.5	26.5	12.0	112	1.42	0.71	7.6
10.5	27	0.5	27	12.3	124	1.45	0.73	8.3
10.5	27	0.5	27	12.3	137	1.45	0.73	9.0
11	28	0.5	27.5	12.5	149	1.49	0.75	9.8
11.5	28	0.5	28	12.8	162	1.53	0.77	10.5
12	29	0.5	28.5	13.0	175	1.57	0.79	11.3

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Appendix B

ASTM C 900

Designation: C 900 – 87$^{\epsilon 1}$

AMERICAN SOCIETY FOR TESTING AND MATERIALS
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Standard Test Method for Pullout Strength of Hardened Concrete[1]

This standard is issued under the fixed designation C 900; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ϵ) indicates an editorial change since the last revision or reapproval.

$^{\epsilon 1}$NOTE—Editorial corrections were made throughout in June 1988.

1. Scope

1.1 This test method covers determination of the pullout strength of hardened concrete by measuring the force required to pull an embedded metal insert and the attached concrete fragment from a concrete test specimen or structure.

1.2 The values stated in SI units are to be regarded as the standard. The values given in parentheses are for information purposes only.

1.3 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

C 39 Test Method for Compressive Strength of Cylindrical Concrete Specimens[2]

E 4 Practices for Load Verification of Testing Machines[2]

3. Summary of Method

3.1 A metal insert is embedded in fresh concrete. After the concrete has hardened, the insert is pulled by means of a jack reacting against a bearing ring. The pullout strength is determined by measuring the maximum force required to pull the insert from the concrete mass.

4. Significance and Use

4.1 For a given concrete and a given test apparatus, the pullout strength can be related to other strength test results. Such strength relationships depend on the configuration of the embedded insert, bearing ring dimensions, depth of embedment (see 5.1.2), and level of strength development in that concrete. Prior to use, these relationships must be established for each system and each new combination of concreting materials. Such relationships tend to be less variable where both pullout test specimens and other strength test specimens are of consistent size and cured under similar conditions.

NOTE 1—Published reports (**1–13**)[3] by different researchers present their experiences in the use of pullout test equipment.

4.2 Pullout tests are used to determine whether the in-place strength of concrete has reached a specified level so that, for example:

(*a*) post-tensioning may proceed;

(*b*) forms and shores may be removed; or

(*c*) winter protection and curing may be terminated.

4.3 When planning pullout tests and analyzing test results, consideration should be given to the normally expected decrease of concrete strength with increasing height within a given concrete placement in a structural element.

5. Apparatus

5.1 The apparatus requires three basic sub-systems: a pullout insert, a loading system, and a load-measuring system.

NOTE 2—A center-pull hydraulic jack with a suitable pressure gage and bearing ring have been used satisfactorily.

5.1.1 The insert shall be made of metal that does not react with cement. The insert shall consist of a cylindrical head to be embedded in fresh concrete. A shaft to fix embedment depth shall be firmly attached to the head. The insert shaft may be removeable and threaded to the insert head or it may be an integral part of the insert. Metal components of the insert and attachment hardware shall be of similar material to prevent galvanic corrosion.

5.1.2 The loading system shall consist of a bearing ring to be placed against the hardened concrete surface concentrically around the insert shaft (see Fig. 1), and a loading apparatus with the necessary load-measuring devices that can be readily attached to the pullout shaft.

5.1.3 The test apparatus shall include centering features that ensure that the bearing ring is concentric with the insert shaft, and that the applied load is axial to the pullout shaft, perpendicular to the bearing ring, and uniform on the bearing ring.

5.2 Equipment dimensions shall be determined as follows (see Fig. 1):

5.2.1 The diameter of the head of the insert (d_2) shall be determined by the specifier. The thickness of the insert head and the yield strength of the metal shall be sufficient to avoid yielding of the insert during test. The sides of the insert head shall be smooth.

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[1] This method is under the jurisdiction of ASTM Committee C-9 on Concrete and Concrete Aggregates and is the direct responsibility of Subcommittee C09.02.05 on Nondestructive Testing of Concrete.

Current edition approved April 24, 1987. Published June 1987. Originally published as C 900 – 78 T. Last previous edition C 900 – 82.

[2] *Annual Book of ASTM Standards*, Vol 04.02.

[3] The **boldface** numbers refer to the list of references at the end of this test method.

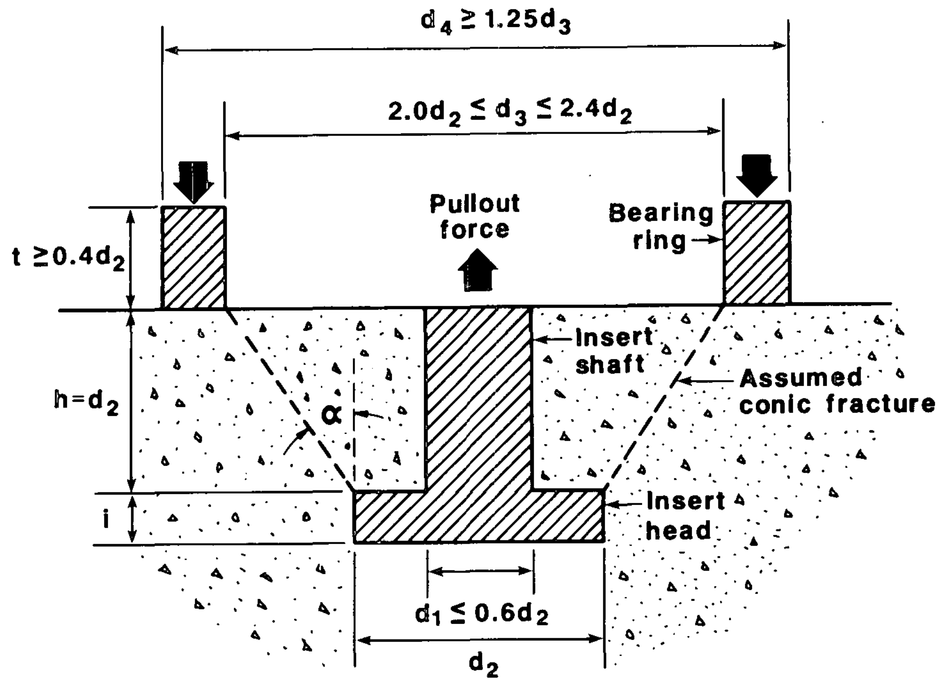

FIG. 1 Schematic Cross Section of Pullout Test

NOTE 3—Typical sizes are 25 and 30 mm (1 and 1.2 in.) in diameter, but smaller and larger sizes have been used.

NOTE 4—The pullout insert may be coated with a release agent to minimize bonding with the concrete, it may be tapered to minimize side friction during testing. The insert head shall be provided with the means, such as a notch, to prevent rotation in the concrete if the insert shaft has to be removed prior to performing the test. As a further precaution against rotation of the insert head, all threaded hardware shall be checked prior to installation to ensure that it is free-turning and can be easily removed.

5.2.2 The length of the pullout insert shaft shall be such that the distance from the insert head to the concrete surface (h) equals the diameter of the insert head (d_2). The diameter of the insert shaft at the head (d_1) shall be no more than 0.60 times the head diameter.

5.2.3 The bearing ring shall have an inside diameter (d_3) of 2.0 to 2.4 times the insert head diameter, and shall have an outside diameter (d_4) of at least 1.25 times the inside diameter. The thickness of the ring (t) shall be at least 0.4 times the pullout insert head diameter.

5.2.4 Tolerances for dimensions of the pullout test inserts shall be ±2 % within a given system.

NOTE 5—The limits for dimensions and configurations for pullout test inserts and apparatus are intended to accommodate various systems.

5.2.5 The loading apparatus shall have sufficient capacity to provide the loading rate prescribed in 7.4 and exceed the maximum load expected.

NOTE 6—Hydraulic pumps that provide a uniform loading rate may give more uniform test results than pumps that apply the load intermittently.

5.2.6 Gages shall have a least division not larger than 5 % of the minimum value in the intended range of use.

NOTE 7—For the most accurate results, gages should have a maximum value indicator that preserves the value of the ultimate load when ultimate failure and subsequent stress release occur.

5.2.7 Pullout apparatus shall be calibrated at least once a year and after all repairs or adjustments. Calibration shall be by one of the methods in Practices E 4, or with a compression testing machine conforming to the requirements of Test Method C 39 using the calibration procedures described in the Annex to this test method.

6. Sampling

6.1 Pullout test locations shall be separated so that the clear spacing between inserts is at least ten times the pullout insert head diameter. Clear spacing between the inserts and the edges of the concrete shall be at least four times the head diameter. Inserts shall be placed so that reinforcement is outside the expected conical failure surface by more than one bar diameter, or the maximum size of aggregate, whichever is greater.

6.2 When pullout test results are used to assess the in-place strength in order to allow the start of critical operations, such as formwork removal or application of post tensioning, at least five individual pullout tests shall be performed for a given placement for every 115 m^3 (150 yd^3), or a fraction thereof, or for every 470 m^2 (5000 ft^2), or a fraction thereof, of the surface area of one face in the case of slabs or walls.

NOTE 8—Inserts shall be located in those portions of the structure that are critical in terms of exposure conditions and structural requirements.

7. Procedure

7.1 Attach the pullout inserts to the forms using bolts or by other acceptable methods that firmly secure the insert in its proper location prior to concrete placement. All inserts for the same tests shall be embedded to the same depth and each shaft shall be perpendicular to the formed surface.

NOTE 9—Inserts may be manually placed into unformed horizontal concrete surfaces. The inserts shall be embedded into the fresh concrete by means that ensure a uniform embedment depth and a plane surface perpendicular to the axis of the insert shaft. Installation of inserts shall be performed or supervised by experienced personnel. Experience indicates that pullout strengths are of lower value and more variable for manually-placed surface inserts than for inserts attached to the formwork.

7.2 When the concrete is to be tested, remove all hardware used for securing the pullout inserts in position. Before mounting the loading system, remove any debris or surface abnormalities to ensure a smooth bearing surface that is perpendicular to the axis of the insert.

7.3 Use a bearing ring for all surface pullout-test configurations. Place the bearing ring around the pullout insert shaft, connect the pullout shaft to the hydraulic ram, and tighten the pullout assembly snugly against the bearing surface, checking to see that the bearing ring is centered around the shaft and flush against the concrete.

7.4 If the insert is to be tested to rupture of the concrete, load at a uniform rate, that will cause pullout rupture to occur in 120 ± 30 s. Record the maximum gage reading to the nearest half of the least division on the dial. If the insert is to be tested only to a specified level for acceptance, load at a uniform rate that will reach the specified level in 120 ± 30 s.

8. Calculation

8.1 Convert test readings to force on the basis of calibration data.

8.2 When a stress calculation is desired, compute a nominal normal stress on the assumed conical fracture surface by dividing the pullout force by the area of the frustum and multiplying by the sine of one-half the apex angle. The following equations may be used:

$$f_n = (P/A) \text{ sine } a \tag{1}$$

$$\text{sine } a = (d_3 - d_2)/2S \tag{2}$$

$$A = \pi S\,(d_3 + d_2)/2 \tag{3}$$

$$S = \sqrt{h^2 + ((d_3 - d_2)/2)^2} \tag{4}$$

where:

f_n = nominal normal stress, MPa (psi),
P = pullout force, N (lbf),
a = ½ the frustum apex angle or: $\tan^{-1}\,(d_3 - d_2)/2h$
A = fracture surface area, mm^2 (in.^{2}),
d_2 = diameter of pullout insert head, mm (in.),
d_3 = inside diameer of bearing ring or large base diameter of assumed conic frustum, mm (in.),
h = height of conic frustum, from insert head to large-base surface, mm (in.), and
S = slant height of the frustum, mm (in.)

9. Report

9.1 The report shall include the following:

9.1.1 Dimension of the pullout insert and bearing ring (sketch or define dimensions),

9.1.2 Identification by which the specific location of the pullout test can later be determined,

9.1.3 Date and time when the pullout test was performed.

9.1.4 Maximum load, N (lbf),

9.1.5 Description of any surface abnormalities beneath the reaction ring at the test location,

9.1.6 Abnormalities in the ruptured specimen and in the loading cycle,

9.1.7 Concrete curing methods used and moisture condition of the concrete at time of test, and

9.1.8 Other information regarding unusual job conditions that may affect the pullout strength.

10. Precision and Bias

10.1 *Precision*—The precision of this test method has not been evaluated. A precision statement will be included after sufficient available data are analyzed.

10.2 *Bias*—The bias of this test method cannot be evaluated since pullout strength can only be determined in terms of this test method.

ANNEX

(Mandatory Information)

A1. CALIBRATION OF PULLOUT-HYDRAULIC LOADING SYSTEM

A1.1 Calibrate the pullout-hydraulic loading system (pump, gage, and hydraulic jack) with a testing machine meeting the applicable requirements of Method C 39.

A1.2 Place the hydraulic jack between the two testing machine bearing blocks. Position the jack and bearing blocks to ensure concentric and axial loading, and extend the piston to the level anticipated for actual pullout testing. Carefully position the testing machine head against the pullout jack.

NOTE—Protection of the bearing blocks will be required to prevent damage to the test machine. Cold-rolled steel plate at least 13 mm (½ in.) thick is recommended.

A1.3 Using the hydraulic pump, apply loads progressively over the range of anticipated use, and record the hydraulic pressure gage reading and the testing machine load at each calibration load level. With available center-pull jacks, friction in the system produces different calibration curves on an increasing series of loads than on decreasing loads and therefore only increasing loads should be used. In general, readings should be taken at approximately 20 load levels distributed within the range of loads anticipated in use.

A1.4 Using readings obtained during calibration loading, calculate a linear regression equation using the least-squares curve-fitting method. Pullout testing may occur within a narrow range of the capacity of the pullout jack. If the test results fall within a narrow range, calculate the regression equation based on calibration readings in that range, excluding those outside the test range.

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Appendix C

ACI 228.1R

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In-Place Methods for Determination of Strength of Concrete

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A. Zoob

The state of the art in the use of methods for determining the in-place compressive strength of concrete is reported. The methods covered include the rebound hammer, probe penetration, pullout, ultrasonic pulse velocity, maturity, and cast-in-place cylinder. The underlying principles and inherent limitations of each method are discussed. Repeatability of test results is reviewed, and recommendations are given for developing the correlation relationship for each test method. Recommendations are given for the number of tests, and statistical techniques for interpretation of test results are described.

Keywords: coefficient of variation; **compressive strength; concretes;** construction; **nondestructive tests; reviews;** safety; sampling; statistical analysis.

CONTENTS

CHAPTER 1—INTRODUCTION

1.1—Scope

In-place tests, which have also been called "nondestructive" tests, are used to obtain information about the properties of concrete as it exists in a structure. In this report, the property of interest is the compressive strength of concrete. Determination of concrete strength is usually performed for two reasons: 1) the evaluation of an existing structure, or 2) monitoring strength development during new construction. This report places emphasis on the latter application since it represents the major use of in-place methods in North American practice. Over the years, dozens of techniques have been proposed for estimating the in-place strength of concrete. No attempt is made to review all these methods; only those methods that have been standardized by ASTM are discussed.

*Task Force Member.
'Task Force Chairman.

1.2—Need for in-place tests during construction

For over 70 years in North American practice, the most widely used test for concrete has been the compression test of the standard cylinder. The test procedure is relatively easy to perform in terms of sampling, preparation of specimens, and the determination of strength. When properly performed, this test has low in-test variation and low interlaboratory variation, and therefore lends itself readily to use as a standard. The strength value obtained is used in design calculations suitably modified by constants that relate design stresses to the compressive strength value. This strength value is therefore an essential parameter in all design codes.

As carried out in accordance with standard procedures, however, the test only represents the potential strength of the concrete as delivered to a site. The test is used mainly as a basis for quality control. It is not intended for determining the strength of the concrete in-place since it makes no allowance for the effects of placing, compaction, or curing. It is unusual for the concrete in a structure to have the same maturity as a standard-cured cylinder. In addition, since standard-cured cylinders are normally tested at an age of 28 days, they cannot be used to determine whether adequate strength exists for safe removal of formwork or the application of post-tensioning. Although portions of a structure, such as columns, may develop maturity equal to that of standard 28-day cylinders, most flexural and prestressed members do not develop equivalent maturity before they are required to accept large percentages of their design loads. For these reasons in-place tests are needed to determine the strength of the concrete in the structure in the locations and at the times required for these various construction operations.

Traditionally, some measure of the strength of the concrete in the structure has been obtained by using field-cured cylinders. These are supposedly cured on or in the structure under the same conditions as the concrete in the structure. The results of tests on field-cured cylinders are often significantly different from the strength of the concrete in place because it is difficult and often impossible to assure identical bleeding, compaction, and curing conditions in the cylinders and in the structure. The test also lends itself to abuse. Improper handling or inappropriate storage of these cylinders may result in misleading data for critical operations.

To meet rapid construction schedules, form removal, application of post-tensioning, termination of curing, and the removal of reshores must be carried out as early as is possible and safe. The determination of in-place strength to enable these operations to proceed safely at the earliest possible time requires the use of reliable in-place tests. Conversely, it is clear that some major recent construction failures would not have occurred had such measures been adopted (Lew 1980; Carino et al. 1983). The use of in-place tests not only increases safety but can result in substantial savings in construction costs by permitting accelerated construction schedules.

1.3—Impact of ACI 318-83

Previous versions of ACI 318 have required testing of field-cured cylinders to demonstrate the adequacy of concrete strength prior to removal of formwork or reshoring. However, Section 6.2.1.1 of ACI 318-83 allows the use of alternative procedures to testing field-cured cylinders. The alternative procedures must be approved by the building official prior to use.

Most of the design provisions in the ACI Building Code are based on the compressive strength of standard cylinders. Thus, to evaluate structural capacity under construction loading, it is necessary to have a measure of the cylinder strength of the concrete as it exists in the structure. If in-place tests are used, it is necessary to have a correlation relationship between the results of in-place tests and the compressive strength of cylinders. At present, however, there are no standard practices for developing the required relationship. There are also no generally accepted guidelines for interpretation of in-place test results. These deficiencies have been additional impediments to more widespread adoption of in-place tests.

1.4—Objective of report

This report reviews the state of the art of the widely used methods for determining the in-place strength of concrete. Chapter 2 discusses the underlying principles and inherent limitations of these methods. Statistical analysis of in-place test data is discussed in Chapter 3.

The overall objective of this report is to provide the potential user with a guide to assist in implementing and in interpreting the results of in-place testing.

CHAPTER 2—REVIEW OF METHODS

2.1—Introduction

The objective of an in-place test is to obtain an estimate of the properties of concrete in the structure without having to drill and test core samples. Very often the desired property is the cylinder compressive strength. To make a strength estimation it is necessary to have a known relationship between the result of the in-place test and the strength of the concrete. Usually such a relationship is empirically established in the laboratory. Fig. 2.1 is an illustration of such a relation-

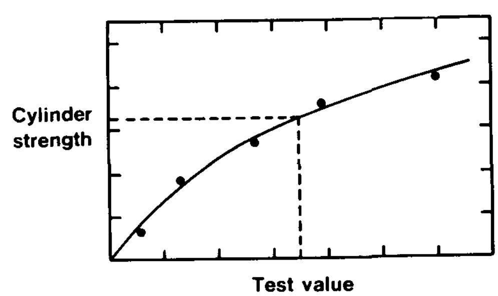

Fig. 2.1—Example of correlation relationship for prediction of in-place strength

ship, in which the cylinder compressive strength is plotted as a function of an in-place test result. This relationship would be used to estimate the strength of concrete in a structure based on the value of the in-place test result obtained from testing the structure. The accuracy of the strength prediction depends directly on the degree of correlation between the strength of concrete and the quantity measured by the in-place test. Thus, the user of in-place tests should have an understanding of what quantity is measured by the test and how this quantity is related to the strength of concrete. The purpose of this chapter is to explain the underlying principles of the widely used in-place test methods, and to point out those factors other than concrete strength that can influence the test results. Additional background information on these methods is available in the references by Malhotra (1976) and Bungey (1982).

The methods to be discussed include the following:

- rebound hammer
- probe penetration
- pullout
- ultrasonic pulse velocity
- maturity
- cast-in-place cylinder

2.2—Rebound hammer

The operation of the rebound hammer (also called the Schmidt Hammer or Swiss Hammer) is illustrated schematically in Fig. 2.2. The hammer consists of the following main components: 1) outer body, 2) plunger, 3) hammer, and 4) spring. To perform the test, the plunger is extended from the body of the instrument and brought into contact with the concrete surface. When the plunger is extended, a latching mechanism engages the hammer to the upper end of the plunger. The body of the instrument is then pushed toward the concrete member. This action causes an extension of the spring connecting the hammer to the body [Fig. 2.2(b)]. When the body is pushed to its limit, the latch releases and the spring pulls the hammer toward the concrete member [Fig. 2.2(c)]. The hammer impacts the shoulder area of the plunger and rebounds [Fig. 2.2(d)]. The rebounding hammer moves the slide indicator, which records the rebound distance. The rebound distance is measured on a scale numbered from 10 to 100 and is recorded as the "rebound number" indicated on the scale.

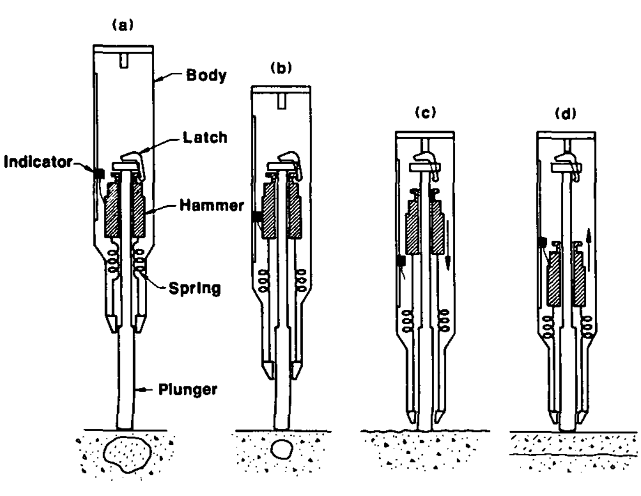

Fig. 2.2—Schematic illustration of the operation of the rebound hammer

The key to understanding the inherent limitations of this test for strength prediction is understanding the factors influencing the rebound distance. From a fundamental point of view, the test is a complex problem of impact loading and stress-wave propagation. Basically, the rebound distance depends on the value of kinetic energy in the hammer prior to impact with the shoulder of the plunger and how much of that energy is absorbed during the impact. Part of the energy is absorbed as mechanical friction in the instrument, and part of the energy is absorbed in the interaction of the plunger with the concrete. It is the latter factor that enables one to use the rebound number as an indicator of the concrete properties. The energy absorbed by the concrete depends on the stress-strain relationship of the concrete. Therefore, absorbed energy is related to the strength and the stiffness of the concrete. A low-strength, low-stiffness concrete will absorb more energy than a high-strength, high-stiffness concrete. Thus the low-strength concrete will result in a lower rebound number. Since it is possible for two concrete mixtures to have the same strength but different stiffnesses, there could be different rebound numbers even though the strengths are equal. Conversely, it is possible for two concretes with different strengths to result in the same rebound numbers if the stiffness of the low-strength concrete is greater than the stiffness of the high-strength concrete. The aggregate type has an effect on the stiffness of the concrete, which is why it is necessary to develop the correlation relationship on concrete made with the same materials that will be used for the concrete in the structure.

In rebound-hammer testing, only the concrete in the immediate vicinity of the plunger influences the rebound value. Hence, the test is sensitive to the local conditions where the test is performed. If the plunger is located over a hard aggregate particle, an unusually high rebound number will result. On the other hand, if the plunger is located over a large air void or over a soft aggregate particle, a lower rebound number will occur. To account for these possibilities, ASTM C 805 requires that 10 rebound numbers be taken for a test. If one of the readings differs by more than seven units from the average, that reading should be discarded and a new average should be computed based on the remaining readings. If more than two readings differ from the average by seven units, the entire set of readings is to be discarded.

Because the rebound hammer test probes only the near-surface layer of concrete, the rebound number may not be representative of the interior concrete. The presence of a layer of carbonation can result in higher readings than are indicative of the interior concrete. Likewise, a dry surface will result in higher rebound numbers than for the moist-interior concrete. Slightly absorptive oiled plywood will absorb moisture from the concrete and produce a harder surface layer than concrete cast against steel forms. Similarly, curing conditions have a greater effect on the strength of the surface concrete than several inches from the surface. The surface texture may also influence the rebound number. When the test is performed on rough textured concrete, local crushing occurs under the plunger and the indicated concrete strength will be lower than the true value. Rough surfaces should be ground before testing. If the formed surfaces are reasonably smooth, grinding is unnecessary. A hard surface, such as produced by trowel finishing, can result in higher rebound numbers. Finally, the rebound distance is affected by the orientation of the instrument, and the correlation relationship must be developed for the same instrument orientation as will be used for in-place testing.

In summary, while the rebound number test is a very simple test to perform, there are many factors other than concrete strength that will influence the test result. The user needs to be aware of these effects when evaluating the test results.

2.3—Probe penetration

The probe-penetration technique involves the use of a specially designed gun to drive a hardened steel rod (or probe) into the concrete. (The common commercial system for this type of test is known as the Windsor Probe.) The amount of penetration of the probe is used as an indicator of the concrete strength. From a fundamental point of view, this method is similar to the rebound hammer test, except that the probe impacts the concrete with much higher energy than the plunger of the rebound hammer. A theoretical analysis of this test is even more complicated than the rebound test, but again the essence of the test involves the initial kinetic energy of the probe and energy absorption by the concrete. The probe penetrates into the concrete to the distance required for the absorption of its initial kinetic energy. The initial kinetic energy is governed by the size of the powder charge to fire the probe, the location of the probe in the gun barrel prior to firing, and frictional losses as the probe travels through the barrel. An essential requirement of this test is that the probe have a consistent value of initial kinetic energy. ASTM C 803 requires that the exit velocities of probes should not have a coefficient of variation greater than 3 percent based on 10 tests by approved ballistic methods.

The probe penetrates into the concrete until its initial kinetic energy is completely absorbed by the concrete. Some energy is absorbed by friction between the probe and the concrete, and some is absorbed by crushing and fracturing of the concrete. There are no rigorous studies of the factors affecting the geometry of the fracture zone, but its general shape is probably as illustrated in Fig. 2.3. There is an approximately cone-shaped region in which the concrete is heavily fractured, and this is the zone where most of the probe energy is absorbed.

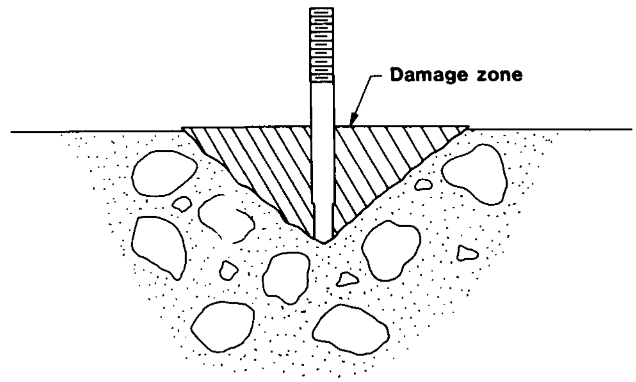

Fig. 2.3—Approximate shape of failure zone in concrete during probe penetration test

The probe tip travels through mortar and aggregate; in general, cracks in the fracture zone will be through the mortar matrix and the coarse-aggregate particles. Hence, the strength properties of both the mortar and the aggregates influence the penetration distance. This contrasts with the behavior of concrete in a compression test, where the mortar strength has a predominant influence on the measured compressive strength. Thus, an important characteristic of the probe penetration test is that the type of coarse aggregate has a strong effect on the correlation relationship between concrete strength and probe penetration. For example, Fig. 2.4 is a schematic illustration of empirical correlation relationships between compressive strength and probe penetration for concretes made with a soft aggregate (such as limestone) and with a hard aggregate (such as chert). For equal compressive strengths, the concrete with the soft aggregate results in greater probe penetration than the concrete with the hard aggregate. More detailed information on the influence of aggregate type on correlation relationships can be found in Malhotra (1976) and Bungey (1982).

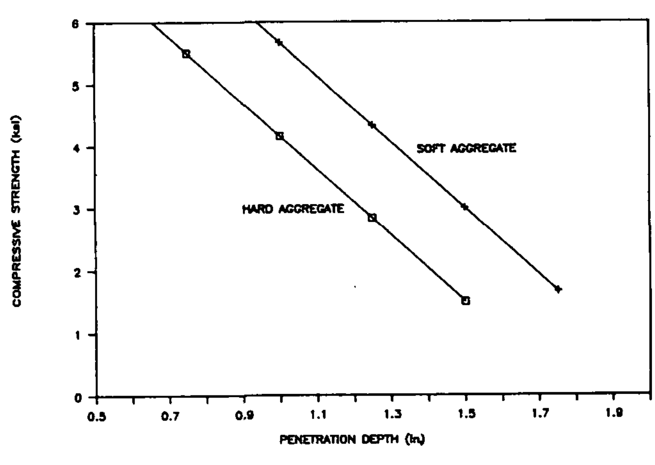

Fig. 2.4—Effect of aggregate type on relationship between concrete strength and depth of probe penetration

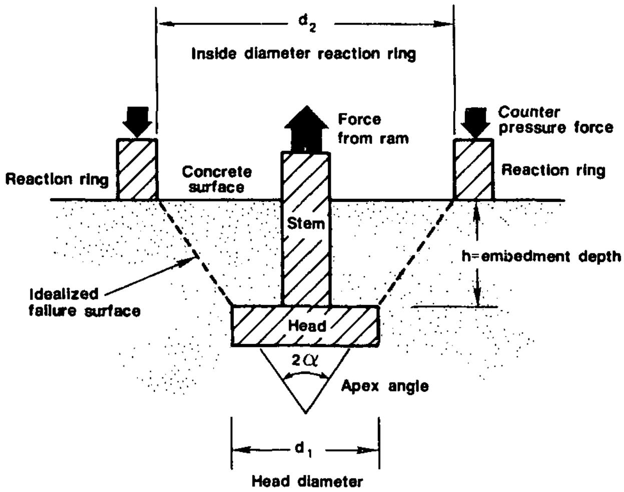

Fig. 2.5—Schematic of pullout test

Because the probe penetrates into the concrete, test results are not affected by local surface conditions such as texture and moisture content. However, a harder surface layer, as would occur in trowel finishing, can result in low penetration values and excessive scatter of data. In addition, the direction of penetration into the concrete is unimportant provided that the probe is driven perpendicular to the surface. In practice it is customary to measure the exposed length of the probes. However, the fundamental relationship is between concrete strength and *depth* of penetration. Therefore, when assessing the variability of test results (see Chapter 3), it is preferable to express the coefficient of variation in terms of penetration depth rather than exposed length.

2.4—Pullout test

The pullout test measures the ultimate load required to pull an embedded metal insert with an enlarged head from a concrete specimen or structure. The pulling load is applied by a tension jack, which reacts against the concrete surface through a reaction ring concentric with the insert (Fig. 2.5). As the insert is pulled out, a roughly cone-shaped fragment of the concrete is also extracted. The diameter d_2 of the reaction ring determines the large diameter of the conic fragment, and the small diameter d_1 is determined by the insert-head diameter. The requirements for the testing configuration are given in ASTM C 900. The embedment depth and head diameter must be equal, but there is no requirement on the magnitude of these dimensions. The inner diameter of the reaction ring can be any size in the range of 2.0 to 2.4 times the insert-head diameter. This means that the apex angle of the conic frustum defined by the insert-head diameter and the inside diameter of the reaction ring can vary between 54 to 70 deg. The same test geometry must be used in developing the correlation relationship as will be used for the in-place testing.

Unlike the rebound hammer and probe penetration tests, the pullout test subjects the concrete to a static loading that is amenable to stress analysis. Using the finite element method, the stresses induced in the concrete have been calculated for the case before any cracking has developed (Stone and Carino 1984) and for the case where the concrete has cracked (Ottosen 1981). There is agreement that the test subjects the concrete to a highly nonuniform three-dimensional state of stress. Fig. 2.6 shows the approximate directions (trajectories) of the principal stresses acting in radial planes (those passing through the center of the insert) prior to any cracking and for apex angles of 54 and 70 deg. Because of symmetry, only one-half of the specimen is shown in the figures. These trajectories would be expected to change after cracking develops. It is seen that prior to cracking there are tensile stresses that are approximately perpendicular to the eventual failure surface, and that compressive stresses are directed from the insert head towards the ring. The magnitude of these principal stresses are nonuniform and are greatest near the top edge of the insert head.

A series of analytical and experimental studies have been carried out to determine the failure mechanism of the pullout test, some of which are critically reviewed

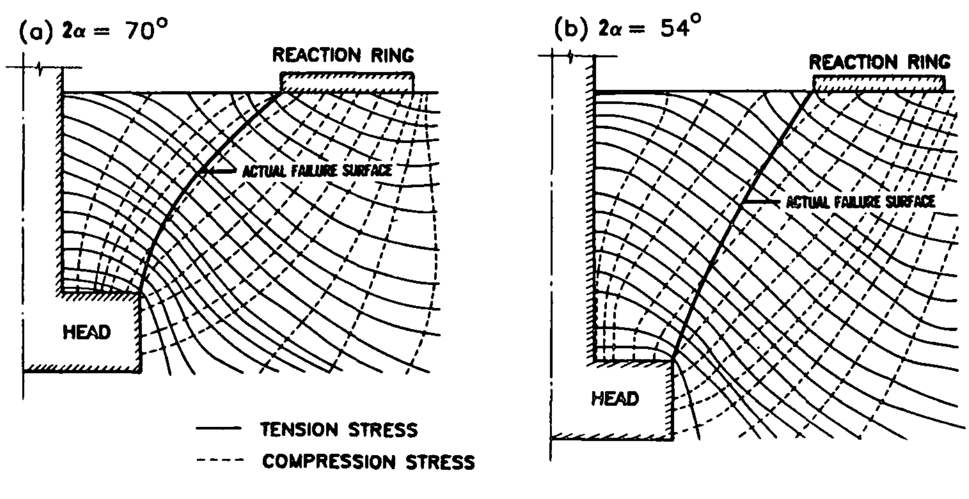

Fig. 2.6—Directions of principal stresses in concrete prior to cracking for pullout test: (a) apex angle = 70 deg; (b) apex angle = 54 deg (Stone and Carino 1984)

by Yener and Chen (1984). While the conclusions have differed, it is generally agreed that circumferential cracking (producing the failure cone) begins in the highly stressed region adjacent to the insert head at a pullout load that is a fraction of the ultimate value. With increasing load, the circumferential cracking propagates toward the reaction ring. However, there is no agreement on the nature of the final failure mechanism governing the magnitude of the ultimate pullout load.

In one analytical study (Ottosen 1981), it was concluded that ultimate failure is due to "crushing" of concrete in a narrow band between the insert head and the reaction ring. Thus, the pullout load is said to depend directly upon the compressive strength of the concrete. In another analytical study (Yener and Vajarasathira 1985), ultimate failure is said to occur by outward crushing of concrete around the perimeter of the failure cone near the reaction ring. Using linear-elastic fracture mechanics and a two-dimensional model, others (Ballarini, Shah, and Keer 1986) have concluded that ultimate load is governed by the fracture toughness of the matrix. In an experimental study (Stone and Carino 1983), it was concluded that prior to ultimate load, circumferential cracking extends from the insert head to the reaction ring; additional load is resisted by aggregate interlock across the circumferential crack. In this case, ultimate failure is said to occur when sufficient aggregate particles have been pulled out of the mortar matrix. According to the aggregate interlock argument, ultimate load is not directly related to the compressive strength. However, there is good correlation between ultimate pullout load and compressive strength of concrete, because both values are influenced by the mortar strength (Stone and Carino 1984). Another finite element study, using nonlinear fracture mechanics and a discrete cracking model, showed excellent agreement between the predicted and observed internal cracking in the pullout test (Hellier et al. 1987). This study also concluded that ultimate failure does not occur because of a compressive failure of the concrete.

Unlike other in-place tests, the standard pullout test requires preplannning the locations of the inserts on the formwork. In addition, the test cannot be performed on structures lacking embedded inserts. However, other types of pullout tests are available, which can be carried out on existing construction. These involve drilling a hole and inserting some type of expanding device that will engage in the concrete and produce a failure in the concrete when the device is extracted. These methods are still in their developmental stages and have not been standardized as ASTM test methods.

A positive feature of the pullout test is that it produces a rather well-defined failure in the concrete and measures a static strength property of the concrete. However, since there is not a consensus on what this strength property is, it is necessary to develop an empirical correlation relationship between the pullout strength and the compressive strength of the concrete. The relationship is applicable to only the particular test configuration and concrete materials employed in the correlation testing.

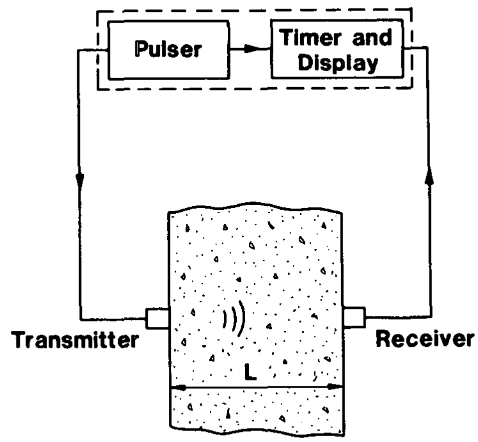

Fig. 2.7—Principle of operation of apparatus for measuring ultrasonic pulse velocity

The pullout strength is governed primarily by that portion of the concrete located adjacent to the conic frustum defined by the insert head and reaction ring. Commercial inserts have embedment depths on the order of 25 to 30 mm. Thus, only a small volume of the concrete is tested, and because of the inherent heterogeneity of concrete, the average within-batch coefficient of variation of these pullout tests has been found to be in the range of 7 to 10 percent, which is about two to three times that of standard cylinder compression tests.

2.5—Ultrasonic pulse velocity

The ultrasonic pulse velocity test, as prescribed in ASTM C 597, determines the propagation velocity of a pulse of vibrational energy through a concrete member. The operational principle of modern testing equipment is illustrated in Fig. 2.7. A pulser sends a short-duration, high-voltage signal to a transducer, causing the transducer to vibrate at its resonant frequency. At the start of the electrical pulse an electronic timer is switched on. The transducer vibrations are transferred to the concrete through a viscous coupling fluid. The vibrational pulse travels through the member and is detected by a receiving transducer coupled to the opposite concrete surface. When the pulse is received, the electronic timer is turned off and the elapsed travel time is displayed. The direct path length between the transducers is divided by the travel time to obtain the pulse velocity through the concrete.

The pulse velocity is proportional to the square root of the elastic modulus and inversely proportional to the square root of the mass density of the concrete. If it is assumed that the elastic modulus of concrete is proportional to the square root of the compressive strength, as suggested by ACI 318, then the pulse velocity is proportional to the square root of the square root of the compressive strength. This means that, for a given concrete mixture, as the compressive strength increases with age there is a proportionately smaller increase in the pulse velocity. For example, reported data (RILEM 1981) indicate that an increase in early-age compressive strength from 500 to 1500 psi (3.4 to 10.3 MPa) may increase the velocity from about 13,100 to about 15,000 ft/s (4000 to 4600 m/s). On the other hand, at later ages a gain in compressive strength from 4000 to 5000 psi (27.6 to 34.5 MPa) may increase the velocity from about 16,700 to only about 17,100 ft/s (5090 to 5220 m/s). Thus at later ages the pulse velocity of concrete is not sensitive to gain in strength.

Factors other than concrete strength can affect pulse velocity, and changes in pulse velocity due to these factors may overshadow changes due to strength (Sturrup, Vecchio, and Caratin 1984). One of the most important factors is moisture content. As the moisture content of concrete increases from the air-dry to saturated condition, it is reported that pulse velocity may increase up to 5 percent (Bungey 1982). Thus, if the effects of moisture are not taken into account, erroneous conclusions may be drawn about in-place strength, especially in mature concrete. Empirical correlation relationships between compressive strength and pulse velocity should be determined at moisture conditions similar to those expected for the concrete in place.

Another influencing factor is the presence of steel reinforcement. Since the pulse velocity through steel is about 40 percent greater than through concrete, the pulse velocity through a heavily reinforced concrete member may be greater than through one with little reinforcement. This is especially troublesome when reinforcing bars are oriented parallel to the pulse-propagation direction. The pulse may be refracted into the bars and transmitted to the receiver at the pulse velocity in steel. The resulting apparent velocity through the member will be greater than the actual velocity through the concrete. Failure to account for the presence and orientation of reinforcement may lead to incorrect conclusions about concrete strength. Correction factors, such as those discussed in Malhotra (1976) and Bungey (1982), have been proposed, but their reliability is questionable.

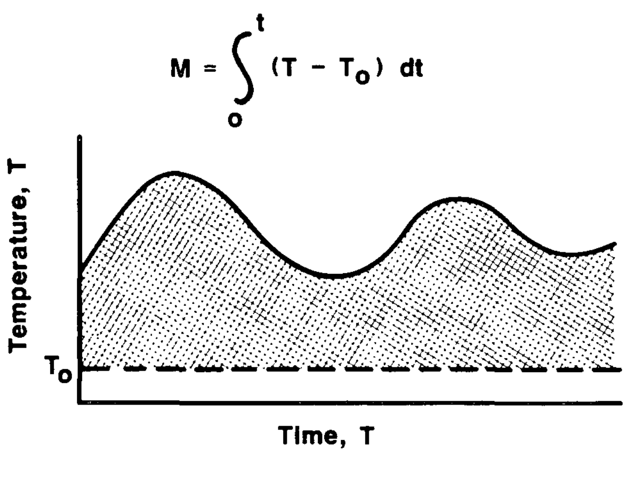

Fig. 2.8—Maturity function based on assumption that rate of strength gain varies linearly with temperature

The measured pulse velocity may also be affected by the presence of cracks or voids along the propagation path from transmitter to receiver. The pulse may be diffracted around the discontinuities, thereby increasing the travel path and travel time. Without additional knowledge about the interior condition of the concrete member, the apparent decrease in pulse velocity could be incorrectly interpreted as a low compressive strength.

In this test method, the concrete between the transmitting and receiving transducers affects the travel time. Test results are, therefore, relatively insensitive to the normal heterogeneity of concrete. For this reason, the test method has been found to have an extremely low within-batch coefficient of variation. However, these favorable results should not be interpreted to mean that highly reliable in-place strength predictions can be routinely obtained.

2.6—Maturity method

Freshly placed concrete gains strength as a result of the exothermic chemical reactions between the water and cementitious materials in the mixture. Provided sufficient moisture is present, the rates of the hydration reactions are influenced by the concrete temperature; an increase in temperature causes an increase in the reaction rates. The extent of hydration and, therefore, strength at any age is a function of the thermal history of the concrete.

The maturity method is a technique for accounting for the combined effects of temperature and time on strength development. The thermal history of the concrete and a so-called "maturity function" are used to compute a "maturity" value that quantifies the combined effects of time and temperature. The strength of a particular concrete mixture is expressed as a function of its maturity by means of a "strength-maturity relationship." If samples of the same concrete are subjected to different curing conditions, the strength-maturity relationship for that concrete and the thermal histories of the samples can be used to predict their strengths.

The maturity function is a mathematical expression that converts the thermal history of the concrete to a maturity value. Several such functions have been proposed and are reviewed in Malhotra (1971) and RILEM (1981). The key feature of a maturity function is the expression used to represent the influence of temperature on the rate of strength development. Two expressions have found widespread usage. In one case it is assumed that the rate of strength development is a linear function of temperature, and this leads to the simple maturity function shown in Fig. 2.8. In this case the maturity at any age is quantified by the area between a datum temperature T_o and the temperature curve of the concrete. The term "temperature-time factor" is used for this area and is calculated as follows

$$M(t) = \Sigma (T_a - T_o) \Delta t \tag{2-1}$$

where $M(t)$ = the temperature-time factor at age t, degree-days or degree-hours
Δt = a time interval, days or hours
T_a = average concrete temperature during time interval Δt
T_o = datum temperature

Traditionally, the datum temperature has been the temperature below which strength gain ceases, which has been assumed to be about 14 F (– 10 C). However, it has been suggested that a single value for the datum temperature is not the most appropriate approach and that the datum temperature should be evaluated for the specific materials in the concrete mixture (Carino 1984). ASTM C 1074 recommends a datum temperature of 32 F (0 C) for concrete made with Type I cement when the concrete temperature is expected to be between 32 and 104 F (0 and 40 C). A procedure is also provided for experimentally determining the datum temperature for other types of cement and for different ranges of curing temperature.

The other expression used in a maturity function assumes that the rate of strength gain varies exponentially with concrete temperature. This exponential function is used to compute an "equivalent age" of the concrete at some specified temperature as follows

$$t_e = \Sigma e^{-Q\,(1/T_a\,-\,1/T_s)}\,\Delta t \qquad \text{(2-2)}$$

where t_e = equivalent age at a specified temperature T_s, days or hours
Q = activation energy divided by the gas constant, deg K
T_a = average temperature of concrete during time interval Δt, deg K
T_s = specified temperature, deg K
Δt = time interval, days or hours

To use the exponential function one needs the value of a characteristic known as the "activation energy," which depends on the nature of the cementitious materials. The relationships between activation energy and datum temperature are described by Carino (1984). ASTM C 1074 recommends a Q-value of 5000 deg K, for use in Eq. (2-2), for concrete made with Type I cement.

To utilize the maturity method requires establishing the strength-maturity relationship for the concrete that will be used in the structure. The temperature history of the in-place concrete is continuously monitored and from this data the in-place maturity (temperature-time factor or equivalent age) is computed. Knowing the in-place maturity and strength-maturity relationship, the in-place strength can be estimated. Instruments are available that automatically compute maturity, but care should be exercised in their use because the maturity function used by the instrument may not be applicable to the concrete in the structure. ASTM C 1074 gives the procedure for using the maturity method and provides examples of how to compute the temperature-time factor or equivalent age from the recorded temperature history of the concrete. ACI 306R also illustrates the use of the temperature-time factor (referred to as the "maturity factor" in ACI 306R).

The maturity method is intended for estimating strength development of concrete. Strength estimates are based on two important assumptions: 1) there is always sufficient water for continued hydration, and 2) the concrete in the structure is the same as that used to develop the strength-maturity relationship. Proper curing procedures (as provided in ACI 308) will assure that the first condition is satisfied. To satisfy the second condition requires additional confirmation that the concrete in the structure has the correct strength potential. This can be achieved by performing accelerated strength tests on concrete sampled from the structure or by performing other in-place tests that give positive indications of the strength level. This verification is essential when estimates of in-place strength are used for timing critical operations such as formwork removal or application of post-tensioning.

2.7—Cast-in-place cylinders

This is a technique for obtaining cylindrical concrete specimens from newly cast slabs without drilling cores. The method is described in ASTM C 873 and involves using a mold, as illustrated in Fig. 2.9. The outer sleeve is nailed to the formwork and is used to support a cylindrical mold. The sleeve can be adjusted for different slab thicknesses. The mold is filled when the slab is cast and the concrete in the mold is allowed to cure along with the slab. The objective of the technique is to obtain a test sample that has been subjected to the same

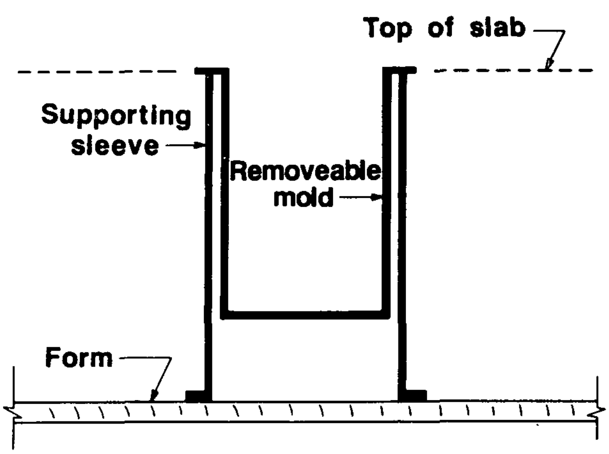

Fig. 2.9—Special mold used to obtain cast-in-place cylindrical concrete sample

Table 2.1 — Summary of survey on usage of in-place tests

Type of test	No. of labs performing test		
	Canada	U.S.	Total
Rebound hammer	19	27	46
Probe penetration	2	22	24
Pullout	6	6	12
Pulse velocity	3	10	13
Maturity	3	1	4
Cast-in-place cylinder	5	3	8

Table 2.2 — Comparison of experience rating

Type of test	Canada				U.S.			
	R	S	A	E	R	S	A	E
Rebound hammer	F-G	VG-E	F-G	VG-E	F	G-VG	F-G	G-VG
Probe penetration	F-G	VG	G	VG	G	G	G	F-G
Pullout	G-VG	G-VG	VG	G-VG	G	G	G	F-G
Pulse velocity	G-VG	VG	G-VG	G-VG	F-G	F	F-G	F-G
Maturity	F	F-P	G	F-P	G	P	G	P
Cast-in-place cylinder	VG	VG-E	VG	VG	G	G-VG	G	F-G

Legend

R = reliability
S = simplicity
A = accuracy
E = economy

P = poor
F = fair
G = good
VG = very good
E = excellent

thermal history as the concrete in the structure. When it is desired to know the in-place strength, the mold is removed from the sleeve and stripped from the concrete cylinder. The cylinder is capped and tested in compression. For cases in which the length-to-diameter ratio of the cylinders is less than 2, the measured compressive strengths need to be corrected by the factors in ASTM C 42.

2.8—Combined methods

Data from two methods can be combined in a single correlation relationship to improve accuracy (e.g., rebound hammer plus ultrasonic pulse velocity, or probe penetration plus ultrasonic pulse velocity). Combinations such as pulse velocity and rebound hammer have been reported to improve accuracy, but in most applications the improvement has only been marginal (Tanigawa, Baba, and Mori 1984; Samarin and Dhir 1984; Samarin and Meynink 1984). Hence, the use of combined methods may not be economically justified.

2.9—Summary

A review has been presented of methods that can be used for estimating the in-place strength of concrete. While other procedures have been proposed (see Malhotra 1976 and Bungey 1982), the methods that have been discussed are the ones commonly used in North American practice. A survey was conducted of commercial testing laboratories in Canada and the United States to determine the relative usage of these tests. A total of 25 laboratories from Canada and 27 laboratories from the U.S. responded to the survey, the results of which are summarized in Table 2.1. The survey indicates that the rebound hammer is the most widely used in-place test method, and the probe penetration method is the second most common test. The laboratories were also asked to rate each of the methods in terms of their reliability, simplicity of use, accuracy, and economy. The results of this rating are summarized in Table 2.2. Because of the small number of laboratories using techniques other than the rebound hammer and probe penetration methods, it is difficult to reach definite conclusions about the performance rating of all the methods. However, it is clear that while the rebound hammer is widely used because of its simplicity and economy, it is not the method of choice in terms of reliability and accuracy. The probe-penetration and pullout test appear to have good to very good ratings in the four categories. There seems to be a difference of opinion between U.S. and Canadian users of pulse velocity, but this may be due to the small sample size rather than a difference in practices.

In-place tests provide alternatives to core tests for estimating the strength of concrete in a structure, or can supplement the data obtained from a limited number of cores. These methods are based on measuring a concrete property that bears some relationship to strength. The accuracy of these methods is, in part, determined by the degree of correlation between strength and the physical quantity measured by the in-place test. For proper evaluation of test results, the user must be aware of those factors other than concrete strength that can affect the test results. Additional fundamental research is needed to improve our understanding of how these methods are related to concrete strength and how the test results are affected by factors other than strength.

An essential step for using these methods to estimate the in-place strength is the development of a correlation relationship between strength and the quantity measured by the in-place test. The data acquired for developing the correlation relationship provide valuable information on the reliability of the predictions. Chapter 3 of this report gives recommendations for the development and use of correlation relationships.

CHAPTER 3—STATISTICAL ANALYSIS OF TEST RESULTS

3.1—Introduction

In designing a structure to safely resist the expected loads, the engineer uses the specified compressive strength f'_c of the concrete. The strength of the concrete in a structure is variable and, as indicated in ACI 214, the specified compressive strength is generally assumed to represent the strength which is expected to be exceeded with about 90-percent probability. To insure that this condition is satisfied, the concrete supplied for the structure must have an average standard-cured cylinder strength in excess of f'_c as specified in Chapter 4 of ACI 318. When the strength of concrete in a structure is under scrutiny because of low standard-cured cylinder strengths or because of suspected curing deficiencies, ACI 318 states that the concrete is structurally adequate if the in-place strength, as represented by the average core strength, is not less than $0.85 f'_c$.

In assessing the ability of a partially completed structure to safely resist construction loads, it is therefore reasonable that the tenth-percentile in-place compressive strength (strength exceeded with 90-percent probability) should be equal to at least 0.85 of the required compressive strength at the time of application of the construction loads. The required strength means the compressive strength used in computing the nominal load resistance of structural elements. In-place tests can be used to estimate the tenth-percentile strength with a high degree of confidence only if test data are subjected to statistical analysis.

The use of the tenth-percentile strength as the level of the in-place strength that should be relied upon to resist applied construction loads is perceived as a reasonable procedure by users of in-place tests. The critical nature of construction operations in partially completed structures, the sensitivity of early age strength on the previous thermal history of the concrete, and the general lack of careful consideration of construction loading during the design of a structure, dictate the use of a conservative procedure for evaluating in-place test results. Inadequate strength at the time of a proposed construction operation can usually be remedied by simply providing for additional curing before proceeding with the operation.

Some in-place tests may also be used to evaluate the strength of an existing structure. Often they are used to answer questions that arise because of low strengths of standard cylinders. Failure to meet specified acceptance criteria can result in severe penalties for the builder. In such cases, the use of the tenth-percentile strength as the reliable strength level to resist design loads is not the appropriate technique for analyzing in-place test data. The existing criteria for the acceptance of concrete strength in an existing structure are based on testing cores. If the average compressive strength of three cores exceeds 85 percent of the specified compressive strength and no single core strength is less than 75 percent of the specified strength, the concrete strength is deemed to be acceptable. However, there are no analogous acceptance criteria for equivalent in-place compressive strength based on in-place tests. Additional study is required to develop acceptance criteria for the results of in-place tests that are consistent with the existing criteria for core strengths.

In arriving at the value of the tenth-percentile in-place compressive strength by means of in-place tests, one must account for various sources of variability or uncertainty:

1. The uncertainty of the average value of the in-place test results based on a limited number of in-place tests.
2. The uncertainty of the correlation relationship derived from a limited number of correlation tests.
3. The variability of the in-place compressive strength.

The following sections deal with these items.

3.2—Repeatability of results

The uncertainty of the average value of the in-place test results is indicated by the standard deviation of the results and by the number of tests. The standard deviation is in turn a function of the repeatability of the test and the variability of the concrete in the structure.

In this report, repeatability is defined as the standard deviation or coefficient of variation of repeated tests by the same operator on the same material. This is often called the "within-test" variation and is indicative of the inherent scatter associated with a particular test.

Data on the repeatability of some in-place tests are provided in the precision statements of the ASTM standards governing the tests. Some information on the repeatability of other tests may be found in published reports. Unfortunately, the majority of published data deal with correlations with standard strength tests, rather than with repeatability. As will be seen, conclusions about repeatability are often in conflict because of differences in test designs or in data analysis.

3.2.1 *Rebound hammer*—In the precision statement of ASTM C 805, the within-test standard deviation is reported to be 2.5 rebound numbers. The results of two recent studies (Keiller 1982; Carrette and Malhotra 1984), which were performed to evaluate the performance of various in-place tests, provide additional insight into the repeatability of the rebound test. In one study (Keiller 1982), eight different mixtures were used and twelve replicate rebound readings were taken at ages of 7 and 28 days. In the other study (Carrette and Malhotra 1984), four mixtures were used along with twenty replications at ages of 1, 2, and 3 days. Fig. 3.1 shows the standard deviations of the rebound numbers as a function of the average rebound number. The data from the two laboratories appear to be consistent. The average standard deviation for the CANMET study (Carrette and Malhotra 1984) is 2.4 rebound numbers, which is remarkably close to that quoted in ASTM C 805. However, the average standard deviation in the Cement and Concrete Association (C & CA) study (Keiller 1982) is 3.4 rebound numbers. In Fig. 3.1, there appears to be a trend of increasing standard deviation with increasing rebound number. If such is the case, the coefficient of variation may be a better measure of repeatability. Fig. 3.2 shows the coefficients of variation plotted as a function of average rebound number. In this case there does not appear to be any clear trend with increasing rebound number. The average values of coefficient of variation for the two laboratories are exactly the same, 11.9 percent.

In Fig. 3.2, the coefficients of variation are not constant. However, it must be realized that the values are based on sample estimates of the true averages and standard deviations. With finite sample sizes there will

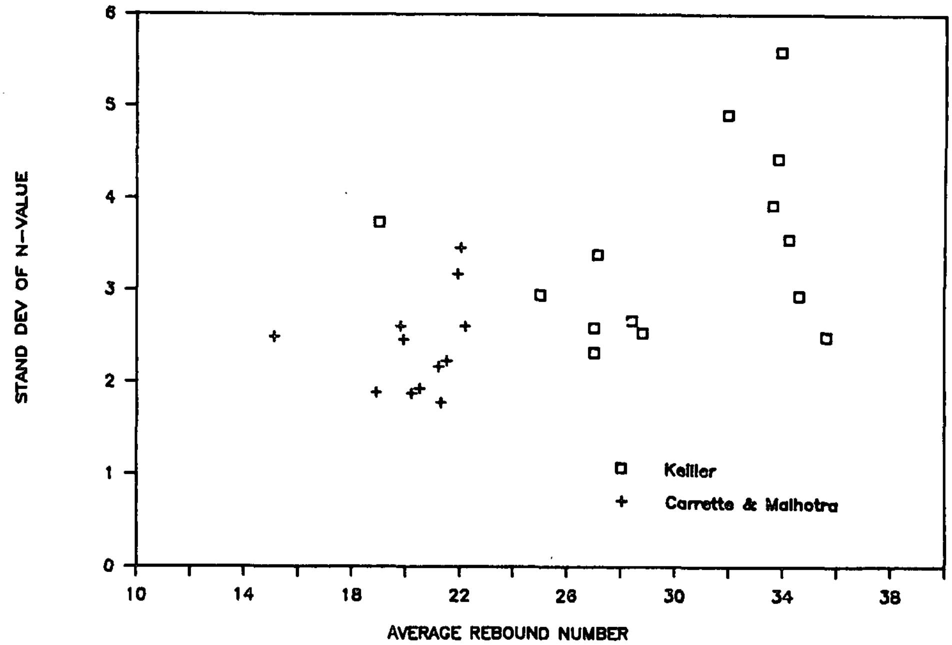

Fig. 3.1— Within-test standard deviation as a function of average rebound number

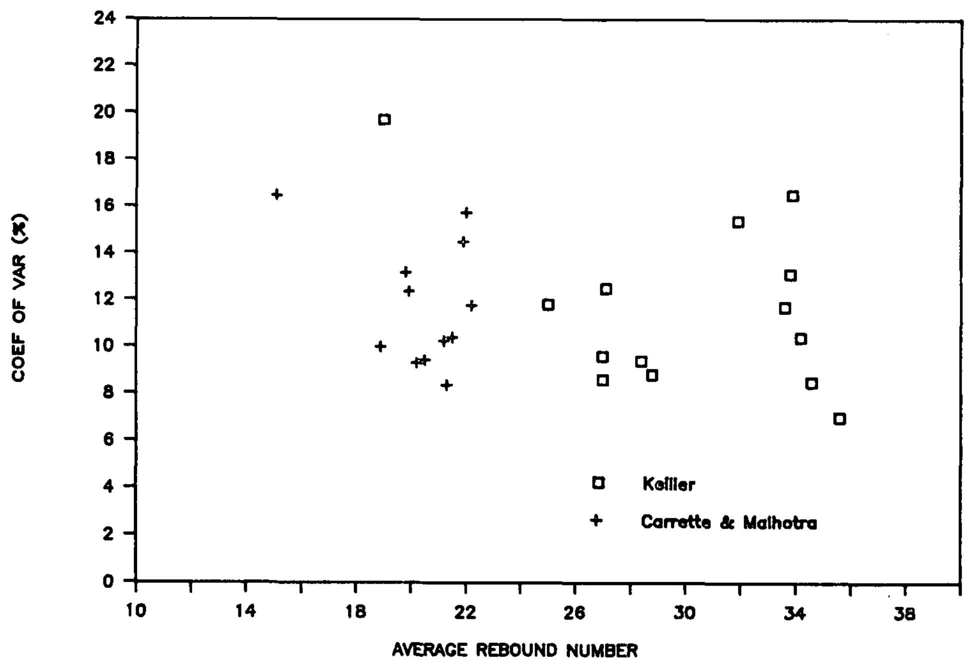

Fig. 3.2—Within-test coefficient of variation as a function of average rebound number

be errors in these estimates, and a random variation in the computed coefficient of variation is expected even though the true coefficient of variation is constant. Thus it appears that the repeatability of the rebound number technique may be described by a constant coefficient of variation, which has an average value of about 10 percent.

3.2.2 *Probe penetration*—In the precision statement of ASTM C 803, it is reported that the within-test standard deviations of exposed probe length for three replicate tests are 0.08, 0.10, and 0.14 in. (2.0, 2.5, and 3.6 mm) for tests in mortar, in concrete with 1 in. (25 mm) maximum aggregate size, and in concrete with 2 in. (50 mm) maximum aggregate size, respectively.

The data provided in the CANMET (Carrette and Malhotra 1984) and C & CA (Keiller 1982) studies, which cover concrete strengths in the range of 1500 to 7000 psi (10 to 50 MPa), give additional insight into the

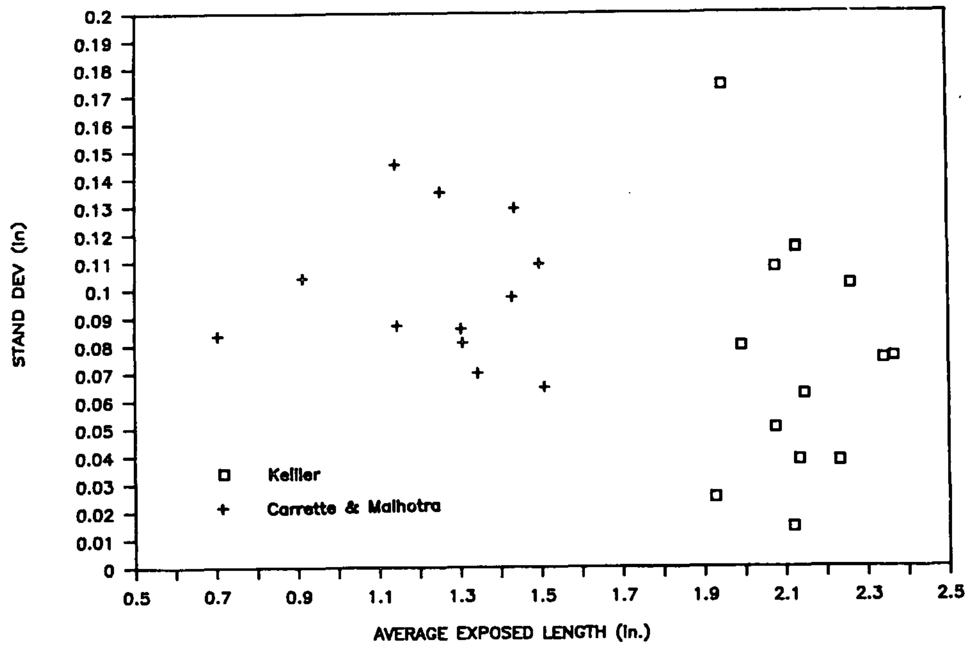

Fig. 3.3—Within-test standard deviation as a function of average exposed length of probes

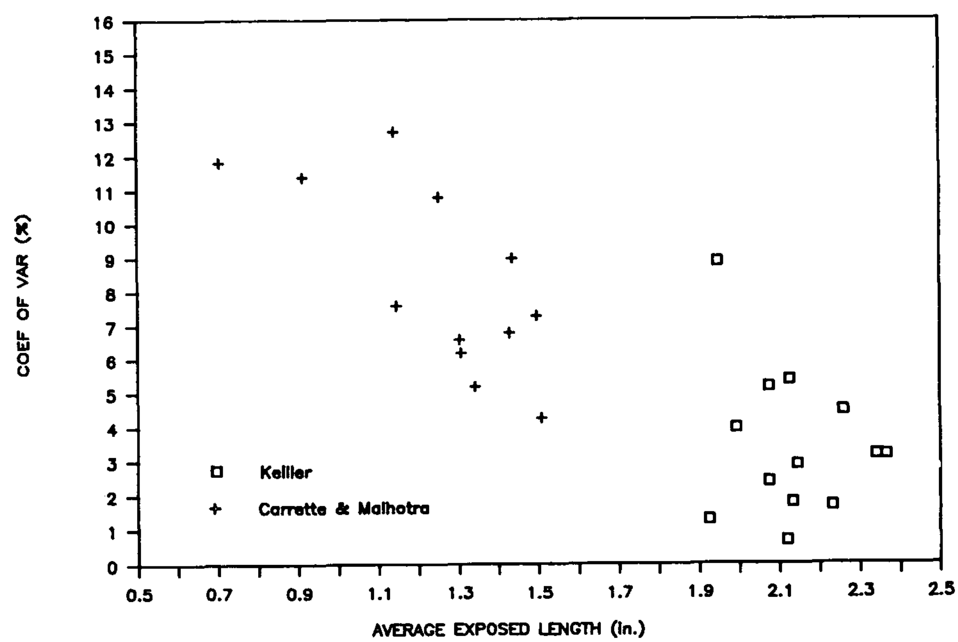

Fig. 3.4—Within-test coefficient of variation as a function of average exposed length of probe

underlying measure of repeatability for this test. Fig. 3.3 shows the standard deviations of the exposed length of the probes as a function of the average exposed length. The CANMET values are based on the average of six probes, while C & CA results are based on three probes. Except for one outlying point, there is a trend for decreasing within-test variability with increasing exposed length. In Fig. 3.4, the coefficients of variation of exposed length are shown as a function of the average exposed length. The decreasing trend with increasing concrete strength is more pronounced than in Fig. 3.3. Thus the repeatability of the exposed length is neither described by a constant standard deviation nor a constant coefficient of variation.

The customary practice is to measure the exposed length of the probes, but concrete strength has a direct effect on the depth of penetration. A more logical approach is to express the coefficient of variation in terms

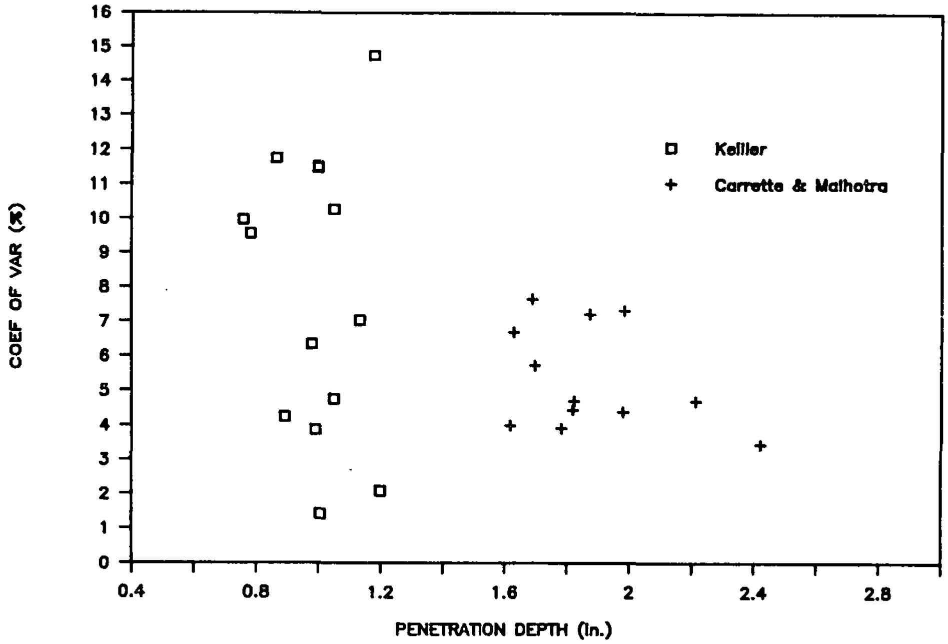

Fig. 3.5—Within-test coefficient of variation as a function of average penetration depth of probes

of *depth of penetration*. Fig. 3.5 shows the coefficient of variation of the penetration depth as a function of average penetration. In this case there is no clear trend with increasing penetration. The higher scatter of the values from the C & CA tests may be due to their smaller sample size compared with the CANMET tests. Note that the standard deviation has the same value whether exposed length or penetration depth is used. However, the value of the coefficient of variation depends on whether the standard deviation is divided by average exposed length or average penetration depth.

Hence, it appears that a constant coefficient of variation of the penetration depth can be used to describe the within-test variability of the probe penetration test. The CANMET work (Carette and Malhotra 1984) is the first known study that uses this method for defining the repeatability of the penetration test. However, other test data using the probe penetration system can be manipulated to yield the coefficient of variation of penetration depth provided two of these three quantities are given: average exposed length; standard deviation; and/or coefficient of variation of exposed length. Using the data given in Table 6 of Malhotra's 1976 review, the following values for average coefficients of variation for depth of penetration have been calculated:

Maximum aggregate size	Coefficient of variation of penetration depth
2 in. (50 mm)	14.0%
1 in. (25 mm)	8.6%
¾ in. (19 mm)	3.5, 4.7, and 5.6%

In the CANMET study (Carrette and Malhotra 1984), the maximum aggregate size was ¾ in. (19 mm) and the average coefficient of variation is 5.4 percent, while in the C & CA study (Keiller 1982) it was 7.8 percent for the same maximum aggregate size. Other work (Swamy and Al-Hamad 1984) used ⅜ in. (10 mm) maximum aggregate size, and the coefficients of variation ranged between 2.7 and 7 percent. For commonly used ¾ in. (19 mm) aggregate, it is concluded that a coefficient of variation of 5 percent is reasonable.

3.2.3 *Pullout test*—The current ASTM test method for the pullout test (ASTM C 900) does not have a precision statement. However, there are published data on the within-test variability of this test. Recent work at the National Bureau of Standards (NBS) (Stone, Carino, and Reeve 1986) examined whether standard deviation or coefficient of variation is the best measure of repeatability. Four test series were performed. Three of them used a 70-deg apex angle but different aggregate types: river gravel, crushed limestone, and expanded lightweight shale. The fourth series was for a 54-deg angle with river-gravel aggregate. The embedment depth was about 1 in. (25 mm), and compressive strength of concrete ranged from about 1500 to 6000 psi (10 to 40 MPa). Fig. 3.6 shows the standard deviation, using 11 replications, as a function of the average pullout load. It can be seen that there is a tendency for the standard deviation to increase with increasing pullout load. Fig. 3.7 shows the coefficient of variation as a function of the average pullout load. In this case there is no trend between the two quantities. Thus, it may be concluded that the coefficient of variation should be used as a measure of the repeatability of the pullout test.

Table 3.1 gives the reported coefficients of variation from different laboratory studies of the pullout test. In addition to these data, the work of Krenchel and Peter-

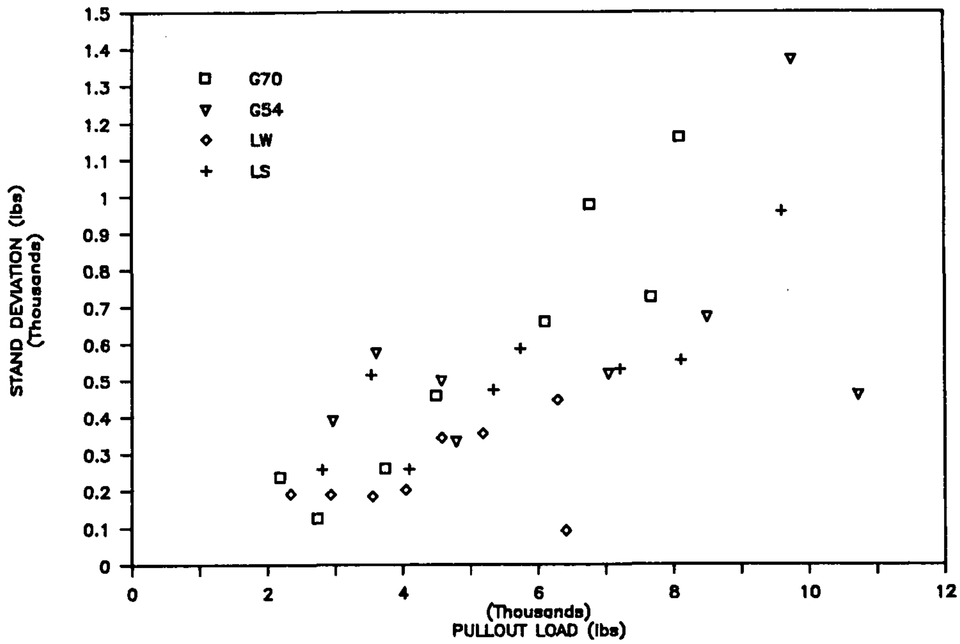

Fig. 3.6—Within-test standard deviation as a function of average pullout load (Stone, Carino, and Reeve 1986)

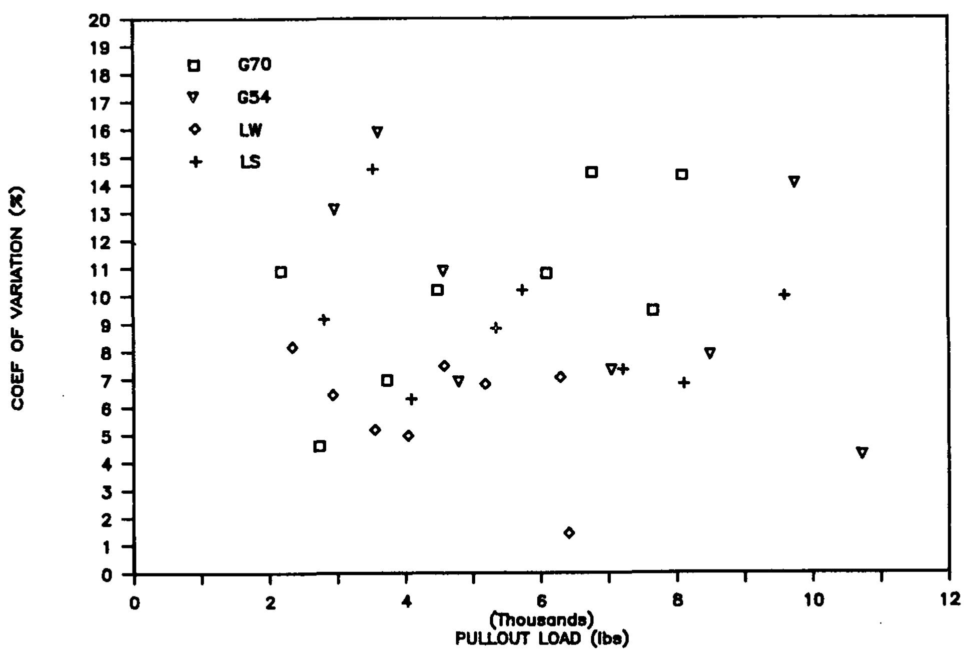

Fig. 3.7—Coefficient of variation as a function of average pullout load (Stone, Carino, and Reeve 1986)

son* summarizes the repeatability obtained in 24 correlation testing programs involving an insert with a 1 in. (25 mm) embedment and a 62-deg apex angle. The reported coefficients of variation ranged from 4.1 to 15.2 percent, with an average of 8 percent. The tests reported in Table 3.1 and by Krenchel and Peterson† have involved different test geometries and different types and sizes of coarse aggregate. In addition, the geometry of the specimens containing the embedded inserts were varied, with cylinders, cubes, beams, and slabs being common shapes. Because of these testing differences, it is difficult to draw firm conclusions about the repeatability of the pullout test.

Table 3.2 summarizes the coefficients of variation obtained in a study by NBS (Stone and Giza 1985) designed to examine the effects of different variables on

*Krenchel, H. and Peterson, C. G., "In Situ Pullout Testing with Lok-Test—Ten Years Experience," manuscript submitted for publication to *Nordisk Betong*.

†Ibid.

Table 3.1 — Summary of within-test coefficient of variation of pullout test

Reference	Apex angle	Embedment depth, in.	Maximum aggregate size, in.	Type of aggregate	Sample size	Range of C.V.,* percent	Average C.V., percent
Malhotra & Carrette 1980	67	2	1	R.Gravel'	2	0.9-14.3	5.3
Malhotra 1975	67	2	¾	Limestone	3	2.3-6.3	3.9
Bickley 1982	62	1	⅜	?	8	3.2-5.3	4.1
Khoo 1984	70	1	¾	Granite	6	1.9-12.3	6.9
Carrette & Malhotra 1984	67	2	¾	Limestone	4	1.9-11.8	7.1
Carrette & Malhotra 1984	62	1	¾	Limestone	10	5.2-14.9	8.5
Keiller 1982	62	1	¾	Limestone	6	7.4-31	14.8
Stone, et al. 1986	70	1	¾	R.Gravel	11	4.6-14.4	10.2
	70	1	¾	Limestone	11	6.3-14.6	9.2
	70	1	¾	Lightweight	11	1.4-8.2	6.0
	54	1	¾	R.Gravel	11	4.3-15.9	10.0
Bocca 1984	67	1.2	½	?	24	2.8-6.1	4.3

1 in. = 25.4 mm.
*C.V. = Coefficient of variation.
' R.Gravel = river gravel.

Table 3.2 — Summary of results from investigation of pullout test (Stone and Giza 1985)

Test series	Apex angle	Embedment depth, in.	Maximum aggregate size, in.	Type of aggregate	Sample size	Range of C.V.,* percent	Average C.V., percent
Apex angle	30	0.98	¾	R.Gravel'	2 x 11	9.1-11.4	10.3
	46	0.98	¾	R.Gravel	4 x 11	5.6-18.7	11.1
	54	0.98	¾	R.Gravel	2 x 11	6.3-6.7	6.5
	58	0.98	¾	R.Gravel	2 x 11	8.6-10.0	9.3
	62	0.98	¾	R.Gravel	2 x 11	7.5-9.6	8.6
	70	0.98	¾	R.Gravel	4 x 11	8.0-10.1	8.8
	86	0.98	¾	R.Gravel	2 x 11	9.0-10.8	9.9
Embedment	58	0.47	¾	R.Gravel	1 x 11		12.9
	58	0.78	¾	R.Gravel	2 x 11	7.7-14.0	10.9
	58	0.91	¾	R.Gravel	2 x 11	6.5-6.7	6.6
	58	0.98	¾	R.Gravel	2 x 11	8.8-10.7	9.8
	58	1.06	¾	R.Gravel	2 x 11	9.1-11.1	10.1
	58	1.69	¾	R.Gravel	2 x 11	11.5-11.9	11.7
Aggregate size	70	0.98	¼	R.Gravel	2 x 11	6.5-7.0	6.8
	70	0.98	⅜	R.Gravel	5 x 11	4.9-6.5	6.0
	70	0.98	½	R.Gravel	5 x 11	3.3-10.6	6.7
	70	0.98	¾	R.Gravel	4 x 11	8.0-10.1	8.8
Aggregate type	70	0.98	¾	Lightweight R.Gravel	2 x 11	5.6-5.7	5.7
	70	0.98	¾	Crush	4 x 11	8.0-10.1	8.8
	70	0.98	¾	Gneiss	2 x 11	7.2-16.8	12.0
	70	0.98	¾	Porous Limes	2 x 11	7.7-10.9	9.3

*C.V. = coefficient of variation.
'R.Gravel = river gravel.

test repeatability. The column labeled "sample size" indicates the number of groups of tests, with each group containing 11 replications. For the conditions studied it was found that depth of embedment and apex angle did not have a pronounced effect on repeatability. On the other hand, the maximum nominal aggregate size appeared to have some effect, with the ¾-in. (19-mm) aggregate resulting in slightly greater variability than for smaller aggregates. In addition, the aggregate type also appears to be important. For tests with lightweight aggregate, the variability was lower than for tests with normal weight aggregates. In this study, companion mortar specimens were also tested and the coefficients of variation varied between 2.8 and 10.6 percent, with an average value of 6.2 percent. Thus, the repeatability with lightweight aggregate is similar to that obtained with mortar.

Experimental evidence suggests that the variability of the pullout test is affected by the ratio of the mortar strength to coarse aggregate strength and by the maximum aggregate size. As aggregate strength and mortar strength become similar, repeatability is improved. This explains why the NBS tests with lightweight aggregate performed like tests with plain mortar. Bocca's results (Bocca 1984), which are summarized in Table 3.2, also lend support to this pattern of behavior. In this case, high-strength concrete was used and the high mortar strength approached that of the coarse aggregate. This condition, plus the use of small maximum aggregate size, may explain why the coefficients of variation were lower than typically obtained with similar pullout test configurations on lower strength concrete.

In summary, a wide variety of test data has been accumulated on the repeatability of the pullout tests. Differing results can often be explained because of differences in the materials and the testing conditions. In general, it appears that an average coefficient of variation of 8 percent is typical for pullout tests conforming with the requirements of ASTM C 900 and with embedment depths on the order of 1 in. (25 mm). The actual value expected in any particular situation will be primarily affected by the nature of the coarse aggregate, as discussed in previous paragraphs.

3.2.4 *Pulse velocity*—In contrast to the previous test techniques, which probe a relatively thin layer of the concrete in a structure, the pulse-velocity method probes the entire thickness of concrete between the transducers. Any localized differences in the composition of the concrete because of inherent variability are expected to have a negligible effect on the measured travel times of the ultrasonic pulses. Thus the repeatability of this method is expected to be much better than the previous techniques.

The following coefficients of variation have been obtained in various laboratory studies:

Reference	Range of coefficient of variation	Average coefficient of variation
Keiller	0.5 to 1.5%	1.1%
Carrette and Malhotra	0.1 to 0.8%	0.4%
Bocca	0.4 to 1.2%	0.7%

In ASTM C 597, it is reported that for path lengths from 0.3 to 6 m through sound concrete and for different operators using the same instrument or one operator using different instruments, the repeatibility of test results is within 2 percent.

3.2.5 *Maturity method*—In the maturity method, the temperature history of the concrete is recorded and used to compute a maturity value. Therefore, the repeatability of the maturity values is dependent on the instrumentation that is used. One would expect the repeatability to be lower when using an electronic "maturity meter" than when maturity is computed from temperature readings on a strip-chart recorder. However, at present there are no published data on repeatability of maturity measurements using different instrumentation.

3.2.6 *Cast-in-place cylinder*—This test method involves the determination of the compressive strength of cylindrical specimens cured in the special molds located in the structure. Thus, the repeatability would be expected to be similar to other compression tests on cylinders. Few data have been published. Bloem (1968) reported a within-test coefficient of variation ranging from 2.7 to 5.2 percent with an average of 3.5 percent for three replicate tests at ages from 1 to 91 days. Richards* reported values from 1.2 to 5.8 percent with an average of 2.8 percent for two replicate tests at ages of from 7 to 64 days.

ASTM C 873 states that the coefficient of variation for three cylinders cast from the same batch has been determined to be 3.6 percent.

3.3—Correlation

3.3.1 *General*—The term "correlation" as used in this report means establishing the relationship between the quantities measured by each type of in-place test and the corresponding compressive strength of standard specimens. The standard specimen may be the standard cylinder, such as used in North American practice, or standard cubes. Very often, the in-place tests are correlated with the compressive strength of cores, since core strength is the most established and accepted measure of in-place strength. The statistical techniques for establishing the correlation relationship are independent of the type of "standard" specimen. However, the specimen type is important when interpreting the results of in-place tests.

The common aspects of correlation are discussed in this section and subsequent sections present the specific procedures being used for each type of in-place test.

The preferred approach is to establish the correlation by a laboratory testing program, which is performed prior to using the in-place test method in the field. The testing program typically involves preparing test specimens using the same concreting materials to be used in construction. At regular intervals, measurements are made using the in-place test techniques, and the compressive strengths of standard specimens, such as molded cylinders or cores, are also measured. The paired data are subjected to regression analysis to determine the best-fit estimate of the correlation relationship.

For some techniques it may be possible to perform the in-place test on standard specimens without damaging them, and the specimens can be subsequently tested for compressive strength. However, in most cases in-place tests are carried out on separate specimens, and it is extremely important that the in-place tests and standard tests are performed on specimens at the same maturity and compaction. This may be achieved by using curing conditions that insure similar internal temperature histories in all specimens. Alternatively, internal temperatures can be recorded and test ages can

*Personal communication from Owen Richards.

be adjusted so that the in-place tests and standard tests are performed at the same maturity.

The testing program should be planned so that the range of compressive strength includes the lowest and highest values that may be encountered in the field. This assures that the correlation relationship will be used only for interpolation and not extrapolation. The uncertainty associated with the correlation relationship is reduced with increasing number of tests, but of course the expense of the testing program would increase. It is not easy to determine the optimum number of points, i.e., strength levels, to use for correlation. However, it is recommended that as a minimum, six strength levels that are approximately evenly spaced should be used. This is achieved by testing specimens from the same mixture at different ages.

The number of replicate tests at each strength level should be chosen so that the compressive strength and in-place test values are measured with the same degree of certainty. Thus the ratio of the number of in-place tests to the number of standard tests equals the ratio of the squares of the corresponding within-test coefficients of variation

$$\frac{n_i}{n_s} = \left(\frac{V_i}{V_s}\right)^2 \tag{3-1}$$

where n_i = number of in-place tests
n_s = number of standard tests
V_i = coefficient of variation of in-place test
V_s = coefficient of variation of standard test

For planning purposes, it may be assumed that the coefficient of variation of compressive strength of standard cylinders is 4 percent. If compressive strength is determined by testing cores, a coefficient of variation of 5 percent may be assumed.

The usual practice is to treat the average values of the replicate compressive strength and in-place tests at each strength level as one data pair. The data pairs are plotted using the in-place test value as the X-variable and compressive strength as the Y-variable. Regression analysis is performed on the data pairs to obtain the best estimate for the correlation relationship. It is common to assume that the relationship is a straight line and to use programmed handheld calculators to determine the best estimates of the slope and Y-intercept of the line. Ordinary linear regression analysis assumes that there is no error in the X-variable, which is clearly not a good assumption for in-place tests. A procedure has been developed for modifying the formulas in ordinary linear regression which account for the error in the X-variable (Mandel 1984).

It is common practice to compute the correlation coefficient of the regression line. This quantity indicates how well the data fit the straight line. A more relevant statistic is the "standard error of estimate," which is the weighted average deviation of the Y-values from the straight line. The standard error is used to compute the confidence limits for the correlation relationship, which are useful when evaluating in-place compressive strength from the results of the in-place tests. The formulas for computing these confidence limits are given in books on experimental data analysis (Natrella 1963).

3.3.2 *Rebound hammer*—At each test age, a set of ten rebound numbers should be obtained from each of a pair of cylinders held firmly in a compression testing machine or other suitable device at a load of about 500 psi (3 MPa). The rebound hammer tests should be made in the direction relative to gravity in which they will be made on the structure. The cylinders should then be tested in compression. If it is not feasible to test the cylinders with the hammer in the same orientation as will be used to test the structure, the correction factors supplied by the equipment manufacturer can be used to account for differences in orientation. As mentioned in Section 2.2, the surface produced by the material of the cylinder molds can differ from the surface produced by the form material for the structure. This factor should also be considered in the correlation testing. If considerable difference is expected between the surfaces in the structure and of the cylinders, additional prismatic specimens should be prepared for rebound tests. These specimens should be formed with the same type of forming materials that will be used in construction and they should be similar in size to the cylinders so that they will experience similar thermal histories.

For accurate estimates of in-place strength, the moisture content and texture of the surfaces of the cylinders at the time of the correlation tests must be similar to those anticipated for the concrete in the structure at the time of in-place testing. As a practical matter, the only easily reproducible moisture condition for concrete surfaces is the saturated condition.

3.3.3 *Probe penetration*—To permit tests at six different ages, a set of 12 standard cylinders and a test slab large enough for 18 probe penetration tests should be cast. For in-place testing of vertical elements, the recommended procedure is to cast a wall specimen and take cores adjacent to the probe tests. All test specimens should be cured under identical conditions of moisture and temperature. At each test age, two compression tests and three probe penetration tests should be made.

The recommended minimum thickness for the test slab is 6 in. (150 mm). (See ASTM C 803 for minimum spacing between test probes).

3.3.4 *Pullout*—A number of techniques have been used. Pullout inserts have been cast in the bottom of standard cylinders, and a pullout test was made prior to the standard cylinder being tested in compression (Bickley 1982). In this case, the pullout test is stopped when the maximum load has been attained, which is indicated by a drop in the load with further displacement. The insert is not extracted and the cylinder can be capped and tested in compression. Alternatively, companion cylinders have been cast with and without inserts, and the pullout test has been performed on one standard cylinder and the other cylinder tested in compression. Some investigators have experienced difficulty with both of these procedures, particularly at high strengths, with radial cracking at the end of the cylinder containing the pullout insert; this is believed to result in lower ultimate pullout loads.

A third alternative has been to cast standard cylinders for compression testing and to place pullout inserts in cubes (or slabs or beams) so that the pullout tests can be made at the same time as the standard cylinder tests but in the companion specimen.

This latter approach is the preferred method, providing compaction is consistent between the standard cylinders and the cubes or other specimens containing the pullout inserts, and the maturity of all specimens tested is the same. The minimum size of cube recommended is 8 in. (200 mm) for 1 in. (25 mm) diameter inserts. Four pullouts can be placed in each cube, one in the middle of each vertical side. For each test age, cast and test two standard cylinders and perform eight pullout tests.

3.3.5 *Ultrasonic pulse velocity*—It is preferable to develop the correlation relationship from concrete in the structure. Tests should be on cores obtained from the concrete being evaluated. Tests with standard cylinders can lead to unreliable correlations because of different moisture conditions between the cylinders and the in-place concrete.

Because the geometry of the test specimens has an effect on the determination of the pulse velocity, the correlation data should represent conditions similar to the testing configuration used in the field. A convenient manner to obtain this is to select certain areas in the structure which represent various levels of pulse velocities. At these locations, it is recommended that five velocity determinations be made to assure a representative average value of the pulse velocity. For each measurement, the transducers should be uncoupled from the surface and then recoupled, to avoid systematic errors due to poor coupling. Then obtain at least two cores from each of the same locations for compressive strength testing. Pulse velocity determinations for these cores, once they have been removed from the structure, will usually not be the same as the determinations on the structure and would not be representative of the pulse velocity of the structure.

3.3.6 *Maturity*— The following procedure is given in ASTM C 1074:

Prepare cylindrical concrete specimens according to ASTM C 192 using the mixture proportions for the concrete intended for the structure. Embed temperature sensors at the centers of at least two specimens. Connect the sensors to maturity instruments or to a multichannel temperature recording device.

Moist-cure the specimens in a water bath or in a moist room meeting the requirements of ASTM C 511. At ages of 1, 3, 7, 14, and 28 days, perform compression tests according to ASTM method C 39. Test at least three specimens at each age.

At each test age, record the average maturity value for the instrumented specimens. On graph paper, plot the average compressive strength as a function of the average maturity value. Draw a best-fit curve through the data. The resulting curve is the strength-maturity relationship to be used for estimating in-place strength.

3.3.7 *Cast-in-place cylinder*—Test results should be corrected for the height-diameter ratio using the values given in ASTM C 42. Otherwise no other correlation is needed since the specimens are representative of the concrete in the placement and the test is a uniaxial compression test.

3.4—Sampling

3.4.1 *General*—The number of in-place tests to be made and their location on the structure will depend on the purpose of the tests. Generally, in-place testing falls into two categories:

1. The investigation of an older structure.
2. Tests made at an early age of the structure to determine a safe time for form removal or post-tensioning, or to control curing and reshoring.

It has also been demonstrated (Bickley 1984) that in-place testing can be used during construction for the acceptance of concrete, but additional time and study is required before widespread application of in-place testing for this purpose can be realized.

Careful planning is required to gain maximum useful information from in-place testing. Factors such as the number of tests and their location in the structure should be carefully selected. A number of ASTM Standard Practices are available, which can be of use in planning an in-place testing program.

3.4.2 *ASTM E 105—Practice for probability sampling of materials*—According to ASTM E 105, probability sampling must be used to make valid inferences about the properties of the population from the sample test results. The use of random-number tables is recommended as a means for objectively choosing which samples shall be tested. Objective sampling is important to apply probability theory to the sample statistics (average and standard deviation). For tests, such as pullout, which require preplanning, a random number scheme could be used to select where to place the testing hardware prior to concrete placement.

3.4.3 *ASTM C 823—Practice for Examination and Sampling of Hardened Concrete in Constructions*—ASTM C 823 provides a series of guidelines for implementing the requirements of ASTM E 105. The standard deals primarily with the drilling of cores or the taking of sawn samples, but there is a section dealing with the use of in-place testing. Two sampling situations may be encountered. In one situation, all of the concrete in the structure is believed to be of similar composition and quality. For this condition, random sampling should be spread out over the entire structure and the results treated together. The other situation is when there is information to suggest that the concrete in different portions of the structure may be of different composition or quality. For this situation, random sampling should be conducted within each portion of the structure that is suspected of being different. Test results from different portions of a structure should not be combined unless it is shown by statistical tests (Natrella 1963) that there are no significant differences between the means and standard deviations of the results. In selecting sampling locations, it should be kept in mind that it is well established that tests at the top of a column or slab will yield lower strength than at the bottom (Murphy 1984; Munday and Dhir 1984).

3.4.4 *ASTM E 122—Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process*— The sample size to use depends on three factors:

1. The maximum allowable difference (or error) acceptable between the sample average and the true average.
2. The variability of the test method.
3. The acceptable risk that the allowable difference is exceeded.

Fig. 3.8 shows how the sample size varies with the allowable error and with the coefficient of variation of the test for a 5-percent risk. This figure can be used to answer the question: How many in-place tests should be done to have the same confidence in the average as would be obtained from testing cores? For example, if three cores are tested and the coefficient of variation of core strengths is 5 percent, this corresponds to an allowable error of about 6 percent. If an in-place test is used with a coefficient of variation of 10 percent, then 12 tests should be done, i.e., four times the number of core tests. This only assures that the average value of the in-place test result is within about 6 percent of the true average. This is not equivalent to saying the in-place compressive strength can be predicted to within 6 percent of its true value.

This standard stresses that economy should also be considered when selecting the sample size. In some cases, increasing the sample size may only result in a minimal decrease in the allowable error, and the additional testing cost may not be justified. Fig. 3.9 (which comes from the same data as Fig. 3.8) shows how the allowable error is affected by sample size for different coefficients of variations (the risk is again 5 percent). As the sample size increases (for a particular coefficient of variation), the incremental reduction in error diminishes. Clearly there is a sample size for which the cost of an additional test is not justified. This optimum sample size will depend on the cost of the test and on the risk the user is willing to accept that the true average differs from the sample average. For this reason, a single sample size cannot be specified for each test method that should be used in all situations.

3.4.5 *Number of in-place tests*—The following typical values for coefficient of variation are suggested for guidance in selecting the number of in-place tests:

	Within-test coefficient of variation, percent
Probe penetration	5
Pullout	8
Core	5
Standard cylinder	4
Cast in-place cylinder	4
Rebound hammer	10
Ultrasonic pulse velocity	2

Assuming two standard cylinders are acceptable as a test result and using Eq. (3-1), the following numbers of the other tests would insure that the average in-place test result is known with the same degree of confidence as the average cylinder strength:

Probe penetration	3
Pullout	8
Core	3
Cast-in-place cylinder	2
Rebound hammer	12
Ultrasonic pulse velocity	5

Since the cost of taking additional pulse velocity readings at one location is minimal, the recommended number of pulse velocity tests is not based strictly on

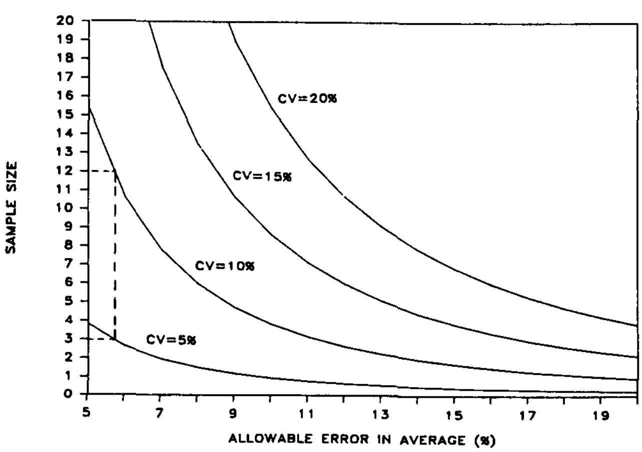

Fig. 3.8—Sample size as a function of allowable error that the true average exceeds the sample average (risk 0.05). Curves are for various values of coefficient of variation

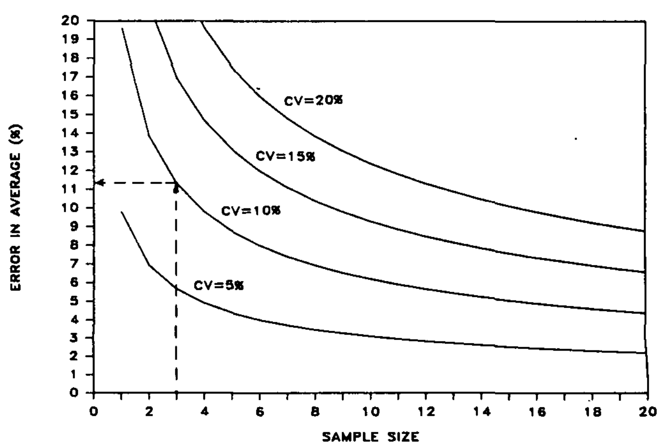

Fig. 3.9— Allowable error in average versus sample size for different coefficients of variation (risk = 0.05)

Eq. (3-1). Five readings are recommended to assure that a representative reading is obtained because of variability in the efficiency of transducer coupling to the specimen.

The pulse velocity technique is not typically used in North America for early-age strength during construction. Since the test requires access to opposite faces of a member, it is impractical for estimating concrete strength in slabs, which are often the critical elements in high-rise construction. In addition, as explained in Chapter 2, various factors (Sturrup, Vecchio, and Caratin 1984) may influence the results, and experienced personnel are needed for data interpretation. In some countries (Facaoaru 1984), the technique is used routinely. However, in this case, extensive background research has been performed and a comprehensive series of correction coefficients have been developed to account for some of these factors.

As stated in Section 3.4.1, there are two principal applications of in-place tests: 1) evaluation of concrete in existing structures, and 2) strength evaluation at early ages during construction. The recommended values for the number of tests are intended for the second application. When these techniques are used on existing structures, the number of tests should be based on engineering judgement, taking into consideration such factors as the importance of concrete strength on the overall strength of the structure, the perceived variability of concrete quality in the structure, and the inherent variability of the test method.

3.4.6 *Field practice—Early-age strength determination*—The question of how many tests to make on a placement of concrete has to be, in part, answered by the principles reviewed in the preceding and, in part, by engineering judgment. If the strength of a structural component has to be determined by test, current codes require that a minimum of three cores be obtained from that component and tested. It is, of course, the decision of the engineer to elect to have a larger number of tests made if this is judged to be prudent. If the minimum number of three cores is taken to obtain the average core strength at a given location, then based on the values for coefficients of variations suggested in Section 3.4.5, the minimum numbers of other in-place tests at a given location would be as follows:

Probe penetration	3
Pullout	8
'Cast-in-place cylinder	2
Rebound number	12

This number of in-place tests assures that the average test result is known with the same degree of certainty as the average core strength. However, this does not mean that the compressive strength of the concrete will be known with the same degree of certainty. As explained in the following section, the uncertainty of the correlation relationship affects the reliability of the estimated in-place compressive strength.

In practice, the appropriate number of tests to make has to be related to the purpose of the tests. Where concrete is tested at an early age to determine a safe time to remove forms, a larger number of tests may be required than the preceding numbers would indicate. Whatever sample sizes and locations are determined to meet statistically sound requirements, there is an overriding safety aspect that requires the application of common sense to deciding the sample size.

Where tests are made at early ages, the strength of the concrete will be more variable than at later ages. Also, extreme variations from batch to batch are more critical to safety at early ages than later ages. A floor placement of 100 yd^3 (60 m^3) might comprise 10 truckloads of concrete. The sample size must provide assurance about the strength of all batches of concrete.

A model specification developed for pullout testing, but which could be applied to all methods, states, "Install at least 15 pullout inserts per 100 m^3 pour of concrete. For pours in excess of 100 m^3, provide at least 1 additional insert per 20 m^3. Generally at least 10 tests will be made."

Where common sense suggests more tests than the number determined by statistical theory (which assumes a homogenous population), more tests should be made.

3.5—Interpretation

3.5.1 *General*—Interpretation of in-place tests should be made by the use of standard statistical procedures. It is not sufficient to simply average the values of the in-place test results and then compute the equivalent compressive strength by means of the previously established correlation relationship. It is necessary to account for the uncertainties that exist. While no procedure has yet been agreed upon for determining the tenth-percentile in-place strength based on the results of in-place tests, proponents of in-place testing have developed and are using statistically based interpretations. Three approaches suggested to date, while developed for pullout testing, are applicable in principle to all tests. The first two methods to be described are relatively simple to use, requiring only tabulated statistical factors and a handheld calculator. The third procedure is more complex, and for practical applications a personal computer is needed.

3.5.2 *Danish approach (Bickley 1982)*—In this approach, the pullout strengths obtained from the field tests are directly converted to equivalent compressive strengths by means of a relationship (correlation equation) that has been determined by regression analysis of previously generated data for the particular concrete being used at the construction site. The standard deviation of the converted data is then calculated. The tenth-percentile compressive strength of the concrete is obtained by subtracting the standard deviation times a factor (which varies with the number of tests made and the desired level of confidence) from the mean of the converted data. The factors used in this approach are one-sided tolerance factors (Natrella 1963), which are discussed in Section 3.5.3. The values for the factors K for different numbers of tests are given in Column 2 of

Table 3.3 — One-sided tolerance factors for tenth percentile level (Natrella 1963)

No. of tests	Confidence level		
	75 percent	90 percent	95 percent
Col. 1	Col. 2	Col. 3	Col. 4
3	2.501	4.258	6.158
4	2.134	3.187	4.163
5	1.961	2.742	3.407
6	1.860	2.494	3.006
7	1.791	2.333	2.755
8	1.740	2.219	2.582
9	1.702	2.133	2.454
10	1.671	2.065	2.355
11	1.646	2.012	2.275
12	1.624	1.966	2.210
13	1.606	1.928	2.155
14	1.591	1.895	2.108
15	1.577	1.866	2.068
16	1.566	1.842	2.032
17	1.554	1.820	2.001
18	1.544	1.800	1.974
19	1.536	1.781	1.949
20	1.528	1.765	1.926
21	1.520	1.750	1.905
22	1.514	1.736	1.887
23	1.508	1.724	1.869
24	1.502	1.712	1.853
25	1.496	1.702	1.838
30	1.475	1.657	1.778
35	1.458	1.623	1.732
40	1.445	1.598	1.697
45	1.435	1.577	1.669
50	1.426	1.560	1.646

Table 3.3 The following example illustrates the procedure. The example uses 10 test results but some other appropriate number may be used in larger placements.

Example

Individual equivalent compressive strength psi (MPa)*	Calculation
3990 (27.5) 3620 (25.0) 3550 (24.5) 3620 (25.0) 3260 (22.5) 3480 (24.0) 3700 (25.5) 4130 (28.5) 3620 (25.0) 4350 (30.0)	Mean $\bar{X}$ = 3730 psi (25.7 MPa) Standard deviation S_x = 330 psi (2.3 MPa) Factor K = 1.671† Tenth percentile strength = $\bar{X} - K\,S_x$ MPa = 3180 psi (21.9 MPa)

*Converted from pullout force measurements using correlation curve.
†The constants K to be used in the calculation of the tenth percentile are given in Column 2 of Table 3.3.

3.5.3 *Tolerance factor approach (Hindo and Bergstrom 1985)*—The acceptance criteria for strength of concrete cylinders in ACI 214 are based on the assumption that the probability of obtaining a test with strength less than f'_c is less than approximately 10 percent. A suggested approach for evaluating in-place tests of concrete at early ages is to determine the lower tenth percentile of strength, with a prescribed confidence level.

It has been established that the variation of cylinder compressive strength can be modeled by the normal or the lognormal distribution function depending upon the degree of quality control. In cases of excellent quality control, the distribution of compressive strength results is better modeled by the normal distribution; in cases of poor control, it is better modeled by a lognormal distribution (Hindo and Bergstrom 1985).

In the tolerance factor approach, the lower tenth-percentile strength is developed from in-place test results, by considering quality control, number of tests n, and the required confidence level p. Three quality control levels are considered: excellent, average, and poor, with the distribution function of strength assumed as normal, mixed normal-lognormal, and lognormal, respectively. Suggested values of p are 75 percent for ordinary structures, 90 percent for very important buildings, and 95 percent for crucial parts of nuclear power plants (Hindo and Bergstrom 1985). However, since safety during construction is the primary concern, a single value of p may be adequate for all structures. A value of p equal to 75 percent is widely used in practice.

The tolerance factor K, along with the sample average X, and standard deviation S_x, are used to establish a lower tolerance limit, which is the lower tenth percentile of strength. For a normal distribution function (excellent quality control), the estimate of the tenth-percentile strength $X_{.10}$ can be determined as follows

$$X_{.10} = \bar{X} - K\,S_x \tag{3-2}$$

where
$X_{.10}$ = lower tenth percentile of strength (10 percent defective)
$\bar{X}$ = sample average strength
K = one-sided tolerance factor
S_x = sample standard deviation

The tolerance factor is determined from statistical characteristics of the normal probability distribution and depends upon the number of tests n, the confidence level p, and the defect percentage. Values of K

may be found in reference books such as that by Natrella (1963). The values in Table 3.3 are one-sided tolerance factors for confidence levels of 75, 90, and 95 percent and a defective level of 10 percent.

For the lognormal distribution (poor quality control), the lower tenth percentile of strength can be calculated in the same manner but using the average and standard deviation of the logarithms of strengths in Eq. (3-2).

By dividing both sides of Eq. (3-2) by the average strength X, the following is obtained

$$\frac{X_{.10}}{\overline{X}} = 1 - K\,V_x \qquad (3\text{-}3)$$

where V_x = coefficient of variation (expressed as a decimal).

In Eq. (3-3), the tenth-percentile strength is expressed as a fraction of the average strength. Fig. 3.10 is a plot of Eq. (3-3) for p = 75 percent and for coefficients of variation of 5, 10, 15, and 20 percent. Basically, this figure shows that as the variability of the test results increases or as fewer tests are performed, a smaller fraction of the average strength has to be used for the tenth-percentile strength.

The tolerance factor approach is similar to the Danish approach. The results of the in-place tests are converted to equivalent compressive strengths using the correlation relationship, and the equivalent compressive strengths are used to compute the sample average and standard deviation.

The following example illustrates the application of the tolerance factor approach for probe-penetration tests. It is desired to know whether the in-place strength of concrete in a slab is sufficient for the application of post-tensioning. The compressive strength requirement for post-tensioning is 2900 psi (20 MPa). The numbers in the first column are the measured exposed lengths of each of eight probes, and the second column gives the corresponding compressive strengths based on the previously established correlation relationship for the concrete being evaluated.

Correlation relationship: X (psi) = 145 + 2540 L
X (MPa) = − 1 + 0.69 L

Exposed length L, (mm)	Compressive strength X, psi (MPa)
1.18 (30)	2850 (19.7)
1.38 (35)	3360 (23.2)
1.34 (34)	3260 (22.5)
1.38 (35)	3360 (23.2)
1.50 (38)	3660 (25.2)
1.42 (36)	3460 (23.9)
1.22 (31)	2950 (20.3)
1.18 (30)	2850 (19.7)
Average	3220 psi (22.2 MPa)
Standard deviation	300 psi (2.1 MPa)
Coefficient of variation	0.093

For eight tests and a confidence level of 75 percent, the tolerance factor is 1.74. It is assumed that the normal distribution describes the variation of concrete strength. Thus, by substituting the coefficient of variation and the tolerance factor into Eq. (3-3), the ratio of the tenth-percentile strength to the average strength is found to be 0.835. Therefore, the tenth-percentile in-place strength is 2690 psi (18.5 MPa). Since the tenth-percentile strength is greater than 0.85 × 2900 psi (20 MPa) = 2465 psi (17 MPa), post-tensioning may be applied.

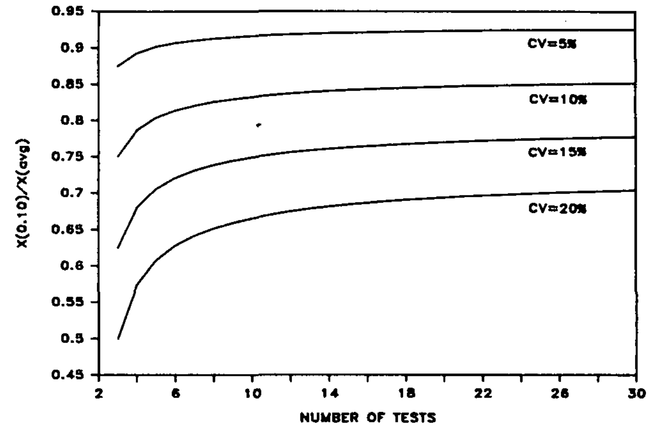

Fig. 3.10—Ratio of tenth-percentile strength to average strength as a function of coefficient of variation and number of tests (normal distribution assumed)

3.5.4 *Rigorous approach (Stone and Reeve 1986)*—The preceding approaches convert each in-place test result to an "equivalent" compressive strength value by means of the correlation relationship. The average and standard deviation of the equivalent compressive strength are used to compute the tenth-percentile in-place strength. Two major objections have been raised to these approaches (Stone, Carino, and Reeve 1986; Stone and Reeve 1986): 1) the correlation relationship is presumed to have no error, and 2) the variability of the compressive strength in the structure is assumed to be equal to the variability of the in-place test results. The first factor will tend to make the estimates of in-place tenth-percentile strength unconservative, while the second factor will tend to make the estimates overly conservative.

The National Bureau of Standards (NBS) has developed a comprehensive technique for the statistical analysis of in-place test results (Stone and Reeve 1986). This rigorous method encompasses three procedures:

1. Regression analysis to determine the correlation relationship.

2. Estimation of the variability of the in-place compressive strength.

3. Computation of the in-place tenth-percentile strength.

The correlation relationship is obtained by a regression analysis procedure that accounts for the fact that the X-variable (pullout strength) has measurement error. This contrasts with routine linear regression, which is based on the assumption that there is no measurement error for the X-variable. Because the within-test variabilities of cylinder strength and pullout strength are best described in terms of constant coefficients of variation, linear regression analysis is performed using the logarithms of the test values obtained during correlation testing. The resulting correlation relationship is a power function fit to the untransformed experimental data. The estimation errors associated with the regression coefficients are computed and used later to calculate the in-place tenth-percentile strength.

In Section 3.2, it was shown that the within-test variability of in-place test results is generally greater than compression-test results. This is why objections have been raised against assuming that the variability of the in-place compressive strength equals the variability of the in-place test results. In the rigorous approach, it is assumed that the variability of compressive strength divided by the variability of the in-place test results is a constant. Thus, the ratio obtained during correlation testing is assumed to be valid for the tests conducted in the field. This provides a means for estimating the variability of the in-place compressive strength based on the results of the in-place tests.

The in-place tenth-percentile strength computed by the rigorous procedure accounts for the error associated with the correlation relationship. The user can determine the tenth-percentile strength at any desired confidence level for a particular group of field test results. In addition, the user can choose the percentile to be a value other than the tenth percentile.

The NBS researchers compared tenth-percentile strengths computed by the three approaches that have been discussed (Stone, Carino, and Reeve 1986). It was found that the Danish and the tolerance-factor approaches give values lower than the rigorous approach. The differences were as high as 40 percent when the in-place tests results had high variability (coefficient of variation = 20 percent). Thus, it was concluded that compared with the rigorous method, the tolerance-factor and Danish approaches tend to be more conservative and do not produce a consistent confidence level. Studies are needed to compare the tenth-percentile

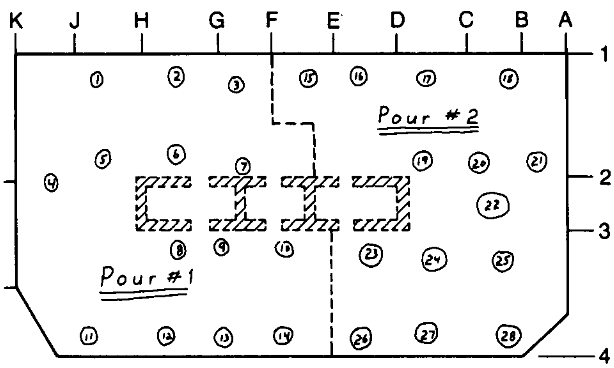

Fig. 3.11—Sample form for identifying locations of in-place tests in a floor slab of a multistory building

TESTING COMPANY

FIELD RECORD OF IN-PLACE TESTING

TEST NUMBER	TEST RESULT	EQUIVALENT COMPRESSIVE STRENGTH
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		

CALCULATIONS

Number of Tests: ________
Equivalent Strength:
Mean ________
Standard Deviation ________

K-Value ________

Minimum Strength:
Mean - (K*SD) ________

Project Number ________
Project Name ________

Location in Structure ________

PLACEMENT:
Date ________
Time ________
Size ________

Mix Number ________

IN-PLACE TESTING:
DATE ________
TIME ________

CURING CONDITIONS

Maturity: ________ C-Hours

Temperature at Test:
Ambient ________C
Within Enclosure ________C

Appearance of Top Surface:
____

REMARKS:
____

Technician ________
Checked by ________
Instrument Number ________

Fig. 3.12—Sample form for recording in-place test results

TESTING COMPANY LETTERHEAD

Project No: ________
Client: ________
Address: ________

Report Number ______
(in sequence)

ATTENTION: ________

Testing of In-Place Strength
Project Name ________
Address ________

Dear Sir:
The following are the results of in-place tests of 4000 psi concrete at the above site.

Location in Structure: ________

	Date	Time
Pour:		
Test:		
Proposed Time of Form Removal		

TEST RESULTS SUMMARY

Number of Tests Made:	
Mean In-Place Strength (psi):	
Standard Deviation (psi):	
Minimum In-Place Strength (psi):	

Individual Test Results (psi)

Remarks:
Requirements of ____ psi mean and ____ psi minimum strength prior to stripping and reshoring are/are not met by the above results

Copy given to Site Superintendant

Date: ________
Time: ________
Signed ________
(for Testing Co.)
____
(for Contractor)

Yours very truly,

Signed ________
(Testing Co. Engineer)

WHITE: Testing Company / YELLOW: Contractor / PINK: Structural Engineer

Fig. 3.13—Sample form for reporting in-place test results

strength predicted by these different approaches with the value obtained from a large number of core tests. Only then can the reliability of these approaches be evaluated.

The rigorous method is best suited for implementation on a personal computer. For example, an interactive computer program has been developed* that permits the user to develop the correlation relationship for each particular construction site and then use the relationship to estimate the in-place strength (for any desired confidence level) based on the field test results. The program prompts the user at each step of the analysis and provides guidance in interpreting the test results. In addition, a simplified procedure has been developed that gives results very similar to the rigorous method. The simplified method has been implemented on a personal computer using commercial "spreadsheet" software (Carino and Stone 1987).

3.5.5 *Reporting results*—For the different tests and for different purposes, a variety of report forms will be appropriate. In most cases, relevant ASTM standards provide guidance as to the information required on a report. Where in-place testing is made at early ages, some particular reporting data are desirable. A set of forms similar to those developed for use in pullout testing is shown in Fig. 3.11 to 3.13. These may serve as useful models for developing forms for reporting the results of other types of in-place tests.

Briefly, the three forms provide for the following:

1. Record of test locations (Fig. 3.11)—This form gives a plan view of a typical floor in a multistory building. The location of each test is noted. Where maturity meters are installed, their location would also be shown. Location data is considered important in the event of low or variable results. Where tests are made at very early ages and the time to complete a placement is relatively long, there may be a significant age-strength variation from the start to the finish of the placement.

2. Record of field-test results (Fig. 3.12)—This is the form on which test data, the calculated results, and other pertinent data are recorded at the site. The form has been designed for evaluating the data with the Danish or tolerance-factor approaches (minimum strength is the tenth-percentile strength). It includes provisions for entering information on maturity data, protection details, and concrete appearance used to corroborate the test data in cold weather. Due to the critical nature of formwork removal, a recommended procedure is for the field technician to phone the data to a control office and obtain confirmation of the calculations before giving the results to the contractor.

3. Report of test results (Fig. 3.13)—This is the report form for the in-place tests. It is a printed multicolor self-carbon form designed to be completed on site by the technician, with copies given on site to both the contractor's and structural engineer's representatives as soon as the results have been checked. It provides for identification of the placement involved, the individual results, and the calculated mean and minimum strengths. It records the engineer's requirements for form removal and states whether or not these requirements have been met. It requires the contractor's representative's signature on the testing company's copy.

*Stone, W. C., personal communication.

CHAPTER 4—REFERENCES

4.1—Recommended references

The documents of the various standards-producing organizations referenced in this report are listed below with their serial designation, including year of adoption or revision. The documents listed were the latest effort at the time this report was revised. Since some of these documents are revised frequently, generally in minor detail only, the user of this report should check directly with the sponsoring group if it is desired to refer to the latest revision.

American Concrete Institute

214-77 (Reapproved 1983)	Recommended Practice for Evaluation of Strength Test Results of Concrete
306R-78 (Revised 1983)	Cold Weather Concreting
308-81	Standard Practice for Curing Concrete
318-83	Building Code Requirements for Reinforced Concrete

ASTM

C 31-87a	Standard Practice for Making and Curing Concrete Test Specimens in the Field
C 39-86	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 42-84a	Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 192-81	Standard Method of Making and Curing Concrete Test Specimens in the Laboratory
C 511-85	Standard Specification for Moist Cabinets, Moist Rooms, and Water Storage Tanks Used in the Testing of Hydraulic Cements and Concretes
C 597-83	Standard Test Method for Pulse Velocity Through Concrete
C 803-82	Standard Test Method for Penetration Resistance of Hardened Concrete

C 805-85	Standard Test Method for Rebound Number of Hardened Concrete
C 823-83	Standard Practice for Examination and Sampling of Hardened Concrete in Constructions
C 873-85	Standard Test Method for Compressive Strength of Concrete Cylinders Cast in Place in Cylindrical Molds
C 900-87	Standard Test Method for Pullout Strength of Hardened Concrete
C 1074-87	Standard Practice for Estimating Concrete Strength by the Maturity Method
E 105-58 (Reapproved 1975)	Recommended Practice for Probability Sampling of Materials
E 122-72 (Reapproved 1979)	Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process

These publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 19150
Detroit, MI 48219-0150

ASTM
1916 Race St.
Philadelphia, PA 19103

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