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16. Abstract


The concrete tests currently in widespread use were developed decades ago, and although there have been continual updates and refinements, there are inherent limitations which cannot be overlooked. Most important, current test methods are often rendered unrepresentative of the concrete in the field, especially at early ages or under different curing conditions. The study described herein was conducted to evaluate the applicability and effectiveness of several nondestructive test methods for assessing concrete strength in the field.

The test methods used in this study included compressive cylinders, flexural beams, penetration resistance, rebound hammer, pullout, maturity, ultrasonic pulse velocity, and drilled cores. Test results from the different methods being evaluated were compared at concrete ages ranging from 1 to 28 days. Each test method was also evaluated for within-test variability among sets of companion specimens.

Three different concrete mix designs were used, including cement plus fly ash contents ranging from 300 to 500 pounds per cubic yard, three maximum sizes of river gravels, and one maximum-size crushed limestone coarse aggregates. Specified design concrete strengths were 3,500 and 5,000 psi, resulting in measured 28-day cylinder compressive strengths ranging from 3,700 to 8,700 psi. Full-size test slabs were cast and cured outdoors under simulated field conditions during the period from August through May. Test specimens were cured both under laboratory-controlled conditions and under field conditions adjacent to the slabs.

Of all the test methods studied, the maturity method exhibited the lowest variability and most consistent agreement with the generally-accepted standards for concrete testing, including compression cylinder and flexural beams for test ages after 1 day. The maturity method can also be applied to predict the strength of concrete at 1 day; however, special considerations and curve-fitting techniques can lead to the development of an accurate strength versus maturity relationship.


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# EARLY AGE STRENGTH OF CONCRETE: A COMPARISON OF SEVERAL NONDESTRUCTIVE TEST METHODS

by

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**Research Report Number 1198-1F**

Research Project 3-9-89/0-1198

Concrete Strength Determination at Early Ages in the Field

conducted for

**Texas State Department of Highways and Public Transportation**

in cooperation with the

**U. S. Department of Transportation Federal Highway Administration**

by the

**CENTER FOR TRANSPORTATION RESEARCH**

Bureau of Engineering Research

THE UNIVERSITY OF TEXAS AT AUSTIN

January 1991

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There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

# PREFACE

This report summarizes a detailed study evaluating the currently-available concrete testing practices which may aid in evaluating and estimating the strength of concrete in the field, particularly at early ages before the standard 28-day cylinder strength or 7-day beam strength is available. Both laboratory and field-cured specimens cast during hot and cold weather conditions were given special consideration.

This research study, Project 3-8-89/0-1198, "Concrete Strength Determination at Early Ages in the Field," was conducted at the Construction Materials Research Group Laboratory as part of the overall research program of the Center for Transportation Research, Bureau of Engineering Research, of The University of Texas at Austin. The work was sponsored jointly by the Texas State Department of Highways and Public Transportation and the Federal Highway Administration.

The overall study was directed jointly by Dr. Ramon L. Carrasquillo, Professor of Civil Engineering and Dr. David W. Fowler, Professor of Civil Engineering. For their invaluable assistance and contributions to the study, special thanks are extended to Mr. Mike Leary, Federal Administrator Coordinator; Mr. Billy Neeley of D-9; Mr. John Finley of D-6; and especially to Mr. Jerry Lankes, our technical coordinator from D-9.

# ABSTRACT

The concrete tests currently in widespread use were developed decades ago, and although there have been continual updates and refinements, there are inherent limitations which cannot be overlooked. Most important, current test methods are often rendered unrepresentative of the concrete in the field, especially at early ages or under different curing conditions. The study described herein was conducted to evaluate the applicability and effectiveness of several nondestructive test methods for assessing concrete strength in the field.

The test methods used in this study included compressive cylinders, flexural beams, penetration resistance, rebound hammer, pullout, maturity, ultrasonic pulse velocity, and drilled cores. Test results from the different methods being evaluated were compared at concrete ages ranging from 1 to 28 days. Each test method was also evaluated for within-test variability among sets of companion specimens.

Three different concrete mix designs were used, including cement plus fly ash contents ranging from 300 to 500 pounds per cubic yard, three maximum sizes of river gravels, and one maximum-size crushed limestone coarse aggregates. Specified design concrete strengths were 3,500 and 5,000 psi, resulting in measured 28-day cylinder compressive strengths ranging from 3,700 to 8,700 psi. Full-size test slabs were cast and cured outdoors under simulated field conditions during the period from August through May. Test specimens were cured both under laboratory-controlled conditions and under field conditions adjacent to the slabs.

Of all the test methods studied, the maturity method exhibited the lowest variability and most consistent agreement with the generally-accepted standards for concrete testing, including compression cylinder and flexural beams for test ages after 1 day. The maturity method can also be applied to predict the strength of concrete at 1 day; however, special considerations and curve-fitting techniques can lead to the development of an accurate strength versus maturity relationship.

# SUMMARY

The primary goal of this study was to evaluate the currently-available concrete testing practices which may aid in evaluating and estimating the strength of concrete in the field, particularly at early ages before the standard 28-day cylinder strength or the 7-day beam strength is available. Special consideration was given to including both laboratory and field-cured specimens during hot and cold weather conditions.

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# IMPLEMENTATION STATEMENT

This report summarizes an experimental study aimed at developing sufficient information to allow highway engineers to evaluate the potential for using different nondestructive testing techniques for concrete strength determination in concrete highway construction.

Recommendations are made regarding the applicability and variability of the different testing methods considered. Implementation of the findings from this study will result in improved assessment of the quality of concrete in the field, especially at early ages. This allows for construction to proceed on an accelerated schedule and ensures that the quality of the concrete is adequately monitored.

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# CHAPTER 1. INTRODUCTION

Concrete technology and concrete construction practices have made great advances in the twentieth century, but concrete testing methods have failed to keep pace with new demands. The concrete tests currently in widespread use were developed decades ago, and, although there have been continual updates and refinements, there are inherent limitations which cannot be overlooked. Most important, current test methods are often unrepresentative of the concrete in the field, especially at early ages or under different curing conditions.

## 1.1 RESEARCH SIGNIFICANCE

For both technological and economic reasons, the construction industry is faced with the need to update testing practices. Researchers and equipment manufacturers have responded with the development of a variety of new test procedures and techniques. Information on these test procedures is often lacking, incomplete, and/or conflicting. Proponents assert that these new procedures are either more convenient or more economical or that they give a better estimation of actual in-place concrete strength. If the assertions are proved to be correct, practicing engineers, contractors, owners, and the public all stand to benefit by receiving better-quality strength data at a lower cost. And most important, these data could be available at early ages, thus allowing for accelerated construction schedules.

## 1.2 OBJECTIVE OF THE RESEARCH

The primary goal of this study was to identify the currently-available concrete testing practices which may aid in evaluating the strength of concrete, particularly at early ages before the 28-day cylinder strength or the 7-day beam strength is available. Special consideration was given to including both laboratory and field-cured specimens during hot and cold weather conditions.

## 1.3 RESEARCH PLAN

Eight nondestructive tests were selected for evaluation. Different mix designs, typical of those currently used in highway construction in Texas, were used. Quality-control specimens and full-size slabs were cast under actual field conditions. Tests were conducted at various test ages up to 28 days. Testing was conducted on field-cured slabs and quality-control specimens, and the results were compared directly with those obtained from standard laboratory-cured cylinder and beam tests. Results were also analyzed for consistency within groups of companion-test specimens for each test condition.

## 1.4 FORMAT

This report is divided into six chapters. A review of available literature was conducted and is presented in Chapter 2. The test procedures and materials used in the study are described in Chapter 3. The test results are presented in Chapter 4 and discussed in Chapter 5. In Chapter 6, a summary and several conclusions are presented. Test data and details of calculations involved in the study are included in the appendices.

# CHAPTER 2. LITERATURE REVIEW

## 2.1 INTRODUCTION

Concrete evaluation has traditionally been based on compressive cylinders and, in some cases, on flexural beams. However, in recent years, there have been efforts to change that practice, as well as increasing demands for new approaches to acceptance testing on construction sites. Contractors—facing reduced budgets, increasing costs, and tighter schedules—have pushed the construction industry to a point where 28 days for cylinder strength or 7 days for beam strength is far too long to wait for data about the quality of concrete in place. As a result, the traditional tests simply take too much time.

This chapter is a review of several different types of concrete tests. Included are those that are currently in widest use, such as compressive cylinders, drilled cores, and flexural beams. Also included are some that have been proposed as replacements for or additions to the customary acceptance-testing routine, such as pullout, maturity, ultrasonic pulse velocity, penetration, and rebound number methods. For each variety of test, there is a discussion of how the test is done as specified by the American Society for Testing and Materials (ASTM). A history of the test and research associated with it, including the views of some who favor it and some who oppose it, are also presented. The last section of the chapter is a summary of the literature review.

## 2.2 CYLINDERS

### *2.2.1 SPECIFICATIONS*

Most concrete testing is currently done according to American Society for Testing and Materials Standard C172, Sampling Freshly Mixed Concrete[7], Standard C31, Practice for Making and Curing Concrete Test Specimens in the Field[7], and Standard C39, Compressive Strength of Cylindrical Concrete Specimens[7].

Following these standards, a representative sample of concrete is taken from the supply of concrete as it is being placed and a number of 6-inch-by-12-inch cylinders are made at the construction site at the time of casting. Each cylinder is made by filling a cardboard, plastic, or steel mold with concrete and consolidating it. Each filled mold is then floated so the top has a relatively smooth surface. A cover is placed over the top to prevent moisture loss or the entire cylinder is otherwise protected against drying. The cylinders are left at the construction site for 16 to 32 hours to begin hydrating at a temperature between 60 and 80°F. On the day after the concrete is cast, the cylinders are stripped of their molds and placed in a more carefully controlled environment. This may be a water bath or a moist-cure room, but in each case the temperature is kept at a constant 73°F (20°C) and the cylinders are not allowed to dry. Twenty-eight days later, the cylinders are removed from the controlled environment and their strength is determined by compression testing.

### *2.2.2 HISTORY*

The American Society for Testing Materials adopted Standard C31, Practice for Making and Curing Concrete Test Specimens in the Field, in 1920. Originally it called for cylinders to be cast at the construction site and stored there in wet sand or sawdust. Within a few years, Young[98] reported that the cylinder strength test was quite reproducible, having a "mean variation where the test specimens were made at one time... [of] six to eight percent." However, it also had its limitations, and there were varying views as to what its uses should be. In 1927, Young summarized his findings as follows:

> There are two distinctly divergent view points as to how concrete samples should be cured. There are those who believe that test specimens should be cured under job conditions, receiving the same treatment that is received by the concrete they represent. This view is predicated on the idea that, if the sample receives the same treatment as the structure, its strength will be identical with similar concrete therein. Tests made in this way are useful to indicate when it is safe to strip forms, remove shores, open a road to traffic or for some similar purpose. The weakness of this system of testing lies in the fact that a given set of curing conditions will affect the small specimens differently than the larger bodies of concrete it represents. The method, too, has a serious fault; the results obtained are partly related to the inherent quality of the concrete and partly to the subsequent treatment the specimen receives. In a given case it is always difficult to determine the degree to which each factor operates.
>
> The other view point held is that the specimens should receive a standard treatment, such as is provided in the Standard Methods of the Society. The theory underlying this attitude is that specimens cured under carefully controlled conditions are an index of the inherent quality of the concrete—the optimum quality, that with proper treatment, the concrete will possess. The advantage in this system of curing is that the tests are comparable with similar tests elsewhere or to laboratory tests made by similar methods. It agrees with the established principles of testing, that the tests on a material should disclose the properties of that material and that supplementary tests or subsequent inspection should be depended upon to see that the material is not abused while it is being fabricated.

Cautions and calls for different methods have continued through the years since then. In 1930, Edwards[30] discussed the relation of structural concrete to laboratory-cured cylinders:

> In 1929 work, tests were designed to determine the relation of structural concrete to the standard laboratory-cured cylinders, but cool air temperatures made test results of concrete cured on the job so variable that further efforts were deferred until a recording thermometer became available to record the air and concrete temperatures during the curing period.

Research continued into various other methods of testing concrete. In 1938, Skramtajew[85] noted:

> There are two possible ways of insuring proper control of the strength of concrete in a structure. Both are based on some form of test to determine the actual strength being produced: (1) Preparing specimens simultaneously with the placing of concrete in structures and testing these specimens. (2) Determining the concrete strength in the completed structure or in individual structural members.
> Generally only the first method is being used, but a successive use of both methods is necessary. The determination of concrete strength by the use of ordinary specimens is not sufficient....

Skramtajew went on to discuss fourteen different proposed tests from around the world, including the ancestor of the pullout test, for determining the concrete strength in a completed structure.

Researchers in the 1960's tried to correlate the strength indicated by carefully cured cylinders with their best estimate of the strength of concrete in the structure, and their conclusions parallel what Young, Edwards, and Skramtajew had put forth decades before. A paper by Bloem[12] summarized the results of several studies in the United States and Sweden. In Bloem's tests and those he cited, the strength of the concrete in the structure was determined by core-drilling and by cast-in-place cylinders (push-out cylinders) which remained in the structure until it was time for compression testing, and this was compared to the laboratory-cured cylinders. He concluded that these standard tests do not provide a quantitative measure of the load-carrying capacity of the concrete in a structure. He brought out a suggestion somewhat similar to that of Skramtajew thirty years earlier:

> We should at least consider the feasibility of separating our concepts of strength into two distinct categories:
>
> 1. *Design* strength for development of structural sections and calculation of load-carrying capacity.
> 2. *Control* strength as a measure of proper quality and uniformity of the concrete used in the work.
>
> The design strength should correspond, at least in a relative sense, to the level which will be attained in the particular structure under the actual conditions of construction. The control strength should be selected for the specific situation and materials to assure attainment of the design strength in the structure. Inspection should aim at enforcing the control strength proportions and seeing that construction practices are such as to assure development of the design strength.

In the early 1970's two presidents of the American Concrete Institute again brought up the issue of cylinder testing. In 1972, Cohen[26] stated, "The method of strength control... needs drastic improvement." He went on to say that, "Although the standard cylinder is useful for selecting mix proportions, it is a poor procedure for quality control." Two years later, Philleo[71] echoed Cohen's sentiments:

> I am concerned with another problem which arises not infrequently—the determination of the in-place quality of concrete. It arises because no cylinders are available when the information is needed or because the environmental exposure of the test specimen and the structure have been so different that information from the specimen is obviously not applicable to the structure.

He listed nine proposed methods, then noted:

> The obvious objection to them is that they measure something other than compressive strength. But they measure something related to the hydration of cement since the results change as the cement hydrates. If we could shed our compressive-strength hang-up and admit that there are other ways to evaluate concrete maturity, we might be more tolerant of a less-than-perfect correlation with compressive strength.

Yener and Chen[97] later noted that each of the other methods that are available "measures different characteristics of the in-place concrete." They went on to say that "a direct comparison [with cylinders] is usually not possible."

A few years later, Philleo again published a paper suggesting it was time to consider something other than 28-day cylinder breaks for acceptance of concrete.[70] He recounted in general terms that researchers have measured the stress distribution of structures at failure and compared it to the failure stress of companion cylinders and they found that the two do not coincide. At the same time, as the practice of engineering has developed, it has become possible to define the performance of a structural element cast and cured under field conditions in terms of the strength of a 6-inch-by-12-inch cylinder broken after storage at 100 percent humidity at a temperature of 73°F. Philleo emphasized that, historically, empirical factors have been used to relate the failure stress in a structure to the failure stress of a cylinder, and on this foundation the rectangular stress block was developed for concrete design. If the cylinder curing requirements were changed, the theory would still be the same, "... only the constants would be different."[70] Based on this, he proposed basing evaluations of concrete on the accelerated strength as determined by ASTM C684[7], which provides results in 3.5 hours, 24 hours, or 48 hours, depending on the test. He suggested this could be done by converting the accelerated strengths to equivalent 28-day strengths; or, in a similar but more revolutionary-sounding move, simply redimensioning the stress block in terms of accelerated strengths.

Bloem[13] and Dilly and Ledbetter[28] reported that strength of concrete may vary considerably, depending on its location within a structure. Both reports said that concrete strength at the top of a column may be only 60 percent of the strength at the bottom of the column. Sabnis and Mirza[80] mentioned previous research which had found:

> change in quality of cast material due to water gain in the top layers and water leakage of the forms. Differences of up to 10 percent in strength were noted between vertically cast and horizontally cast specimens that were otherwise identical.

They went on to assert that some skill is required to select a correct scale for a model study and the proper size of cylinder to evaluate concrete compression strength in model specimens. They suggested it is entirely possible that some of the empirical data currently in use are based on incorrectly sized specimens.

Mirza et al[57] noted that, from batch to batch, a cylinder strength coefficient of variation of 7 to 10 percent is about as good as one will get, and 15 to 20 percent is more likely. Mirza proposed a random variable relating in-situ strength to real cylinder strength.

Ramakrishnan wrote in 1976 that the idea that cylinders represent the in-situ concrete is incorrect and it prevents the development of more realistic and economic concepts:

> Acceptance of the tests on comparison test specimens as representative of the concrete used inhibits the realistic design for in-place concrete. The lack of knowledge of the actual strength of concrete in a structure requires the use of a larger factor of safety than would otherwise be necessary. Further, when a structure is rated or evaluated for its load-carrying capacity at a later date, the representative test cylinders will not be available for estimating the strength of concrete; therefore, inplace testing becomes necessary.

Regarding the factor of safety, there can be problems associated with excess strength caused by a desire to make sure a minimum is always achieved. Tuthill[95] stated:

> Elevated strengths usually mean higher modulus of elasticity and creep values which reduce strain capacity and thus resistance to cracking.

More and more engineers are calling for a "new" way of assessing concrete quality, frequently agreeing with the views Skramtajew[85] voiced in 1938, mentioned previously. For instance, Bartos[8] noted that ACI Committee 306 recommended use of in-situ testing to indicate when shores may be pulled. In the late 1970's, he made an informal survey of researchers and other concerned engineers and concluded that many agree on several points:

(1) there formerly were no reliable in-situ test techniques, but now there are;
(2) use of in-situ tests is increasing, particularly among contractors and precasters; and
(3) it is time to standardize in-situ testing and make it a requirement, rather than an option.

In another example of negative opinions about reliance on cylinders, Yener and Chen[97] wrote in 1984 that

> presently adopted procedures have been successfully used by the concrete construction industry for many years. However, in view of the latest technological developments in this field, these procedures actually hinder the efficient construction of concrete structures. Economic considerations make early post-tensioning, early form removal, and quick termination of winter heating essential.

On the other hand, Bungey[16] pointed out in 1982 that there are hazards associated with in-situ testing. Conceding that there is a trend toward in-situ compliance testing in North America, and this type of testing does offer the benefit of timely assessment of strength, he still maintains:

> Difficulties in obtaining an accurate quantitative estimate of in-situ concrete strength can be considerable; wherever possible the aim of testing should be to compare suspect concrete with similar concrete in other parts of the structure which is known to be satisfactory, or of proven strength.[16]

Investigations into the exact nature of the fracture of cylinders during testing have been carried on in recent years, in the wake of Bloem's studies in the 1960's and Philleo's suggestions in the 1970's. In 1984, Kotsovos and Cheong[48] of Imperial College in London experimented to assess Kotsovos's previous theoretical efforts in this regard. They found:

> experimental information is in compliance with results of analytical work that indicated that concrete

> structural forms under increasing load collapse before the strength of concrete in compression [as assessed using cylinders] is exceeded anywhere within the structure.[48]

In other words, the analysis and experiments of Kotsovos and Cheong showed by detailed investigation what people like Young, Edwards, Skramtajew, Bloem, Cohen, and Philleo had been approaching empirically and intuitively for decades: a concrete structure and its cylinders behave differently. Kotsovos and Cheong demonstrated that the mechanics of the two situations are different and this makes it impossible for cylinders to precisely represent structures, because the cylinders are inherently stronger. There were additional effects of curing conditions and the variations in strength development due to structure size or shape, but these simply added to, or perhaps compensated for, the differences between cylindrical specimens and the structures they supposedly represent. The previous researchers had suspected and suggested this, but now the underlying theory was developed. That theory indicated the two should indeed behave quite differently.

It was thus shown that specimens (cylinders) cannot give an exact indication of what is happening in a structure. As Philleo suggested in the 1970's, the specimens and the structure have always been related empirically, and Kotsovos and Cheong determined that it is the only way they can be related.

Yet in 1980, as in-situ testing methods were developed and refined, many researchers still promoted the cylinder test. In the summary of a discussion of in-situ tests, Yener and Chen said:

> With these test methods standardized, a sufficient amount of compatible laboratory and field data may be gathered in order to develop simple statistical relationships between in-place test results and concrete strength. Such standardized test procedures would ensure adequate safety in the completed structure even when faults affecting strength occur during construction. For this reason, some experts believe that a viable in-place test procedure may eliminate entirely the necessity of conducting the presently adopted cylinder and core tests. However, the present authors contend that both cylinder tests, to evaluate the characteristics of the concrete mix, and in-place tests, to determine the strength in the structure, should be conducted in order to avoid substandard quality concrete in structures.[97]

Parallel to the continuing debate over what a cylinder represents and how it should be used, there has also been another debate surrounding cylinder manufacture and handling. This debate centers on how cylinders are made and whether the test measures just the quality of the concrete or whether it also measures such extraneous factors as the skill and care with which the cylinder is made. In 1951, the editors of the ACI Journal reported that their most recent annual convention had discussed the question, "Is the compression test for determining quality of concrete obsolete?" They then introduced a reprint of a paper published in Australia by L.B. Mercer and entitled "Concrete Strength Variations—60 Contributory Causes."[55] The list was not substantiated by experimental data, but 25 of the causes related directly to test specimen variables, including such items as the choice of cubical or cylindrical molds, incorrect tamping of the specimen during casting, allowing the specimen to dry out, and failing to center the specimen in the testing machine.

By the time of that report, effects of curing had been thoroughly investigated. Some difficulties which were identified at that time seem to be surviving into the 1990's. In 1951, Price[73] reported that his investigation confirmed that of the Bureau of Reclamation in which field cylinders were found to make lower strengths during the hot summer months than during cooler months. Price said that it seemed the concrete was weakened by rapid setting at high temperatures in its first day, and the weakening could not be overcome in the subsequent 27 days of curing at 73°F. ASTM C31 thus requires cylinders to be stored at a temperature of 60 to 80°F for 24 hours (plus or minus 8 hours), at which time they may be moved to the curing facilities, but these temperature and time restrictions are sometimes neglected on construction sites.

Occasionally, field-curing of cylinders is proposed as an alternative to laboratory curing. By curing the cylinders in, as nearly as possible, the same way that the structure is cured, it is supposed that a more representative value is obtained for the strength of concrete. Bloem[12] investigated this and concluded that "field-cured cylinders may be misleading in that they are less adversely affected by improper curing than the structure itself." Richards[77] noted that field-cured cylinders averaged 57 percent of the potential strength in an experiment by the North Carolina Highway Commission. Yet, more than twenty years after Bloem, field-cured cylinders survive as a seemingly-reasonable option that is occasionally used in practice. Section 9.3 of ASTM C31 gives details of how to store field-cured cylinders, and the report of ACI Committee 306 on Cold Weather Concreting[3] mentions the use of field-cured cylinders in Chapter 6, "Protection for Structural Concrete Requiring Construction Supports."

In the 1980's, Shilstone[83], in agreement with some of the conclusions of Mercer's 1951 paper, asserted that:

> 1. When elongated or flat coarse aggregate is used in a mix, the ASTM specified method of casting cylinders can result in specimens which are not representative of the in-place concrete and can

cause test results to be less than the actual in-place concrete strength.

2. In the majority of cylinders cast, the three lifts made during the casting are not comparable as a result of technician-caused variations. The test identifies the *lowest* strength potential of concrete rather than the true potential of the concrete when properly placed, compacted, and cured.

Shilstone did not indicate that these conclusions were the result of a systematic study, and he offered no data to support his conclusions. He simply quoted Mercer's unsubstantiated estimate of a 40 percent variation in strength due to forcing the elongated or flat particles into a vertical orientation.

A paper by Popovics[74] updated, expanded, and quantified Edwards's and Bloem's ideas on the curing method. Popovics determined that relatively small changes in the overall moisture content of a cylinder can make dramatic differences in its strength, and his study indicated that it is the condition of the surface which is most important. Indeed, he found that by leaving cylinders to cure in dry air for three days before testing them he could show an increase in strength of 25 percent over those that spent 27 days in a moist curing room.

Questions have arisen regarding the preparation of the ends of a cylinder for testing. ASTM C39[7] and C617[7] require that the ends of test specimens be plane within 0.002 inches; otherwise, they must be ground, sawed, or capped with sulfur. Sulfur capping has become common, but it must be done by a skilled technician and can be hazardous. It requires liquid sulfur, which melts at about 300°F and gives off noxious gases. Recently, however, elastomeric pads have been developed to replace the sulfur. Preliminary studies by Carrasquillo and Carrasquillo[21] have shown that certain elastomeric pad systems give mean values quite similar to sulfur caps, and they give within-test results having greater uniformity than those of sulfur caps. They are limited, however, to strengths below 10,000 or 11,000 psi. Published discussion of the Carrasquillo paper showed considerable interest in the matter, with similar findings reportedly made by the Victoria (Australia) Road Construction Authority, the Virginia Highway and Transportation Research Council, Caltrans, and the U. S. Army Corps of Engineers.[21]

In the 1950's, research by Burmeister[14] indicated that the type of mold used in making cylinders can make a significant difference in both the mean compressive strength and the standard deviation of the compressive strength tests. At that time, tests compared 6-inch-by-12-inch paper molds with steel molds.

More recently, a 1986 study by Carrasquillo and Carrasquillo[22] showed that plastic molds produce slightly lower strengths than steel molds (an average of 3 percent) for strengths in the range of 7,000 to 13,000 psi. Also in the 1980's, several groups of researchers investigated the use of cylinders of other sizes besides the standard 6-inch-by-12-inch. Using steel molds, some researchers[68] found that 4-inch-by-8-inch cylinders show about 110 percent of the 6-inch-by-12-inch cylinder strength in the range of 7,000 to 11,000 psi, while others[22] found that 4-inch-by-8-inch cylinders averaged 93 percent of 6-inch-by-12-inch cylinders in the range of 7,000 to 12,000 psi.

A third group suggested that 3-inch-by-6-inch cylinders should become the standard.[61] They found a 3 percent higher strength from using the smaller cylinders, and coefficients of variation of 6.5 to 8.5 percent compared to 3.1 to 3.7 percent for the larger cylinders on tests done on concrete with strength ranging from 2,600 to 4,200 psi. In the discussion of the paper, an Australian wrote that their standards committee had concluded ten years earlier that the size of the aggregate has a great deal of influence on the strength of concrete in smaller molds. The Australians were not surprised that the paper reported the smaller molds unsuitable for aggregate sizes over 3/4 inch.

In the late 1950's and early 1960's came the introduction of statistical methods for the evaluation of concrete tests. ACI Standard 214[1] was apparently somewhat confusing to many practicing engineers at first and it became the subject of a symposium in 1971 to explain the standard and its applications. The random nature of concrete tests continues to cause concern among engineers, but there has apparently been some improvement in cylinder testing procedures over the years. As cited previously, Young's 1927 paper[98] reported variations in cylinder strength of 6 to 8 percent, but ACI Standard 214 now says that the within-test coefficient of variation (standard deviation divided by the mean) for field control testing of cylinders is considered poor if it is above 6.0 percent. If it is below 3.0 percent, it is considered excellent. ACI 214 explicitly states that these values are not applicable to other strength tests. Nonetheless, in the minds of many engineers, this level of consistency has become a standard by which other testing methods are judged.

However, it has been pointed out that the ASTM standard for compressive strength (ASTM C39, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens[7]) is silent on certain issues of variability. In 1990, a Canadian study[34] was published, describing the results of experiments conducted between 1981 and 1987 which focused on the within-laboratory variability and the laboratory-to-laboratory variability of cylinder tests. The paper proposes a precision statement for the ASTM standard, noting that the average variability is directly proportional to the average compressive strength of the test concrete. With a 95 percent certainty, their data indicate that the variability approaches 10 percent within one laboratory and 15 percent between laboratories. Or, in the words of the study:

> the difference, in absolute value, of two test results obtained in different laboratories on the same material will be expected to exceed approximately 15 percent of the average compressive strength of the test concrete about five percent of the time, and there is reason to question one or both of the test results only if such a difference is found to be larger than this value.

Recently, contractors, governmental agencies, and others have begun to endorse certification programs for technicians involved in making and testing concrete cylinders.[27] An engineer conducting tests before and after a technician-certification class reportedly found that improper preparation and handling of specimens accounted for 300 to 500 psi variations in test results on a 4,000 psi mix.[79]

### *2.2.3 SUMMARY OF HISTORY*

It is apparent there has always been some controversy about the philosophy behind the methods used to evaluate the strength of concrete. Fairly soon after the standard for cylinders was adopted, engineers began dividing into factions, one believing that cylinders were useful only as an index of the quality of the concrete and the other desiring to use cylinders in, for example, deciding when shores could be removed. Then it was proposed that cylinders should be cast for concrete quality control and some other indicator should be used to determine concrete strength in place. This came about because engineers knew intuitively or realized empirically, and later showed analytically, that cylinders cannot be expected to represent conditions inside a structure. There are too many variables involved, including curing temperature, curing humidity, compaction of concrete in the structure, and size and shape of structure.

Beyond the philosophy involved, engineers have continually raised questions about cylinder testing procedures. As a result, detailed requirements have been developed for sampling concrete, making cylinders and storing them overnight before sending them to a controlled environment for curing, capping and testing them, and finally reporting the results. These procedures dictate conditions which will allow fair comparisons. However, the procedures are sometimes complicated, with many opportunities to seriously undermine the strength of the cylinder and no way to identify later whether the problem developed on the construction site or in the testing laboratory. The tests require skilled technicians and elaborate curing and testing facilities, all of which are expensive, particularly if concrete is placed at the end of a work week and curing must begin outside of normal working hours.

The procedures are time-consuming and sometimes hazardous, and there can be a significant delay between the time a test is needed and the time it can be performed and reported. Sometimes additional tests are needed, and they cannot be performed because no difficulties were anticipated and no extra cylinders were cast at the time of placement. Test cylinders are vulnerable to damage in their first day as they sit on the construction site or as they are transported to curing facilities. Strengths of cylinders can be affected by the type of mold used, the care in handling, and a host of other details. Nonetheless, they are still thought of as representative of the strength of concrete in a structure, even though research has shown they have no exact correlation with this strength.

In fact, research now indicates that they indicate higher-than-actual strengths compared to what is in the structures they supposedly represent. All in all, one ACI president characterized them as a "poor procedure for quality control,"[26] yet there is no other procedure so widely accepted.

Cylinders are used to assess the variability of the output of concrete batch plants. They are frequently suspected and accused of causing suppliers to have to add extra cement at considerable expense when it is not necessary and may prove harmful, as Tuthill stated[95] and was mentioned above.

On the other hand, cylinders are a highly repeatable and consistent standard. They are a known standard, and engineers and technicians throughout the country are at least aware of the requirements, even if not all know or obey all the rules. The constants used in developing current ultimate strength design equations, which presume to know the strength of concrete in a structure, are based on cylinder strengths, and throughout the country mix designs are determined by statistical analysis of concrete cylinder strengths.

## 2.3 BEAMS

### *2.3.1 SPECIFICATIONS*

Various sizes and shapes of flexural beams are allowed, but ASTM C31 requires a standard beam to be 6 inches wide by 6 inches deep with a length 2 inches more than three times the depth. This is suitable for aggregate sizes up to 2 inches maximum size. The standard beam is filled in two lifts, each lift being hand-rodded 60 times or else mechanically vibrated (either internally or externally). The filled and consolidated beam mold is struck off and floated smooth, and then protected from loss of water and evaporation. It must be stored at 60 to 80°F for 24 hours, plus or minus 8 hours. The beam may then be stripped of its mold. After it is demolded, it must be stored at 73.4°F, plus or minus 3.0°F, with free water on its surfaces at all times. A minimum of 20 hours prior to testing it must be stored in saturated limewater, and it must be kept wet between removal from the limewater and testing.

### *2.3.2 HISTORY*

Many issues discussed in Section 2.2 regarding cylinders also apply to beams. Ostensibly they are samples of concrete representing the material in-situ, as it is in a structure. However, these specimens may be useful only as an index of the quality of the concrete that was mixed and delivered to the construction site. The relationship of the strength of specimens to the strength of the concrete in a structure is questionable. The details involved in making and curing beams are similar to those required in making and curing cylinders, and concrete kept under a controlled curing regimen may not truly represent the concrete in an uncontrolled environment. The process of making flexural beams and keeping them is as costly and laborious as the process of making and keeping cylinders, and perhaps more difficult because one 6-inch-by-6-inch-by-20-inch specimen may weigh over 60 pounds, perhaps twice as much as a cylinder. As with cylinders, at times questions may remain about the strength of the structure. After all of the flexural beam specimens have been tested, the owner, builder, and engineer may still be left with uncertainty regarding the quality of the concrete.

As in the case of cylinders, the method is well known and repeatable. As with cylinders, various refinements of technique have been proposed. Still, the beam test follows cylinders as the target of criticism because the mechanics of the test specimen are generally not precisely the same as the mechanics of the structure it supposedly represents. In 1984, Yener and Chen[97] noted that the true tensile strength of concrete is difficult to measure, and that is why it is usually represented by other tests, including the modulus of rupture test, commonly known as the flexural beam test. They noted that the substitute representations are flawed because of the presence of stress gradients. These gradients cause the modulus of rupture tests to result in strength values larger than those which would be found in uniform axial tension.

## 2.4 PULLOUT

### *2.4.1 SPECIFICATION*

The pullout test method is summarized in ASTM C900[7] this way:

> 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.

The standard notes that 25 mm and 30 mm are common sizes, but there is no required size. The depth to the insert must be the same as the diameter of the insert. The shaft connecting the insert to the pulling jack cannot have a diameter greater than 60 percent of the insert itself. The bearing ring must have an inside diameter of 2 to 2.4 times the insert diameter, and the bearing ring outside diameter shall be at least 1.25 times its own inside diameter. The jack must be calibrated at least once a year, and loading shall be such that failure occurs in 120 seconds, plus or minus 30 seconds. Tests shall be spaced at least 10 insert diameters apart, they shall be at least 4 insert diameters from an edge, and they shall be clear of reinforcement by at least one bar diameter or one maximum-sized aggregate particle. Inserts can be at the top, side, or bottom surface of the concrete, but the standard points out that manually placed top-surface tests are more variable and of lower value than others.

Yener and Chen[97] describe the test this way:

> The pullout tests measure the force required to pull an embedded anchor plate out of the concrete. Because of the shape of the pullout assembly, a small cone of concrete is extracted. This pullout force is divided by the area of conic fracture to give the pullout strength.

There is a related test called the "cut and pullout" or CAPO test. In this test, no embedded anchor is required. Instead, a hole just over 1 inch deep is drilled into the hardened concrete and a disk-shaped cavity 1 inch in diameter is milled out at the 1-inch depth. A steel ring is placed in the hole and expanded to fill the cavity, so that the fully deployed apparatus closely resembles (in size, location, and orientation) the embedded anchor plate used in the pullout test. This test is not mentioned in the ASTM standard. However, the manufacturer sells it as a pullout test that simply requires no preplaced inserts.

### *2.4.2 HISTORY*

In 1938, Skramtajew[85] reported that I.V. Volf developed the first pullout test in the Union of Soviet Socialist Republics in the mid-1930's. Volf's ratio of pullout strength to cylinder strength was consistent within a range of plus or minus 9 percent but this was for low strength concretes (up to 1,500 psi). Volf embedded in the concrete a steel rod with a sphere at the end of it. The rod was embedded about 1-1/2 inches deep and was 3/8 inch in diameter, with the sphere 1/2 inch in diameter. He pulled the rod out with a dynamometer specially fitted to a frame with a bearing plate through which the rod was pulled. This gave a cone with an apex angle of about 90 degrees.

In 1944, Tremper[94] reported that early-age pullout testing worked well. For comparing compression tests to pullout strengths of up to 3000 psi, a linear equation worked well, while for strengths up to 3,500 psi a logarithmic curve fit best. Tremper reported that pullout tests "can be duplicated with nearly as great a degree of accuracy as the compression test." However, his coefficient of variation for cylinder tests was 8.4 percent, well above the modern standards listed in ACI 214; so his 9.6 percent coefficient of variation for pullout tests compared well. In addition, he noted that the values were erratic above 3,500 psi and concluded that pullout testing should not be used for higher strengths. Tremper's system used a 3/4-inch cylinder embedded 1-1/16 inch deep and pulled out by a hydraulic ram with a 6-inch-diameter bearing ring. This set up would give failure cones with an apex angle of about 130 degrees.

In the early 1960's in Denmark, development began on a simple pullout testing system which later evolved into the one marketed under the name Lok-Test. A 25-mm (1-inch) diameter steel disk is embedded 25 mm below the surface of the concrete, and a hand-held hydraulic jack with a bearing ring of 55-mm (2.2-inch) diameter is used to pull it out. This arrangement gives failure cones with an apex angle of about 62 degrees.

In the 1970's, after the president of the ACI called for renewed emphasis on other means of determining concrete quality, more researchers began considering the pullout test. In 1975, Malhotra[52] stated that the pullout test measures a combination of shear and tensile strengths of the concrete, and that it appeared the test was most applicable for concrete with compressive strengths below 5,000 psi. He listed the major disadvantages of the test as the damage to the concrete surface and the inability to test at depths more than 2 or 3 inches below the surface.

The method underwent extensive field testing, particularly in Denmark and Canada, in the 1970's. Yener and Chen[97] state that the relation between pullout force and cylinder strength was always found to be linear.

One Canadian field study[11] showed a within-test coefficient of variation for the pullout test ranging between 5 and 10 percent, while for the cylinder tests it ranged from about 2 to 4 percent, and the authors concluded that "the in-test variation of the pullout test is low and is of the same order as the standard cylinder test." They determined that six tests were enough for a statistically valid set. They did not try any fewer than that, and in some cases they used sets of ten tests. Also, they felt that the relationship between pullout force and compressive strength was something which should be determined for each concrete mix design used. Their report also concluded that if tests in the upper part of a slab indicate lower strength than tests in the bottom of a slab, this is because of real differences in the strength of the concrete at the test location.

In 1977, Richards[77] reported recent testing by the North Carolina Highway Commission in which the pullout apparatus was fastened to concrete forms. This apparatus was larger than the Danish apparatus, which Richards felt gave it lower variability. The report states that "relatively rough, impromptu procedures" were used with the somewhat crudely-fabricated American equipment, and the apex angle of the pullout cone was 67 percent. Yet the coefficient of variation was reported to be 2.4 percent, versus 5.5 percent for the standard cure cylinders. Richards quoted other researchers' findings of "precisions of 2.8 percent (46 pullouts) and 2.6 percent (30 cylinders)." He predicted that better fabrication of the apparatus would improve test results. He also stated that a modification of the pullout test assembly allowed placement at any desired location in the concrete surface at the time of casting, but said this form of the test appeared to be effected by top-to-bottom strength differences, such as were noted by Bloem[13] in earlier experiments on cored columns. He did not mention any coefficients of variation on this type of test, which later experimenters termed "finger-placed" pullouts. Richards' paper also mentioned tests by the National Ready-Mixed Concrete Association which had coefficients of variation averaging 12 percent and ranging as high as 18 percent.

Richards found a pullout-to-cylinder strength equation somewhat different from those developed in Denmark:

$$f'_c = 5.3\, f_p - 35$$

where $f'_c$ is the concrete cylinder strength and $f_p$ is the pullout strength. This equation had a correlation coefficient of 0.98. He noted that the Bureau of Reclamation had found a correlation coefficient of 0.97 for tests on concrete and 0.87 for tests on shotcrete.

Other recent studies have also indicated a linear relation between pullout force and compressive strength. Malhotra and Carette[54] looked at five mixes of concrete and found that the ratio of pullout strength to compressive strength decreases slightly as compressive strength increases, from 0.24 at 2860 psi to 0.18 at 7510 psi. Dilly and Ledbetter[28] also reported a linear relationship between pullout force and cylinder compressive strength, as well as a linear relationship between logarithm of maturity and compressive strength, in a 1984 study of one batch of concrete.

Both groups (Malhotra and Carette and Dilly and Ledbetter) used equipment that gave failure cones with apex angles of 67 degrees. Their embedded equipment is reported[97] to be about twice the size of that used by the Lok-Test system, which gives failure cones with apex angles of 62 degrees. Therefore, direct comparison of pullout forces is not possible between the two systems because, as Stone and Carino[87] showed, the failure surface varies with the apex angle of the cone. However, the failure mechanisms are thought to be fairly similar, since the apex angles are both above 54 degrees. Stone and Giza[88] found 54 degrees to be an important threshold angle for theoretical reasons. Hence, the pullout strengths, $f'_p = P/A$, can be expected to be reasonably comparable, where P is the pullout force and A is the area of the convex surface of a frustrum of a right circular cone.

In their paper,[97] Yener and Chen noted that efforts were being made to standardize the apparatus and the procedures, considering all significant variables. They also concluded:

> the pullout testing apparatus and procedure should be modified so as to minimize the effects of adverse environmental conditions and poor workmanship on test results.

Stone and Giza, however, concluded that a large percentage of the variability of the pullout test can be attributed to the random manner in which the aggregate particles cross the failure surface.

Malhotra and Carette[54] noted that the within-test variation of the pullout tests performed at 7 days varied from 0.9 to 14.3 percent with an average value of 5.3 percent. The compressive strength from 7-day-old cylinders (in the range of 2000 to 7000 psi) showed a coefficient of variation of 2.6 to 8.3 percent, and an average of 4.6 percent. They judged these values "comparable."

In 1984, Yener and Chen[97] said their data showed that the type of cement used in the concrete, curing time, and curing conditions had no effect on the pullout test results. Apparent influences of maximum particle size were noted and investigated before ultimately being dismissed as due to improper compaction of the test cylinders. They used both the standard pullout test and the cut and pullout test (or CAPO test), which requires no embedment in the fresh concrete. They found that CAPO test strengths averaged about 7 percent less than the regular pullout tests. The coefficient of variation (COV) was 4.6 percent for the pullout and 5.3 percent for the CAPO tests. Further tests showed COV's of 3.5 percent for cylinders, 7.2 percent for the pullout, and 7.1 percent for the CAPO test. They also showed a close agreement with the work of the Danes, who developed an equation in SI units for relating pullout strength, P, to compressive strength, $f'_c$:

$$P = 5 + 0.8f'_c$$

Field application of the pullout test was discussed by Bickley in two papers in 1984.[9,10] He related how pullout testing was used to judge the strength of concrete and to determine how fast construction could safely proceed in highrise office buildings built in Canada. In some cases the embedded pullout devices were placed in the top of the slab, and in other cases they were installed before casting, attached to the underside forms. One paper recommended that ACI 214 be changed to provide suitable procedures for all to follow if they wished to use this method.[9] The other stated that

> pullout inserts can be used either in the bottom or the top of a slab and that both procedures have about the same variability although the strength in the top of a slab is about six percent less than that of the bottom of a slab.[10]

Meanwhile, Carette and Malhotra at Canada Centre for Mineral and Energy Technology (CANMET) were doing studies in the laboratory to compare several in-situ test methods.[17] Among the methods was one very similar to the Danish Lok-Test equipment, identifiable by its dimensions: 25 mm in diameter, mounted 25 mm deep. They avoided referring to this method by name but simply called it "the commercially available pullout." They also used another type which was about twice as big and inserted twice as deep as the Lok-Test, which they called "the CANMET pullout." All pullout embedments were initially mounted on the forms of the side surfaces of their test slabs, as opposed to being "finger placed" on the top surface. They ranked their several methods from most variable to least variable: rebound test, commercially available pullout, CANMET pullout, and penetration resistance (Windsor probe). They reported the coefficient of variation for the commercially available pullout to be "of the order of 8.5 percent compared to about seven percent for the CANMET pullout, the number of tests being 10 for the former and four for the latter." However, these averages were for all of their tabulated data, from 1 day, 2 days, and 3 days. Review of their individual test results shows within-test variation actually ranges from 1.9 percent to 14.9 percent for the two different pullout tests at various times and on various slabs, and a couple of the farthest-out values had already been screened out before those variations were calculated.

At about the same time, Khoo[46] was doing research in Singapore and finding coefficients of variation of 2.0 to 12.3 for groups of six pullout tests. His inserts were attached to the sideforms of the test specimens. They had an apex angle of about 70 degrees, slightly different from the Lok-Test value of 62 degrees and Richards and others using 67 degrees.

A Korean investigation compared several nondestructive test methods, including the pullout and the CAPO test. Yun, Choi, Kim, and Song[99] found that in-situ tests had, in general, within-test variability two to five times that of compressive strength cylinder tests. They tested sets of eight inserts at 1, 3, 7, 14, 28, and 90 days, and they found coefficients of variability ranging from 2.9 to 27.0 percent.

Dilly and Vogt[29] used pullout tests to investigate the reported variations in concrete strength between the top and bottom of a column. At very early ages, pullout tests in lower parts of the column required twice as much pull force as those in upper parts of the column. The researchers had installed horizontal barriers to minimize the effects of bleed water. From this they deduced that the primary influence on varying strengths in columns is the varying dead load, because lower parts of the column see substantially more weight during curing than do the upper parts.

Field trials of pullout tests in Houston, Texas, were documented in a paper[96] published in 1984. Inserts were "finger placed" or set in the concrete immediately after the concrete was placed and tested at 2, 4, and 7 days. The overall variation was high, as might be expected over a six-month construction project. The authors also noted that the "within-test coefficients of variation are approximately twice as high as those accepted for compressive strength cylinders," ranging from 15 to 22 percent. The authors anticipated improvements in the equipment which would reduce that variation. They were not using the Lok-Test equipment from Denmark.

Ottosen[64] stated that nonlinear finite element analysis of the Lok-Test equipment shows "the failure in a Lok-Test is caused by the crushing of the concrete and not by cracking." He went on to say that the pullout force is directly dependent on the compressive strength of the concrete. Then he noted that the tensile strength of the concrete has some "indirect influence." However, he stated that

> The effect of strain softening in the post-failure region is important....[and] modelling of the strain softening in the post-failure region turns out to be a mandatory pre-requisite for realistic structural predictions.

Stone and Carino[87] disputed Ottosen's conclusions. After reviewing prior efforts at explanation of the pullout test failure mechanism, Stone and Carino conducted scale model tests on a pullout test apparatus that was twelve times larger than actual size. By carefully instrumenting the concrete in the failure zone, they determined that ultimate failure is governed by shear failure rather than tension or compression. They said failure occurred in three phases, and that the second phase would be the final phase but for the presence of large, strong aggregate particles. They stated:

> Even though complete propagation of circumferential cracking has taken place at 65 percent of ultimate load, the presence of randomly spaced large particles mechanically bridging the failure surface prohibits ultimate failure until all such particles have pulled out of the retaining matrix. This assertion is an important one for it means that any analytical model which cannot account for the discrete failure mechanism of aggregate interlock degradation is unapplicable.

This touched off considerable discussion in the literature.[87] Braestrup[41] concluded that the large-scale tests still did not show anything to dispute his previous claim that the pullout test provides a measure of concrete compressive strength. Ottosen took Stone and Carino's results and drew significantly different conclusions and these conclusions validated his own theoretical analysis. Petersen noted that in his experience selling Lok-Test equipment there is always substantial ductility associated with the failure, along with evidence of compression failure by crushing. Yener and Ting wrote that Stone and Carino had incorrectly assessed the work of Jensen and Braestrup[41] in their refutation of it. Yener and Ting also pointed out that a failure mechanism with a highly random nature, such as aggregate interlock, would produce high within-test variations, yet this is not found in the literature. Stone and Carino did not reply to each individual discussion item. However, they commented that the aggregate interlock idea actually explained concrete ductility rather than precluded it. They also questioned what is actually meant by the terms "crushing" and "compressive failure of concrete." They went on to assert that the basic reason why pullout tests can approximate cylinder strengths without using a compression failure mechanism is that in both tests the governing strength parameter is actually mortar tensile strength. They concluded that

> while there are different viewpoints about the failure mechanism of the pullout test, the test still provides a positive measurement of in-place concrete strength.[87]

In 1987, another finite element analysis was carried out by Hellier and several others.[38] It was concluded that pullout of an insert embedded 1 inch is not directly related to the compressive strength of the concrete, but good statistical correlation can be achieved between pullout strength and compressive strength obtained from cylinders. This study said a correlation should be established for each given concrete, implying that the Danish equation is not valid for all cases.

In a 1988 investigation of fiber-reinforced silica fume concrete, Horiguchi, Saeki, and Fujita[39] found that pullout test results correlate better with shear strengths than with compressive strengths.

Malhotra and Carette believed there is an upper bound to the strength range in which the pullout test is useful:

> At compressive strength levels beyond 8,000 to 9,000 psi the pullout strength should reach a maximum value beyond which there may be no increase in the pullout strength, regardless of the compression strength of the concrete.[54]

Nonetheless, they felt that the simplicity of the test was such a positive factor that it outweighed any negative aspects it may have. These negative aspects included (1) that they have to be planned for in advance except with CAPO testing, and (2) that patching is required afterwards unless the concrete makes strength, in which case there is no need to completely pull out the cone. They did not mention that if the CAPO test is performed it introduces the difficulty and danger of using hand-held power tools and a water-cooled drill bit at the same time, perhaps in a location far from ready power supplies. Still, Malhotra and Carette judged:

> It is one of the few tests available which quantitatively measures the strength of concrete in-situ and this alone is sufficient reason for its adaption by the construction industry.[54]

Similarly, Yener and Chen had a highly positive view of the test:

> Of the nondestructive tests available, the pullout tests appear to have the best potential for acceptance as a measure of the compressive strength of in-place concrete.[97]

## 2.5 MATURITY

### *2.5.1 SPECIFICATION*

Using the maturity method requires determination of a strength-maturity relationship, usually in the laboratory, followed by the field measurement of maturity. The field-measured maturity can then be translated into strength by using the strength-maturity relationship. ASTM C1074, "Estimating Concrete Strength by the Maturity Method," defines maturity and tells how to develop the strength-maturity relationship. Maturity is defined as

> 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.

The standard relates that temperature sensors must be embedded in at least two cylinders. The cylinders are then moist cured and the temperature history is recorded by either a temperature recorder or a maturity meter. Sets of three cylinders are then broken at ages of 1, 3, 7, 14, and 28 days, and at each age a record is made of the average maturity value of the instrumented cylinders. Associated strengths and maturity are then plotted and a best-fit line is drawn through them. This line represents the strength-maturity relationship. Once developed, it can be used to assess the in-situ strength of concrete based on the maturity only. However, the standard recommends that in critical situations this method should be supplemented with other test data from pullout tests, penetration tests, or accelerated curing tests.

### *2.5.2 HISTORY*

Investigation of the relationship of time and temperature to concrete strength is said to date back to 1904.[52] However, the maturity concept was firmly established in 1949 by Nurse.[63] In a study of steam-curing techniques, he plotted strength versus the product of time and temperature, and then he used the curve to predict the temperature necessary for the concrete to reach a given strength in 24 hours. He also used the curve to predict strength gain at a given temperature over a given time. He was anywhere from 10 to 30 percent low in his strength predictions.

The maturity concept was further established and defined by Saul. In another investigation of steam-curing techniques he found:

> The 'maturity' of concrete may... be defined as its age multiplied by the average temperature... which it has maintained. ... concrete of the same mix at the same maturity (reckoned in temperature-time) has approximately the same strength whatever combination of temperature and time go to make up that maturity.[82]

Saul thought that maturity should be calculated from a temperature of -10.5°C under ordinary conditions. However, he did acknowledge that was not the only possible choice and stated that

> This will give an approximate comparison under all conditions, but some corrective factor should be applied, temperature having a greater effect at first, and time later. For the purposes of steam curing, involving the comparison of normal curing with short periods at higher temperatures followed by normal curing, calculation of maturity from freezing point tends to correct for the above, and gives comparative results approximating those found by experiment for ages of 24 hours and more.

In the mid-1950's, Plowman[72] reviewed 26 series of tests, all conducted at constant curing temperatures, in published data from as far back as 1915. He determined that it all fit nicely with the maturity concept. Furthermore, a curve of the form

$$S = a + b(\log M)$$

could be fit to each set of data, where S is strength, M is maturity, and a and b are constants. The curves could then be used to establish the strength at any particular time to within, on the average, 3 percent. The range found in the published data was between -9.9 to 8 percent.

In the discussion of this work,[72] Klieger noted that the linear relationship is poor beyond 28 days. If it is to be used, he said, the concrete should be cured at 60 to 80°F, and the concrete should not be allowed to dry out during curing. He concluded that Plowman's method "may have practical use, provided the limitations are kept clearly in mind." Powers argued theoretically that the datum temperature should be -4°C rather than -12°C as Plowman had proposed. Marshall wrote that Plowman's equation was used in his experiments on accelerated curing methods, but the equation gave "too big an estimate of the 28-day strength if calculated from the accelerated tests." McIntosh looked at Plowman's results and commented that they seemed to be considerably in error at the age of 1 day. Therefore McIntosh recommended that the relation

> should be restricted to concrete cured at a fairly uniform temperature and within the range of maturity represented by about 3 to 28 days at normal temperatures.[72]

In 1957, Goral[33] pointed out that admixtures and the quality of component materials affect the rate of strength gain in concrete. He also suggested using a hyperbolic curve to represent the relationship of strength and maturity, rather than the logarithmic curve employed by Plowman. He developed several curves for the same data by selectively excluding certain data points while fitting curves to the data, and then he considered which curves might be most useful for certain purposes. For instance, an inaccurate or unrepresentative test at 2 days might have little effect on a prediction of 7-day strength, but could change the curve enough to significantly affect the 28-day strength prediction.

Klieger performed extensive experiments on the effects of temperature on concrete strength and along the way he looked briefly at the maturity concept. He cured concrete at various temperatures and found that "the 55°F concretes produced higher strengths than the 73°F concretes and still higher strengths than the 120°F concretes."[47] Based on this, he concluded that a concept "as simple as degree-days" was incapable of providing an adequate correlation with strength.

Chin[24] felt the relation between strength and maturity was best represented by the rectangular hyperbola

$$M/S = mM + C$$

where M is the maturity, S is the strength, and m and C are constants. This model showed that the strength approached some maximum value equal to 1/m, rather than continuing to increase for an infinite amount of time. Thus it seemed to be able to accommodate actual later-age data beyond 28 days better than Plowman's equation could.

In 1975, Malhotra summarized the state of the art of nondestructive testing, reporting that some basic ideas had been established:

> It is generally agreed that the concept of maturity gives valid results provided that (1) the initial temperature of concrete is between 15.5 degrees C and 26.6 degrees C (60 degrees F and 80 degrees F); (2) no loss of moisture by drying occurs during the curing period; and (3) maturity is represented by 3 days-28 days of curing at normal temperatures.[52]

In the late 1970's, the maturity method was put to use by analysts reviewing a construction disaster. West Virginia's Willow Island cooling tower collapsed in 1978, and the maturity analysis of that event was presented in a paper published in 1985.[36] Halvorsen and Farahmandnia simply took weather and construction records, along with some analysis of the cooling tower structure, and deduced what must have happened. According to their assessment, the disaster would have happened sooner if it were not for some construction delays which gave previously-placed concrete time to gain strength.

In 1983, Carino, Lew and Volz[20] proposed a three-parameter equation for the strength-maturity relationship as follows:

$$S = ((M-M_o) * S_u) / ((1/A) * (M-M_o) / S_u)$$

where $M_o$ is the maturity at which rapid strength gain begins, A is the initial slope of the strength versus maturity curve, and $S_u$ is the limiting strength as maturity approaches infinity. Application of this equation is considerably more complicated that the method used in ASTM C1074,[7] but the authors said the additional complexity was necessary to accommodate the effects of different initial curing temperatures. They concluded that the classical maturity method was indeed a reliable predictor of strength under outdoor curing conditions, provided the "early age" temperature of the concrete used to develop the strength-maturity relation was similar to the expected outdoor "early age" temperature. However, in another paper by Carino and Lew[19] published at about the same time, the authors reported that they tried to determine what "early age" means and to quantify the limit of it, but they found only that it was not half of initial set time, nor was it initial set time, nor was it final set time. Switching from extreme temperatures such as 41°F or 96°F to standard curing temperatures, 73°F, at those times made no difference to the subsequent strength-maturity relationship.

In 1984, Carino published a detailed treatise called "The Maturity Method: Theory and Application."[18] This paper contained a history of the development of the method. It included the method employed in the ASTM standard and the equivalent age approach, and a discussion of the relationship between the North American practice of using datum temperature and the European practice of using Arrhenius method activation energy. The maturity method assumes a linear relationship between the rate constant and temperature, while the Arrhenius method assumes a logarithmic relationship, yet both fit the data fairly well in the range of 40 to 110°F. Carino explained the basis of the theory, showing why various researchers could establish various datum temperatures as being the best. Carino concluded that the "mathematical simplicity of the traditional maturity method makes it more attractive than the equivalent age approach based upon the Arrhenius equation." He suggested that datum temperatures might be altered for various values of temperature and activation energy, and then went on to describe how the temperature and activation energy can be determined.

Taken together, the 1983 paper by Carino, Lew, and Volz[20] and the 1984 paper by Carino[18] give a description of how to determine the three parameters needed to fix the strength-maturity relationship. This relationship was described by the hyperbolic equation mentioned above, which was originally presented in the 1983 paper. The 1983 paper mentioned a trial-and-error procedure for determining those three parameters: ultimate strength, initial rate of strength gain, and maturity at which strength gain actually begins or maturity offset. It did not show clearly what was involved in that trial-and-error procedure, but comparison with the details shown in the 1984 paper clarify the issue. The basic premise is that one or more of the earliest data points may be ignored in the determination of ultimate strength. Also, one or more of the latest data points may be ignored while the initial rate of strength gain and maturity offset are found. Trial and error are necessary because there is no way to know which points to ignore until all combinations have been tried. Some combinations give absurd results, while others give excellent results, and this is best determined by actually performing the calculations. Note that this trial-and-error method is simply a small refinement of the maturity technique given in the ASTM standards. It provides a way to determine a hyperbolic relationship between strength and maturity, which can then be used in later strength predictions.

Dilly and Ledbetter[28] reported a linear relationship between pullout force and logarithm of maturity, as well as a linear relationship between logarithm of maturity and compressive strength, in a 1984 study of one batch of concrete.

At about the same time, Parsons and Naik were beginning to experiment with different values of the datum temperature. In 1984, they assumed a value of 32°F based on a combination of "careful review of existing data and the exercise of engineering judgement."[66] They first inferred from their data that there was a different maturity curve for every value of curing temperature they used. They also noted that the use of the 32-degree datum changed the high curve to the low curve. Based on this, they concluded that manipulation of the datum could be used to insure some high probability that the maturity method strength prediction was equal to or lower than the actual in-place strength.

Also in the early 1980's, Hulshizer and several other engineers wrote of their work on a 6-mile-long, 19-foot-diameter tunnel.[40] They used a combination of non-destructive tests to determine safe form-stripping times. The project was particularly difficult because they had to haul concrete long distances from the batch plant, yet they needed quick turnaround on their forming operations. In other words, they needed high early strength but they could not use a high early-strength cement because they also needed long set times. They opted to use a 7.5-sack mix with superplasticizers and then monitor in-situ strength very closely. They initially considered maturity, penetration, and pullout testing. Subsequently they dropped the pullout test because they decided it was incompatible with their forming system and because of the large number of pullout units they would have needed to be certain they had reached the desired strengths. They reported finding that their maturity-strength relationships were slightly different for test cylinders cured at 55, 60, and 70 to 73°F. This was particularly important when they were making form-stripping decisions at concrete ages of only 12 hours. They simply chose the most conservative relationship for their estimations and then to be more certain they verified in-situ strengths with penetration tests. They reported no failures and no sagging or cracking of the early-stripped overhead concrete.

Researchers at the National Sand and Gravel Association[32] studied the effects of extended delivery times and high temperatures on concrete strength. They noted numerous references which say slump or strength losses are associated with lengthy periods between mixing and placement, or with placement at temperatures above 90°F. They pointed out that several sources suggest making trial batches under expected conditions, rather than under standard conditions. Another paper[58] said that set retarding admixtures can counteract the unfavorable effects of temperature. These papers suggest that the mechanisms of strength gain are more complicated than Nurse and Saul's basic maturity concept.

Naik[59] studied the effects of several different constant curing temperatures on the maturity-strength relationship. He recommended that for very low-temperature curing of 37°F, the Arrhenius function should be used instead of the Nurse-Saul function. In his experiments, there was significantly less strength gain per degree-hour than the Nurse-Saul function indicated. At the age of about 3 days, the differences for water/cement ratios of 0.7 and 0.6, were about 38 and 32 percent, respectively. For a w/c ratio of 0.5, the difference dropped to about 20 percent. However, for higher temperatures of 55°F and 73°F, he found the Nurse-Saul function had a very good correlation with compressive strength.

Parsons[65] reviewed previous findings on the maturity method, which had determined that the logarithmic equation was the best model for predicting strength gain in cylinders. He analyzed this data using statistical methods suggested by Carino in "Maturity Method: Theory and Application,"[18] and found that the model was dependent on curing temperature. Parsons then used trial-and-error methods and Carino's hyperbolic equation to determine a new datum temperature. The oldest samples were only 7 days old, so he could only make a rough estimate of Carino's ultimate strength, $S_u$. Still, he felt that his model was a good predictor of strength. It was so good, in fact, that when he adjusted the datum to 25°F (-4°C), his statistical analysis showed that the strength gain was no longer affected by curing temperature. Unfortunately, the validity of all this could not be debated in the literature, because the paper was published in a special publication of the American Concrete Institute.

Some years later, Harrell[37] gave a report of successful field use of the maturity method along with other methods on another tunnel-lining project. He said that no one single method was relied on for final answers. Rather, a combination of several considerations was used to make certain that forms could be stripped, but the report showed again that the method could be useful in determining early age strength in the field. The report said reference curves were developed using results of testing performed on several batches of the actual concrete used for the construction project. However, based on the test results, the contractor was able to safely use a fly-ash mix which had lower early-age strength, thus reducing cost and heat-associated cracking problems without sacrificing job progress.

Bartos[8] reported that Naik found the average predicted in-place strength based upon maturity to be 1.1 percent greater than the predicted in-place strength based upon core tests. He termed this "significant" because ACI 318 accepted core strength as the true representation of in-place strength.

More recently, research from China[35] praised the Arrhenius equation, as opposed to the Saul maturity function, and proposed a quadratic expression to simplify application of the Arrhenius equation. The maturity method was seen as poorly suited for estimating the later age strength, as in ASTM C918, "Estimating Later Age Concrete Strength by the Maturity Method."[7] However, it was deemed useful for early-age in-situ strength estimates.

The paper also lauded a proprietary device called the COMA meter. The COMA meter had previously been discussed in several papers by European authors, including Petersen.[69] Its strong points were its ease of operation and close approximation of actual physical processes involved in concrete strength gain. According to the proprietors, the COMA meter is a closed capillary tube containing a special liquid. The tube is broken and embedded in fresh concrete. The liquid evaporates in a manner which integrates time and temperature according to the Arrhenius equation. The device is reported to be unaffected by moisture, wind or air temperature. It reads out in equivalent days of maturity, and may be purchased in 0- to 5-day or 0- to 14-day ranges.

## 2.6 ULTRASONIC PULSE VELOCITY

### *2.6.1 SPECIFICATION*

The method for measurement of pulse velocity of concrete is described in ASTM C597.[7] It lists the testing equipment required as: a pulse generator, a transmitter, a receiver, an amplifier, a time-measuring circuit, a display unit, and connecting cables. The cables are required to be short enough so that the voltage loss shall not exceed 0.5 percent. Frequencies are to be in the range of 10 to 150 kilohertz and the pulse generator must produce 10 to 150 pulses per second. The standard recommends that the transmitter and receiver be located on opposite sides of the concrete being tested, or on adjacent perpendicular sides if necessary, and only as a last resort along the same surface. The standard notes that reinforcing steel can significantly affect readings and should be avoided. Repeatability is good, in the range of 2 percent for distances of up to 20 feet, but cracks in the concrete can increase that range up to 20 percent.

The standard recommends that this test is to be used in assessing the uniformity and relative quality of concrete in-place or "to indicate changes in the properties of concrete," but also states that

> this method should not be considered as a means of measuring strength nor as an adequate test for establishing compliance of the modulus of elasticity of field concrete with that assumed in design.

### *2.6.2 HISTORY*

The first concrete testing using ultrasonic pulse velocity was in the late 1940's by Leslie and Cheesman.[50] They proposed it as a method of studying deterioration and cracking in concrete structures. They also determined the dynamic modulus of elasticity by ultrasonic means.

At about the same time, Jones[42] reported on variations of wave velocity in different concretes, results of experiments regarding Poisson's ratio, and use of ultrasonic methods to assess frost damage. He reported trying without success to determine pavement thickness by ultrasonic means.

Experiments by Andersen and Nerenst[5] showed that ultrasonic wave velocity could be used to compute the modulus of elasticity of concrete. However, they questioned the value of this computation, noting that Jones had found Poisson's ratio to vary considerably at early ages. They preferred to describe concrete in terms of pulse velocity rather than modulus of elasticity. They also concluded that "the wave velocity method is suitable to follow changes in concrete due to hardening." In the discussion of this paper, Leslie and Sturrup questioned Andersen and Nerenst's work, saying

> Experience has shown that velocity measurements along a surface do not necessarily reflect the quality of the interior concrete. For this reason, velocity measurements throughout the structure are normally to be preferred to surface transmissions. The authors do not state explicitly how transmissions through a specimen may be made with their instrument.[5]

By 1959, researchers had determined that pulse velocity did not correlate well with compressive strength. Kaplan reported that

> The ratio of changes in pulse velocity and compressive strength due to a change in water/cement ratio is not generally the same as that due to a change in age. Because of this, the relation between pulse velocity and compressive strength cannot be expected to be independent of age and water/cement ratio.[43]

Kaplan also noted that at early ages for concrete strengths below 4,000 psi, the value of the pulse velocity seemed to be less dependent on increasing age than on higher w/c ratio, while at later ages the reverse was true. In a later paper,[44] he stated that the relation between pulse velocity and compressive strength is dependent on age and on mix proportion ratios such as aggregate/cement and water/cement.

Early pulse-velocity instruments were cumbersome and not very portable. They required skilled operators and data interpreters. Digital readouts and nickel-cadmium batteries improved them significantly, but there are still problems. In 1975, Malhotra noted:

> Laboratory experience has shown that the surface of the concrete specimens has to be perfectly clean; the presence of a few sand particles between the transducers and the concrete surface interferes with the travel of the pulse through the concrete.[52]

In a 1979 article on the current state of nondestructive testing, Bartos reported on discussions with noted authorities in the field. He quoted Malhotra as saying:

> Inasmuch as a large number of variables affect relations between the strength parameters of concrete and its pulse velocity, the use of the latter to predict the compressive and/or flexural strength of concrete is not recommended.[8]

Shortly thereafter, Malhotra and Carette wrote that there might be some correlation between pulse velocity and other in-situ strength tests, but only under very controlled laboratory conditions. They cautioned:

> In general, pulse velocity measurements do not correlate well with strength of concrete and therefore these measurements should not be used to predict pullout or compressive strength of concrete.[54]

In 1981, Anderson and Seals[6] reported on developing models for using 1- or 2-day ultrasonic pulse velocity to predict strength at 28 days or later. Their prediction equations used pulse velocity at 1 day, pulse velocity at 2 days, the change in pulse velocity, slump, water/cement ratio, cement factor, and a factor they called the mix variable. In one case they reported the model lead to a coefficient of variation of 6 to 7 percent, versus about 4 percent within batches of cylinders and 8 percent between batches of cylinders. They noted that different aggregates and varying amounts of entrained air affected these predictions, and they suggested there is a special relationship between concrete maturity and pulse velocity, different from the strength-maturity relationship. They concluded

> Although a generalized equation for predicting strength was not possible, when compositional variables are considered, excellent prediction of 28-day and 90-day strength is possible.[6]

Beyond strength assessment and prediction, research has shown that ultrasonic testing can be used to assess uniformity, help identify defects such as voids or honeycombs, and evaluate fire-damaged concrete. Chung and Law[25] discussed these uses in a paper published in 1983. They emphasized that the methods are approximate, serving only as guides and indicators, not definite measurements.

Millstein and Sabnis commented on the general futility of trying to correlate ultrasonic pulse velocity with strength. They stated that if the failure of concrete is

> entirely determined as a separation, as in the case of axial tension/compression, then shear elasticity and velocity of transverse ultrasonic waves will have poor correlation with strength properties.[56]

They went on to say that, inadequate as they are, transverse waves correlate better with strength than do longitudinal waves, and longitudinal waves are the ones in common use.

In 1984, Carette and Malhotra[17] conducted another study at Canada Centre for Mineral and Energy Technology (CANMET) to compare several in-situ test methods. In a report published in 1984, they ranked several methods from most variable to least variable: rebound test, commercially available pullout, CANMET pullout, and penetration resistance or Windsor probe, and ultrasonic pulse velocity. They reported a coefficient of variation for the ultrasonic pulse velocity testing method of less than 1 percent. However, they concluded it may be undesirable for use in the field because the presence of rebar seriously affects the results.

In the same publication, a paper by Bungey[15] noted that pulses may travel up to 1.9 times faster in reinforcing steel than in the surrounding concrete. There are methods for accounting for the effects of steel on pulse velocity tests in concrete. However, Bungey said the correction factors currently recommended in Europe are inadequate. They may lead one to underestimate the velocity by as much as 30 percent. He proposed a method which accounted for the presence and size of the rebar in both longitudinal and transverse orientation to the pulse direction. He said it provided an accuracy of plus or minus 3 percent under laboratory conditions.

Samarin and Dhir[81] noted that in the 1960's researchers began studying the combination of ultrasonic pulse velocity tests with the rebound hammer, and such studies continued in the 1970's and 1980's. In 1984, Samarin and Dhir showed that 1-day and 7-day ultrasonic pulse velocity testing could be used to predict 28-day strengths within about 10 or 15 percent. They also reported that a combination of rebound hammer readings and pulse velocities could be used to predict core test results within about 30 percent.

Facaoaru[31] found that under the best of conditions, the ultrasonic method can be accurate to within as little as 12 to 16 percent. However, this is true only (1) if specimens or cores are available and (2) if the concrete mix design is available. Knowledge of the materials in the concrete allows comparison of experimental data with reference standard data, and corrections can be made to account for any compositional differences. The accuracy decreases to 14 to 18 percent if the mix design is unavailable, or to 18 to 25 percent if the specimens or cores are unavailable. If neither the mix design nor the specimens or cores are available, then Facaoaru states the error may be much more than 30 percent.

Combining rebound number with ultrasonic tests in a method Facaoaru calls sonreb testing results in a better estimate of strength. Facaoaru found that if a sample and a mix design are available, then the accuracy will be within 10 to 14 percent. If no samples or composition data are available, the accuracy is considered to be above 20 percent. Facaoaru also reported on a sonic coring method, which is an ultrasonic test "adapted to the control of concrete quality in deep foundations such as drilled piers or columns, slurry walls, etc."[31]

Others developing similar models found similar results. Tanigawa, Baba, and Mori[92] reported correlation coefficients of 0.784 for rebound number versus strength and 0.545 for pulse velocity versus strength. These relatively low correlations indicate significant scatter of data and serious uncertainty about the actual correlation. However, they found the correlation improved significantly to 0.936 when the rebound number and pulse velocity were considered together before being plotted against strength. Still, predictions of core strength by the combined method varied from 64 to 123 percent of actual strength. The researchers were encouraged by their findings but recognized that correction factors were required for concrete of various ages. Also, correction factors were required for concrete mixes with different sizes, volume fractions, and types of coarse aggregate, and for concretes with differing cement contents.

Another proposed way of making the ultrasonic test more useful was developed by Swamy and Al-Hamed.[90] In their paper, they described the paste efficiency concept, which uses air-dried cubes as representative samples of the in-situ concrete. These representative samples are used to predict in-place strength. They recorded calculated values which differed from actual values by 2 to 19 percent.

While emphasizing that the pulse velocity test is useful in certain circumstances, Sturrup, Vecchio, and Caratin listed numerous disadvantages and problems associated with it. For instance, they stated that the

> pulse velocity appears to correlate fairly well with compressive strength at early ages... [but] it is most insensitive to even large variations in strength at later ages. A pulse velocity/compressive strength relationship developed at early ages is not applicable at later ages. Similarly, a relationship established during the development stages of concrete cannot be used to follow retrogression of strength as concrete deteriorates.[89]

They said the pulse velocity test may tell more about the overall quality of a deteriorating structure than would cores, but cracks and reinforcing steel could confuse the readings. Furthermore, they emphasized that velocity/strength relationships can only be for a

> particular concrete of specific proportions under controlled conditions. ...[and] no attempt should be made to estimate the strength of concrete from pulse velocity values unless a prior relationship has been established.[89]

They contended that relationships established on laboratory specimens are not likely to be very useful in evaluating field structures. They suggested that field-cured specimens would provide a better evaluation, but the evaluation would still be inaccurate.

Some years later, Nasser and Al-Manaseer[60] reported that their pulse velocity tests were influenced by the type of aggregate used and also by the operator of the instruments.

In 1988, Teodoru[93] published a paper which built on the basis of his own previous work with ultrasonic pulse velocity and ultrasonic pulse attenuation. By his definition, attenuation is a change in the pulse that results from damping due to internal friction loss, scattering, reflection, and geometric considerations. Earlier work by

Teodoru and others had shown that attenuation is less affected than velocity by some external factors. Teodoru considered all three readings together: pulse velocity, pulse attenuation, and rebound number. He determined that by using all three together it was possible to improve the accuracy of compressive strength estimates compared with those made without including attenuation. The process required multiple data correlations, but Teodoru felt it reasonable for those seriously involved in quality control to invest in the computer needed to do the correlations. He reported the standard deviation rarely exceeded 10 to 15 percent, and the maximum relative deviations found were plus or minus 15 to 20 percent. Correlation coefficients for the calibration curves were in the range of 0.972 to 0.996.

Recently, Rix, Bay, and Stokoe[78] have investigated the use of Spectral Analysis of Surface Wave (SASW) techniques to determine in-situ stiffness. Surface waves are different from the longitudinal and transverse waves used by previous investigators. The researchers report some success using wavelengths that are about four times the diameter of the steel and are therefore thought to be unaffected by the presence of the steel. The researchers measure surface wave velocities and use them for calculations in the following equations:

$$V_R C = V_S$$

$$\rho V_S^2 = G$$

$$2G(1+\nu) = E$$

where $V_R$ is the measured velocity of the surface wave, C is a value between 1.05 and 1.14 which depends on Poisson's ratio, $V_S$ is the velocity of the shear wave, $\rho$ is the mass density, G is the shear modulus, $\nu$ is Poisson's ratio, and E is the modulus of elasticity. From a measured velocity of the surface wave, the modulus of elasticity can be computed, and this is related to the strength of the concrete. It should be noted that plots relating concrete stress to strain, thus defining the modulus of elasticity, are characterized by significant scatter in the data, and these data have been presented several times in the last thirty years.[23,67] Also, there is some uncertainty surrounding the value of C and Poisson's ratio. Rix, Bay, and Stokoe note that C changes with Poisson's ratio, and Poisson's ratio may change significantly as concrete begins to gain strength. Their findings agree with those of Jones,[42] who determined in 1949 that Poisson's ratio varies considerably, especially during the first 24 hours.

Rix, Bay, and Stokoe[78] also used their SASW method to determine concrete set times. They reported good agreement with the penetration-resistance test for set time, which is described in ASTM C403, Test Method for Time of Setting Concrete Mixtures by Penetration Resistance.[7] They also report their values of Young's Modulus averaged about 13 percent higher than those determined by the static modulus tests described in ASTM C469, Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression.[7] They thought that the higher values of Young's modulus were accurate, reflecting actual differences in the modulus due to differences in the curing of concrete in a cylinder as opposed to the concrete in a slab. They did not feel these differences were indicators of errors in the SASW technique.

## 2.7 CORES

### *2.7.1 SPECIFICATION*

ASTM C42 states that concrete must be hard enough to allow removal of a core without damage to the mortar-aggregate bond, and it recommends that the concrete be at least 14 days old. A core should be drilled in a direction perpendicular to the bed on which the concrete was placed. It must be at least 4 inches in diameter, and at least as long as it is wide. If the ratio of length to diameter is over 2.10, it should be cut off, and if that ratio is less than 1.94, there are strength correction factors that must be applied to the measured strength. The ends must be smooth and perpendicular to the longitudinal axis of the core and they must be capped before testing. The standard says they should be submerged in lime-water for at least 40 hours immediately prior to compression testing, then kept moist on the way from the lime-water tank to the testing machine. As with standard 6-inch-by-12-inch cylinders, the compressive strength is computed by dividing the cross-sectional area of the core by the maximum compressive force.

### *2.7.2 HISTORY*

In 1965, Bloem at the National Ready Mixed Concrete Association published a report on his study which compared the results of core and cylinder testing. He stated:

> In attempting to interpret the relationships between cores and cylinders remember that existing strength controls based on standard cylinder tests have served well for acceptance of concrete. Further, when reasonable care has been taken to handle and cure the structure properly, acceptance on the basis of standard tests has seemed to provide ample strength in place. Core tests made to check adequacy of strength in place must be interpreted with judgment. They cannot be translated to terms of standard cylinder strength with any degree of confidence, nor should they be expected necessarily to exceed the specified strength, $f'_c$.[13]

In this paper, Bloem showed results of core tests on slabs and compared them to standard 28-day molded cylinders. The core strength ranged from 65 to 107 percent of the molded cylinder strength, depending on the curing procedure and whether the cores were soaked or dried before testing or simply tested immediately after coring. Strength of concrete in well-cured columns was shown to be in the 80 to 90 percent range except in the top 10 inches of the column, where it dropped to 60 percent of 28-day molded strength. Bloem quoted various papers by authors from respected institutions and agencies as emphatically stating that cores will be lower, higher, or just the same as 28-day cylinders, indicating a dramatic divergence of opinion on the subject.

Subsequently, Yener and Chen wrote:

> Recognizing that, in general, cores would give a lower strength than standard cylinders, the ACI Committees 318 and 301 [on both of which Delmar Bloem served] stipulate that if the average core strength is at least 85 percent of the specified strength $f'_c$, with the condition that all cores possess strength above 0.75 $f'_c$, the strength of the cast concrete is adequate.[97]

They went on to quote a paper[83] which mentions that with smooth river gravel, core strengths may be as low as 0.65 $f'_c$.

Malhotra[51] listed numerous factors influencing the strength of cores, including: damage during drilling and handling, poor compaction and curing of concrete in a structure, water gain during drilling or dryness at time of testing, size and age of cores compared to control cylinders, and loss of entrained air during handling and compaction of concrete. He then reviewed literature and reported his own findings on various other factors including the length-diameter ratio of cores, the presence of rebar in cores, the type of aggregate used in the concrete, the direction of drilling, and the curing temperatures and dimensions of the structure from which the cores are taken.

Among the various observations, Malhotra noted that

> cores for limestone concrete test somewhat lower than molded cylinders at all strength levels whereas there is little or no difference between strength of cores and molded specimens for gravel concrete. No explanation is offered for this difference in test results.[51]

Cores were found to follow the same rules as concrete cylinders with respect to curing temperatures. In tests of large blocks of concrete cast outdoors in the winter, Malhotra found that:

> The high compressive strength of cores at later ages is probably due to the low initial curing temperatures of cast concrete.[51]

He summarized with this caution:

> It cannot be overemphasized that unless extra caution is exercised during drilling and testing, and care is taken to allow for the effect of various variables discussed earlier in analyzing the core test results, the evaluation of the core test data presents a rather hazardous situation. ... [I]t has been shown that, more often than not, the drilling and testing of cores can create more problems than they solve if an attempt is made to relate the strength of cores to the strength of control cylinders.[51]

The paper then proceeded with a "new approach" to compressive strength determination, one which harks back to the discussions of the 1930's and the ACI presidents' messages in the 1970's. He proposed, first, the carrying out of sufficient inspection and control to make certain the concrete supplied meets the specifications for water-cement ratio, slump, air content, and cement content; second, elimination of 28-day cylinder tests and substitution of accelerated strength tests; and third, use of standardized in-situ nondestructive tests, such as maturity, pulse-velocity, penetration resistance and rebound tests, and pullout tests. Nine authors contributed to the discussion of this paper, all agreeing with Malhotra to one degree or another.

In 1984, Swamy and Al-Hamad[91] investigated the use of 50-mm cores (slightly less than 2 inches) and found that concrete strength estimated from these cores ranged from 72 to 85 percent of cube strength for regular concrete and 80 to 90 percent for lightweight. The researchers noted that type of aggregate, proportion of cement in the mix, and age of specimen all seemed to influence the variability of their results.

Recently, Bayesian statistical methods and non-destructive testing have been combined with coring in the evaluation of existing structures. A paper by Kriviak and Scanlon[49] gives an example of how results of pulse velocity and rebound hammer testing were used to select locations for coring, and the results of all tests were used to estimate the in-situ compressive strength of the concrete throughout an existing bridge.

## 2.8 PENETRATION RESISTANCE

### *2.8.1 SPECIFICATION*

The penetration resistance method is described in ASTM C803.[7] A known amount of energy is used to propel a steel probe into concrete. The probe is 1/4 inch in diameter and just over 3 inches long. The penetration resistance of the concrete is assessed by measuring the length of probe left exposed after one end of the probe is driven into the concrete.

The probe is driven into the concrete using a gunpowder charge and a specially designed pistol. A metal template is used to position the three probes. When all three probes are driven, a mechanical device can then be used to average the length of probe exposed. That average length is measured using a calibrated depth gage.

The manufacturer of the test equipment provides calibration tables which give the strength associated for every exposed probe length. The size of the powder charge, the corresponding force with which the probe is driven into the concrete, and the concrete aggregate hardness are also accounted for in the tables. Hardness is measured on Mohs' scale, in which talc has a value of 1 and diamond has a value of 10.

Recently, there has been some work published on the use of pins in place of the steel probes,[62] and a proposal has been made to change ASTM C803 to include this method. The pins are propelled by a low-energy, spring-actuated driver rather than the gunpowder-actuated driver used previously. The pins have a smaller diameter than the commonly-used probes, and they appear to test primarily the strength of the mortar paste.

### *2.8.2 HISTORY*

There are reports of testing techniques developed along the same lines in the 1950's,[53] so the Windsor Probe does not appear to be the original penetration resistance test but simply the first to come into wide use. Malhotra reported the Windsor Probe was developed by the Port Authority of New York and New Jersey and the Windsor Machinery Company in the 1960's. He said:

> [it] is a hardness tester. The claim that the penetration of the probe reflects the "precise compressive strength in a localized area" is not strictly true.[52]

Malhotra showed that the calibration tables furnished by the manufacturer are not satisfactory and he recommended that users develop calibration curves for each different aggregate used. However, the manufacturer continued, as of 1989, to supply the calibration curves with each case of probes sold.

Others agreed that the Windsor probe penetration test is basically a test of hardness and it cannot be expected to yield absolute values of strength of concrete in a structure. Ramakrishnan[75] said that penetration tests and rebound numbers provide an excellent means for determining relative strengths in different structures, but emphasized that the limitations of the Windsor probe must be recognized.

Carette and Malhotra[17] at Canada Centre for Mineral and Energy Technology (CANMET) conducted an experiment in the laboratory to compare several in-situ test methods. As was mentioned previously, they ranked several methods from most variable to least variable: rebound test, commercially available pullout, CANMET pullout, penetration resistance (Windsor probe), and ultrasonic pulse velocity. They reported a coefficient of variation for the penetration resistance method of about 5 percent at 1, 2, and 3 days, but they noted this was based on embedded length, not strength.

English experiments discussed in a paper by Keiller[45] showed that the manufacturer's recommendations for using low-power charges were not always appropriate. The manufacturer suggested using a low-power charge for concrete up to 3,625 psi, but the researcher said low-power charges did not always give satisfactory results when the strength was above 2,175 psi. Also, the manufacturer's calibration curves consistently overestimated strengths. Keiller thought the Windsor Probe method was useful for

> comparative exercises to estimate strength differences, but for actual strength estimation it would be necessary to obtain a calibration for the specific type of concrete being investigated.[45]

In a study aimed primarily at evaluating certain coring practices, Swamy and Al-Hamad[91] also investigated penetration tests. They found that penetration tests always overestimated the strength of lightweight concrete and frequently overestimated the strength of dense concrete. In addition, they reported that for concrete aged 360 days, the penetration test was quite inaccurate: "The Windsor probe appeared unable to evaluate strengths of older concrete in the range 25-55 MPa [3600 to 8000 psi]."[91]

A report by Harrell[37] of successful field use of the Windsor probe shows that the method can be helpful in determining when it is safe to remove concrete forms. However, the report indicated that reference curves were developed using results of testing performed on several batches of the actual concrete for the construction project studied. The manufacturer's calibration curves were not used.

## 2.9 REBOUND NUMBER

### *2.9.1 SPECIFICATION*

The method for measurement of rebound number is described in ASTM C805.[7] It lists the testing equipment required as a rebound hammer apparatus and an abrasive stone for grinding rough test surfaces. A hammer within the apparatus is initially propelled by a spring, with a predetermined amount of energy, toward a stationary steel plunger. The steel plunger is in contact with the concrete to be tested. The test simply quantifies the distance travelled by the hammer as it rebounds after impacting the steel plunger.

The standard notes that the rebound number may be used to assess uniformity of the concrete in-situ, and that it "provides useful information" for those making decisions on when to strip forms and shores. However, it bluntly states, "This test method is not intended as an alternative for strength determination of concrete," and it recommends correlating rebound numbers with core testing information.

The standard lists numerous factors that will cause variations in the readings, and discusses what to do about highly variable data.

The ASTM standard quotes a report by the National Ready Mixed Concrete Association as saying that the temperature of the apparatus (especially temperatures well below freezing) may affect readings. Other factors listed as known to affect the rebound readings are concrete temperature if below freezing, direction in which the apparatus is oriented during operation, moistness or dryness of the surface, presence of carbonation, and the texture and finish of the surface. Texture depends on the type of form used, if any. Malhotra cites findings of several earlier researchers who found that surfaces produced by wooden forms yield numbers up to 25 percent lower than troweled surfaces or surfaces produced by metal forms. However, Malhotra notes that troweled surfaces have been shown to give data with a higher degree of scatter. The standard declares that rough surfaces cannot be used for testing, and they should be ground down with a grinding stone or power equipment before tests are conducted, but then the results cannot be compared with results from smooth surfaces which do not require grinding. Test data from one apparatus cannot be compared with data from another apparatus. Ten readings are required for each test area considered but this can be done in a relatively short time. Then the average is calculated and reported.

### *2.9.2 HISTORY*

The rebound number was developed in the 1940's by a Swiss engineer named Ernst Schmidt—hence the common names for the rebound apparatus: Swiss hammer or Schmidt hammer. However, Malhotra[53] credits A.T. Shore with first describing the test in 1911.

In 1957, Zoldners[100] showed that the method had some drawbacks. He found that changing the direction of the impact of the hammer from downward to upward will make a difference of 10 points in the value of the rebound number. Zoldners also noted that two cylinders of the same strength but different ages, i.e., 7 and 28 days, will have rebound numbers from two to five points different. In addition, dry specimens tested five points lower than those soaked in water and tested in a saturated-surface-dry condition. Zoldners thought the hammer had

> accuracy ... sufficient to determine probable strength limits of the concrete in the structure and detect low-strength batches, provided it is calibrated properly and used competently by a skilled operator.[100]

Within the past decade, Akashi and Amasaki concluded that the principle of the rebound hammer

> may be more complex than is assumed when consideration is given only to the simple problem of applying Newton's laws to impacting bodies. It may involve considerable components of longitudinal wave transmission.[4]

They suggested that analysis of the rebound hammer seems to be somewhat similar to analysis of pile-driving hammers. They concluded that the rebound hammer should be calibrated by testing a material with a constant hardness and measuring the resulting impact stress wave.

More recently, ACI Committee 228 issued a report on in-place testing which included some discussion of the rebound hammer[2]. The report commented briefly on the theoretical basis of the rebound hammer and its practical limitations. The energy transmitted to the concrete by the hammer is partially absorbed, and the amount absorbed depends on the strength and stiffness of the concrete. Therefore, the committee points out, 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 committee notes that aggregate type, which affects concrete stiffness, will have an effect on the rebound number. Also, the forms will affect it because plywood will absorb some moisture that a steel form will not. This can slightly alter the water/cement ratio at the surface, changing the concrete strength at that location. Curing conditions may significantly affect surface strength and thus affect rebound numbers. Regardless of the surface effects, however, it should be noted that the rebound number can only indicate surface strength and this may not be representative of the interior concrete.

Some research has been done using the rebound test in combination with other nondestructive tests. Several examples were mentioned in Section 2.6, in the discussion of the ultrasonic pulse velocity method.

In 1987, Nasser and Al-Manaseer[60] reported experiments which confirmed that rebound number also is affected by the type of aggregate used in the concrete, specifically lightweight versus normal-weight aggregate, and by water-cement ratio. They also found that the relationships between cylinder compressive strength and rebound number could be approximated by straight lines with fairly good correlations, although the correlations were not as good as those they found for the pullout test or for the ultrasonic test.

## 2.10 SUMMARY OF LITERATURE REVIEW

Reviewing the literature shows that in seventy years since ASTM standardized cylinder testing no consensus has been reached regarding nondestructive testing of concrete. Bloem[12] indicated this when he compared the question "What is the strength of concrete?" to Pilate's biblical question "What is truth?" Both are apparently simple questions that are actually very difficult to answer.

Cylinders and beams are the standard in the United States but they have obvious drawbacks. They do not reflect the effects of field curing on the strength of the concrete in the structure. Field cure specimens do show some effects of field curing, but they are not necessarily representative of the concrete in the structure. Even cores drilled from the structure and tested in compression have been found to be less than truly representative of the structure. Recent research suggests that no specimen can truly represent a structure unless the specimen is the same shape as the structure and either the same size or else carefully and correctly scaled. Otherwise, the structure may fail before the stress at any location reaches the stress at which the test specimen failed.

Since cylinders and beams are not true indicators of structural strength, in-situ tests have been proposed. Sometimes the results are influenced more by surface hardness or stiffness than strength, as are the rebound number and penetration tests. In these cases, the property measured is not simply strength, and the measured value must be related back to cylinder strength. Thus the in-situ test may account for some effects of field curing, but in doing so it takes a roundabout route and ultimately provides only the cylinder strength, which may or may not be the "truth."

Ultrasonic test methods also take a roundabout route to determine cylinder strengths. They require a significant amount of judgement and skill to perform and still provide only a general description of the strength of the concrete. Perhaps with further development they will become more objective but for now they are too subjective for common use on construction sites.

Pullout test strengths and maturity method strengths have also been related to cylinder strengths in the search for true strength. Like the other in-situ tests, these tests incorporate the effects of field curing of the structure. Some contend that the relation between pullout strength and cylinder strength is a linear one and that the mechanics of a pullout test are similar to those of a cylinder compression test. Others dispute this, and the discussion has led to questions about the nature of crushing of concrete, in which some people ask if concrete is ever simply crushed or whether crushing is a kind of tensile failure. If the proponents are correct, the pullout test indicates the in-situ strength of the inch or two of concrete closest to the surface in terms of laboratory cylinders. However, laboratory cylinders do not represent specific parts of the structure, and their inherent weakness is that they cannot truly represent anything other than a cylindrical structure. Thus it is not certain that true strength is known when the in-situ strength is given in terms of laboratory cylinders. Similarly, maturity method strength is dependent on curing conditions of the structure, and it may also be thought of as an in-situ strength in terms of laboratory cylinders. Again, however, this may not be the true strength.

In summary, the literature indicates that nondestructive tests can provide useful information on strength; however, caution must be exercised in using the results. The results do not necessarily show the true strength of concrete.

# CHAPTER 3. EXPERIMENTAL PROGRAM

## 3.1 INTRODUCTION

The experimental program is described in the following sections. Included are discussions of the details of the test specimens, the curing procedures employed, the test procedures performed, and the test regimen followed.

## 3.2 TEST SPECIMENS

This section describes the concrete used in this experiment, the details of the slabs cast, and the details of other castings.

### *3.2.1 CONCRETE*

Concrete for this study was supplied by a commercial supplier. As shown in Table 3-1, the concrete had a specified design strength of either 3,500 psi or 5,000 psi. The mixes were designed with 20 to 25 percent of the cement replaced with fly ash. Total cementitious content ranged from 370 to 494 pounds per cubic yard. Water/cement plus fly ash ratios by weight ranged from 0.72 for the lower strength concrete to 0.47 for the higher strength concrete. As Table 3-1 shows, maximum aggregate size was usually 3/4 inch; the coarse aggregate was usually rounded river gravel. However, some mixes were made with a different maximum size or with an angular crushed limestone coarse aggregate.

Aside from the nondestructive tests being studied, several other tests were performed on the concrete at the time of casting, including ASTM Standard C173, Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method[7]; Standard C143, Test Method for Slump of Portland Cement Concrete[7]; and Standard C1064, Temperature of Freshly Mixed Portland Cement Concrete.[7] Results of these tests are presented in Table 3-2.

### *3.2.2 SLABS*

Castings 1 through 9 were slabs on grade cast outdoors over a period of 9 months between August 1989 and May 1990. Each slab was nominally 10 inches thick. All slabs were cast on two inches of damp masonry sand or concrete sand. Horizontal dimensions varied from a minimum of 8 feet by 8 feet to a maximum of 10 feet by 18 feet. Tests were conducted along 1-foot grid lines to insure adequate separation and to eliminate effects of nearby tests. No test was conducted within 1 foot of the edge of any slab. Forms consisted of 3/4-inch plywood treated with a form release agent.

Concrete for each slab was delivered by a commercial ready-mix supplier, one truckload per slab. The sizes of the loads varied from 3 to 8 cubic yards. Concrete was placed in two equal lifts, each lift being individually vibrated using internal vibrators immediately after placement. The top of each slab was screeded and then smoothed with a few passes of a bullfloat. The slabs were not troweled. A few hours after placement, at a time when the concrete appeared be past final set, the concrete was covered with several layers of wet burlap and a sheet of 6-mil polyethylene.

The burlap and plastic were left in place for 7 days, except that they were removed for enough time to conduct tests at 1 day and 3 days. Side forms were left undisturbed for 7 days and were then removed.

Concrete samples for quality control specimens were taken out of the middle portion, approximately, of each load. The first lift of concrete was placed in the slab

**TABLE 3-1. DESCRIPTION OF CONCRETES USED FOR TESTING**

| Casting Number | Design Strength (psi) 3,500 | 5,000 | Aggregate Type Rounded | Angular | Maximum Aggregate Size (in.) 3/8 | 3/4 | 1 1/2 |
|---|---|---|---|---|---|---|---|
| 1 | X | | X | | | X | |
| 2 | X | | X | | | X | |
| 3 | X | | X | | | X | |
| 4 | X | | | X | | X | |
| 5 | | X | X | | | X | |
| 6 | | X | | X | | X | |
| 7 | | X | X | | | X | |
| 8 | X | | X | | X | | |
| 9 | X | | X | | | | X |
| 10 | X | | X | | | X | |
| 11 | X | | X | | | X | |

**TABLE 3-2. RESULTS OF TESTS ON FRESH CONCRETE**

| Casting Number | Slump (in.) | Air (%) | Temperature Ambient (°F) | Temperature Concrete (°F) |
|---|---|---|---|---|
| 1 | 6.00 | – | – | – |
| 2 | 6.00 | 2.00 | 92 | 85 |
| 3 | 4.50 | 3.50 | 75 | 77 |
| 4 | 1.25 | 2.50 | 68 | 67 |
| 5 | 5.25 | 3.00 | 55 | 61 |
| 6 | 3.50 | – | 54 | 61 |
| 7 | 4.25 | – | 72 | 77 |
| 8 | 1.75 | – | 59 | 70 |
| 9 | 3.25 | 2.00 | 73 | 81 |
| 10 | 5.75 | 3.50 | 81 | 84 |
| 11 | 5.00 | 2.25 | 82 | 86 |

forms, then sample cylinders and beams were cast, and finally the second lift was placed in the forms. The entire placement and sampling procedure required about 45 to 60 minutes. After the placement was complete, the pullout test inserts and maturity thermocouples were placed as quickly as possible.

### *3.2.3 OTHER CASTINGS*

Castings 10 and 11 consisted solely of two sets of cylinders that were cast and cured under carefully controlled laboratory conditions. Each set of cylinders was made of concrete taken from the middle of a 3-cubic-yard load.

## 3.3 CURING

For the slabs of castings 1, 2, and 3, cylinders and beams were all cured according to ASTM Standards C31, Standard Method of Making and Curing Concrete Test Specimens in the Field,[7] and C78, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading).[7] Cylinders were cast in plastic molds, covered with plastic lids, and left adjacent to the slab overnight. At 20 to 24 hours, they were demolded and taken to a wetroom for curing at 73°F and 100 percent humidity. Beams were cast in steel molds and then covered with several layers of wet burlap and a sheet of 6-mil polyethylene, and left adjacent to the slab. On the following day, the beams were demolded and taken to the wetroom.

For the slabs of castings 4 through 9, two groups of specimens were prepared: the wetroom group and the outdoor group. The wetroom group was cast indoors and left in a temperature-controlled environment overnight before being transferred to the wetroom for further curing. The outdoor group was cast outdoors and left adjacent to the slab overnight. The cylinders were covered with plastic cylinder lids and a sheet of 6-mil polyethylene. The beams were covered with wet burlap and 6-mil polyethylene. After 7 days, the lids, burlap, and polyethylene were removed.

For castings 10 and 11, no slabs were cast, and the cylinders were moved to a controlled environment of 100 percent relative humidity in the laboratory immediately after casting. Details of the curing regimen are given in Section 3.5.3.

## 3.4 TESTS PERFORMED

Eight different types of tests were conducted during this investigation. They were cylinder compression, two types of pullout, penetration resistance, rebound, flexural beam, maturity, and compression using drilled cores.

### *3.4.1 CYLINDER COMPRESSION*

These tests were conducted in accordance with ASTM C39, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.[7] Except as noted, the cylinders were cured for 24 hours at temperatures within the range of 60 to 80°F as specified by ASTM; they were then moved to a controlled environment for curing at 73°F and 100 percent relative humidity until testing. In noted cases, the cylinders were cast and stored next to the slabs receiving the same treatment and curing as the slabs in the field. The compression tests were performed using unbonded neoprene bearing pads instead of sulfur caps. Sets of three companion cylinders were broken on each test day.

### *3.4.2 LOK-TEST PULLOUT TESTS*

These tests were conducted in accordance with ASTM C900, Standard Test Method for Pullout Strength of Hardened Concrete.[7] They consisted of "finger-placed" Lok-Test inserts placed at the top surface of the concrete. Lok-Test inserts are 1 inch in diameter and are embedded 1 inch below the surface of the concrete. Pullout tests were performed in sets of twelve at each test age and were evenly distributed at locations across the surface of each slab. Inserts were pulled out with a center-pull hydraulic jack supplied by the insert manufacturer. As the insert was pulled, the maximum pullout force was read on the pressure gauge of the jack, which was calibrated in kilonewtons of pulling force. The manufacturer of the testing equipment supplies a calibration chart for converting the pullout force in kilonewtons to the strength of the concrete in megapascals; strength in megapascals was converted to strength in pounds per square inch.

#### 3.4.3 CAPO-TEST PULLOUT TESTS

These tests were also conducted in accordance with ASTM C900-87, except as noted. These tests are similar to the Lok-Test, but the insert consists of a steel ring which is expanded into a 1-inch hole drilled in the concrete. The hole is milled into the concrete with a power tool a short time before testing. The expanded ring substitutes for the Lok-Test insert, but otherwise the procedure is the same. CAPO tests, which take considerably more time to conduct than ordinary pullout tests, were performed in sets of six at each age, evenly distributed at locations across the surface of each slab.

#### 3.4.4 PENETRATION RESISTANCE

These tests, commonly known as Windsor Probe tests, were conducted in accordance with ASTM C803-82, Standard Test Method for Penetration Resistance of Hardened Concrete.[7] In this test, steel probes are driven into concrete and the length of the probe left exposed is measured. These tests were done with a low-power charge, the type of charge needed for concrete at early ages. At later ages when the concrete strength had exceeded the range of the low-power charge, the high-power charge was used. The manufacturer provided a calibration chart giving the strength of the concrete in pounds per square inch for each type of charge and each length of probe exposed after being driven into the concrete. This test is designed to be conducted in groups of three shots; a set of tests consisted of two such groups. Thus, one measurement is the strength associated with the mechanical average of the length of three test probes, and the recorded data are the mathematical average of two strength measurements. That is, six individual probes were used for each test slab on each testing day.

#### 3.4.5 REBOUND NUMBER

Also called the Schmidt hammer or Swiss hammer, these tests were performed in accordance with ASTM C805-85, Rebound Number of Hardened Concrete. The readings were taken with the instrument in a downward vertical position. Two groups of ten readings were taken, and the mean of each group was used to calculate the mean standard deviation and coefficient of variation value for the test data. Strengths were read off a chart provided by the manufacturer, which has a curve relating rebound number to strength in pounds per square inch. The lowest value on the chart is 1,700 psi for a rebound number of 10, so the strength associated with a rebound number between 0 and 10 was recorded as 1,700 psi.

#### 3.4.6 FLEXURAL BEAMS

The flexural strength of the concrete was determined using flexural beams tested in third-point loading as per ASTM C78-84.[7] Sets of three beams were tested on each test day. Beams were cured in the same manner as the cylinders described in Section 3.4.1.

#### 3.4.7 MATURITY

These tests were conducted in accordance with ASTM C1074-87, Practice for Estimating Concrete Strength by the Maturity Method.[7] Maturity was measured using a variety of instruments including a Fluke Helios I computer, an Intelicom single-channel maturity meter and a World Tec 4-channel temperature recorder and maturity meter. Maturity totals are the average of three readings, except in cases where equipment failure reduced that number to two or one. Strength-maturity relationships and predictions were developed for cylinders cured at 73°F and 100 percent relative humidity in the wetroom, in accordance with ASTM C918-80, Methods for Developing Early Age Compression Test Values and Projecting Later Age Strengths[7] and ASTM C1074-87.[7] In addition, strength-maturity relationships were developed for outdoor-cured cylinders, flexural strength beams cured in the wetroom or outdoors, and drilled cores. These strength-maturity relationships are also called maturity curves. Temperature readings were made using thermocouples which were embedded 3 to 4 inches deep in the slabs, cylinders, and beams. The thermocouples were centered in the tops of cylinders and placed about 4 inches from the ends of the beams. On each test day, the concrete maturity in degree-hours was recorded. Then strength-versus-maturity curves were used to predict concrete strengths in pounds per square inch.

#### 3.4.8 CORES

Core-drilled cylinders were tested as per ASTM C42, Methods of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.[7] Four-inch-diameter cores were drilled the full depth of the slab and then cut to 8 inches in length. These were drilled in groups of three using a commercial core drill. All cores were drilled and cut on the day of testing and kept in a surface-damp condition until testing. Tests were conducted on sets of three cores except where the slab was too weak for coring or when only one core was obtained because of equipment failure.

## 3.5 TEST REGIMEN

For the first three castings, seven different types of tests were conducted. For the next six castings, two kinds of tests were dropped and six more were added. For the final two castings, only two types of tests were conducted. The first three castings are termed "Phase I," the next six are called "Phase II," and the last are identified as "Phase III." Each phase is discussed below.

### *3.5.1 PHASE I TESTING*

The seven different tests conducted in Phase I were compression cylinder, flexural beam, CAPO pullout, Lok-Test pullout, penetration resistance, rebound number, and maturity. These tests are indicated in Table 3-3 by the seven "X" marks in the columns of castings 1, 2, and 3 between lines A and J.

Each test was conducted for each casting at 1, 3, 7, and 28 days. In addition, compression cylinder tests were conducted at 14 days as required for development of maturity curves.

### *3.5.2 PHASE II TESTING*

After Phase I was completed, the CAPO-test pullout test and the rebound number test were discontinued for the reasons explained in Chapter 5. Four of the tests added in Phase II were variations of the maturity test described in Section 3.4.7. ASTM Standard C1074 requires that laboratory-cured cylinders be used to develop a maturity curve. However, it is noted on lines K through N of Table 3-3 that for these tests the strengths on which the maturity curve was based came from other sources such as flexural beam tests or compression tests on drilled cores.

The other two tests added in Phase II are listed on lines B and D of Table 3-3. These were variations of cylinder and beam tests in which the specimens were cured outdoors next to the slabs rather than in the controlled environment of the laboratory wetroom.

As in Phase I, tests were conducted at 1, 3, 7, and 28 days with additional tests at 14 days if maturity curves were developed. Since only the Lok-Test pullout test and the Windsor probe penetration tests were not associated with maturity curves, they were the only tests not conducted at 14 days.

### *3.5.3 PHASE III TESTING*

As indicated in Table 3-3, only compression cylinder tests and maturity method tests were conducted on the concrete of the last two castings. These tests were completed in 3 days. Concrete was of the same mix design used for casting 3, so maturity strength determinations were based on maturity curves developed with wetroom-cured cylinders for casting 3. These tests were conducted to determine the effects of high curing temperatures on the results of the maturity tests.

In casting 10, the curing temperature of the cylinders was held at 73°F for four hours, then increased from 73 to 110°F in about two hours, held at 110°F for 12 hours, gradually decreased to 73°F in about two hours, and then held at 73°F for 2 days. Meanwhile, a group of control companion cylinders were cured at a constant 73°F. Maturities were recorded for each curing schedule, and cylinders from each group were broken at ages of 1 and 3 days. The maturity readings were used together with the previously developed maturity curve to predict the strength of the cylinders, and then those cylinders were tested in compression to check the strength prediction.

All of the procedures used on casting 11 were the same as those used on casting 10, with two exceptions. First, the maximum curing temperature was 130°F. Second, the comparison between predicted and actual strength was also made at 2 days, in addition to 1 and 3 days.

**TABLE 3-3. LIST OF NONDESTRUCTIVE TESTS CONDUCTED ON EACH CASTING**

| | | | Casting Number | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Phase 1 | | | Phase 2 | | | | | | Phase 3 | |
| | | Type of Test | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
| A | Cylinders | Cured in wetroom | X | X | X | X | X | X | X | X | X | X | X |
| B | Cylinders | Cured outdoors | | | | X | X | X | X | X | X | | |
| C | Beams | Cured in wetroom | X | X | X | X | X | X | X | X | X | | |
| D | Beams | Cured outdoors | | | | X | X | X | X | X | X | | |
| E | Pullout | CAPO - test | X | X | X | | | | | | | | |
| F | Pullout | Lok - test | X | X | X | X | X | X | X | X | X | | |
| G | Pen. Resistance | Windsor probe | X | X | X | X | X | X | X | X | | | |
| H | Rebound Number | Schmidt hammer | X | X | X | | | | | | | | |
| I | Cores | 4 in. diameter x 8 in. | | | | X | X | X | X | X | X | | |
| J | Maturity | Based on cylinders cured in wetroom | X | X | X | X | X | X | X | X | X | | |
| K | Maturity | Based on cylinders cured outdoors | | | | X | X | X | X | X | X | | |
| L | Maturity | Based on beams cured in wetroom | | | | X | X | X | X | X | X | | |
| M | Maturity | Based on beams cured outdoors | | | | X | X | X | X | X | X | | |
| N | Maturity | Based on cores | | | | X | X | X | X | X | X | | |

# CHAPTER 4. RESULTS

## 4.1 INTRODUCTION

Results of the three phases of testing are presented in this chapter. Values reported are the mean standard deviation and coefficient of variation (COV) of several companion tests at a given concrete age. The number of companion tests depends on the test method, as described in Section 3.4. In certain instances, the concrete was too weak to test, so no data are presented. In other instances, only one reading was taken, so no standard deviation could be calculated. All deviations from the standard test procedures are indicated in the test results.

The first three castings are termed Phase I, castings 4 through 9 are termed Phase II, and castings 10 and 11 are termed Phase III. Details of differences between the phases are given in Section 3.5. Results of each phase of testing are presented below, with each phase discussed in a separate section.

## 4.2 PHASE I

Tables 4-1 through 4-4 contain the results of tests performed at 1, 3, 7, and 28 days, respectively, on the castings of Phase I.

Figure 4-1 shows the strength-maturity relationships developed for the Phase I castings. The curves were developed in the manner described in ASTM C918[7] which involves plotting the cylinder strength and the base 10 logarithm of the maturity value for the cylinders and fitting a least-squares regression through the data. ASTM C918 mentions that a visual curve fit may also be used, but this is subject to interpretation and therefore the visual curve fit was not used for this report. Data used to develop the strength and maturity curves for the Phase I slabs may be found in Appendix A.

## 4.3 PHASE II

Phase II consisted of castings 4 through 9. Results are presented first for castings 4, 5, and 6 and then for castings 7, 8, and 9.

### *4.3.1 CASTINGS 4, 5, AND 6*

Testing was performed at 1, 3, 7, 14, and 28 days for all methods which were used in the development of maturity curves and these results are presented in Tables 4-5 through 4-9. The Lok-Test and Windsor probe tests were not used in the development of maturity curves, so these tests were performed only at 1, 3, 7, and 28 days. These results are presented in Tables 4-5 through 4-7 and in Table 4-9.

Figures 4-2, 4-3, and 4-4 show the maturity curves which were developed for castings 4, 5, and 6. Some of the maturity curves were based on wetroom-cured cylinder maturities and compressive strengths, as required in ASTM C1074[7] and C918,[7] but others were also developed based on maturities and strengths from four other sources: cylinders cured outdoors, cores drilled from the slab, beams cured in the wetroom, and beams cured outdoors. Appendix B contains the strength and maturity data from which these curves were developed.

### *4.3.2 CASTINGS 7, 8, AND 9*

Tables 4-10 through 4-14 contain the results of tests performed on the slabs of castings 7, 8, and 9 at 1, 3, 7, 14, and 28 days. Again, the Lok-Test and Windsor probe tests were not performed at 14 days.

Figures 4-5, 4-6, and 4-7 show the maturity curves which were developed for castings 7, 8, and 9. These maturity curves were based on maturities and strengths from several sources including wetroom cylinders (per ASTM C1074[7]), outdoor cylinders, cores drilled from the slab, wetroom beams, and outdoor beams. Appendix C contains the strength and maturity data from which these curves were developed.

## 4.4 PHASE III

In casting 9, the curing temperature of a set of cylinders was varied between 73°F and 110°F while maturity was measured. At the ages of 1 and 3 days, the cylinder maturity and a previously-developed maturity curve were used to predict the strength of the cylinders. At the same time, three of the cylinders were compression-tested to check the prediction. Likewise, the maturity of three cylinders cured at a constant 73°F was used to predict their strength, and the prediction was checked against the actual strength. Results are presented in Table 4-15, which lists strengths predicted by the maturity method and actual strengths from cylinder compression tests.

In casting 10, the same procedure was followed, except that the high curing temperature was 130°F and the tests were conducted at 20, 45, and 69 hours. Results are presented in Table 4-16.

**TABLE 4-1. RESULTS OF 1-DAY TESTS ON CASTINGS 1, 2, AND 3**

| | Cylinders Cured in wetroom | Beams Cured in wetroom | Pullout CAPO-Test | Pullout LOK-Test | Pen. Resist. Windsor Probe | Rebound Number | Maturity from wetroom cylinders |
|---|---|---|---|---|---|---|---|
| Casting 1 | | | | | | | |
| Average Strength (psi) | 1,000 | 290 | 1,840 | 1,530 | 900 | 1,700 | 1,325 |
| Standard Deviation (psi) | 53 | 21 | 250 | 190 | 280 | 300 | 2.9 |
| Coefficient of Variation | 0.053 | 0.072 | 0.136 | 0.124 | 0.311 | 0.176 | 0.002 |
| Casting 2 | | | | | | | |
| Average Strength (psi) | 1,000 | 290 | 1,280 | 1,160 | 650 | 1,700 | 1,288 |
| Standard Deviation (psi) | 7 | 20 | 340 | 240 | 71 | 500 | 2.8 |
| Coefficient of Variation | 0.007 | 0.069 | 0.266 | 0.207 | 0.109 | 0.294 | 0.002 |
| Casting 3 | | | | | | | |
| Average Strength (psi) | 1,280 | 370 | 1,890 | 1,520 | 1,550 | 1,700 | 1,473 |
| Standard Deviation (psi) | 47 | 18 | 416 | 183 | 1,060 | 300 | 4.8 |
| Coefficient of Variation | 0.037 | 0.049 | 0.220 | 0.120 | 0.684 | 0.176 | 0.003 |

**TABLE 4-2. RESULTS OF 3-DAY TESTS ON CASTINGS 1, 2, AND 3**

| | Cylinders Cured in wetroom | Beams Cured in wetroom | Pullout CAPO-Test | Pullout LOK-Test | Pen. Resist. Windsor Probe | Rebound Number | Maturity from wetroom cylinders |
|---|---|---|---|---|---|---|---|
| Casting 1 | | | | | | | |
| Average Strength (psi) | 2,290 | 500 | 3,090 | 2,790 | 3,000 | 2,600 | 2,444 |
| Standard Deviation (psi) | 145 | 23 | 720 | 500 | 280 | 1,500 | 5.9 |
| Coefficient of Variation | 0.063 | 0.046 | 0.233 | 0.179 | 0.093 | 0.577 | 0.002 |
| Casting 2 | | | | | | | |
| Average Strength (psi) | 2,060 | 420 | 2,390 | 2,120 | 1,500 | 1,700 | 2,128 |
| Standard Deviation (psi) | 48 | 10 | 400 | 460 | 990 | 300 | 1.4 |
| Coefficient of Variation | 0.023 | 0.024 | 0.167 | 0.217 | 0.660 | 0.176 | 0.001 |
| Casting 3 | | | | | | | |
| Average Strength (psi) | 2,500 | 500 | 2,340 | 2,520 | 2,550 | 1,700 | 2,390 |
| Standard Deviation (psi) | 74 | 48 | 560 | 190 | 210 | 300 | 7.7 |
| Coefficient of Variation | 0.030 | 0.096 | 0.239 | 0.075 | 0.082 | 0.176 | 0.003 |

**TABLE 4-3. RESULTS OF 7-DAY TESTS ON CASTINGS 1, 2, AND 3**

| | Cylinders Cured in wetroom | Beams Cured in wetroom | Pullout CAPO-Test | Pullout LOK-Test | Pen. Resist. Windsor Probe | Rebound Number | Maturity from wetroom cylinders |
|---|---|---|---|---|---|---|---|
| Casting 1 | | | | | | | |
| Average Strength (psi) | 3,050 | 560 | 4,350 | 3,390 | 3,200 | 2,000 | 3,260 |
| Standard Deviation (psi) | 31 | 45 | 780 | 460 | 280 | 750 | 7.1 |
| Coefficient of Variation | 0.010 | 0.080 | 0.179 | 0.136 | 0.088 | 0.375 | 0.002 |
| Casting 2 | | | | | | | |
| Average Strength (psi) | 2,690 | 520 | 2,990 | 2,460 | 2,800 | 3,000 | 2,722 |
| Standard Deviation (psi) | 80 | 35 | 620 | 520 | 140 | 500 | 5.7 |
| Coefficient of Variation | 0.030 | 0.067 | 0.207 | 0.211 | 0.050 | 0.167 | 0.002 |
| Casting 3 | | | | | | | |
| Average Strength (psi) | 2,770 | 550 | 3,480 | 2,760 | 2,600 | 1,875 | 3,036 |
| Standard Deviation (psi) | 300 | 53 | 475 | 620 | 570 | 500 | 4.6 |
| Coefficient of Variation | 0.108 | 0.096 | 0.136 | 0.225 | 0.219 | 0.267 | 0.002 |

**TABLE 4-4. RESULTS OF 28-DAY TESTS ON CASTINGS 1, 2, AND 3**

| | Cylinders Cured in wetroom | Beams Cured in wetroom | Pullout CAPO-Test | Pullout LOK-Test | Pen. Resist. Windsor Probe | Rebound Number | Maturity from wetroom cylinders |
|---|---|---|---|---|---|---|---|
| Casting 1 | | | | | | | |
| Average Strength (psi) | 4,170 | 670 | 4,370 | 4,500 | 3,750 | 1,700 | 4,619 |
| Standard Deviation (psi) | 160 | 17 | 530 | 580 | 210 | 300 | 0.8 |
| Coefficient of Variation | 0.038 | 0.025 | 0.121 | 0.129 | 0.056 | 0.176 | 0 |
| Casting 2 | | | | | | | |
| Average Strength (psi) | 3,710 | 590 | 4,450 | 2,980 | 3,850 | 4,350 | 3,733 |
| Standard Deviation (psi) | 65 | 9 | 960 | 430 | 210 | 650 | 3 |
| Coefficient of Variation | 0.018 | 0.015 | 0.216 | 0.144 | 0.055 | 0.149 | 0.001 |
| Casting 3 | | | | | | | |
| Average Strength (psi) | 4,650 | 605 | 3,860 | 3,830 | 3,900 | 3,400 | 4,482 |
| Standard Deviation (psi) | 117 | 49 | 390 | 614 | 141 | 475 | 3.8 |
| Coefficient of Variation | 0.025 | 0.081 | 0.101 | 0.16 | 0.036 | 0.14 | 0.001 |

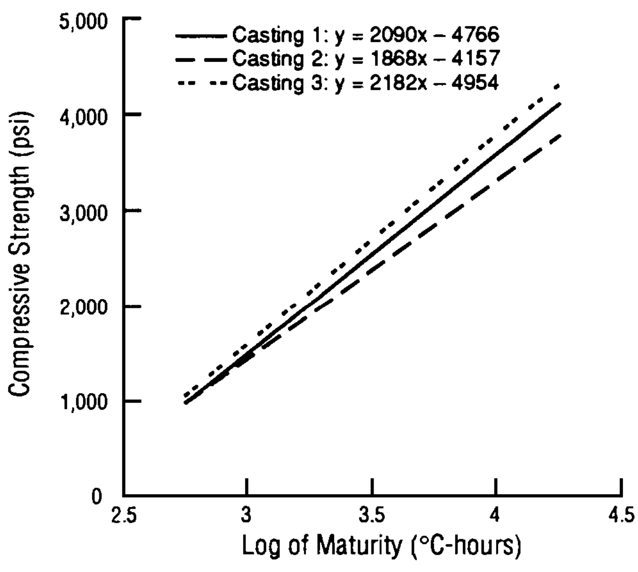


**Fig 4-1. Maturity curves for castings 1, 2, and 3.**

**TABLE 4-5. RESULTS OF 1-DAY TESTS ON CASTINGS 4, 5, AND 6**

| | Cylinders | | | Beams | | | | Maturity Strengths Listed by Basis of Maturity Curve: Cylinders | | | Maturity Strengths Listed by Basis of Maturity Curve: Beams | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors | Pullout LOK-Test | Pen Resist Windsor Probe | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors |
| Casting 4 | | | | | | | | | | | | |
| Av. Strength (psi) | 549 | 489 | 846 | 153 | 113 | 457 | # | 730 | 615 | 998 | 346 | 179 |
| St. Deviation (psi) | 26 | 12 | 86 | 9 | 9 | 134 | # | 4 | 156 | 5 | 2 | 2 |
| Coeff. of Variation | 0.047 | 0.025 | 0.102 | 0.061 | 0.083 | 0.294 | # | 0.005 | 0.254 | 0.005 | 0.006 | 0.009 |
| Casting 5 | | | | | | | | | | | | |
| Av. Strength (psi) | 2,000 | 2,138 | 2,832 | ** | 437 | 2,147 | # | 2,590 | 2,578 | 3,248 | 580 | 442 |
| St. Deviation (psi) | 33 | 23 | 77 | ** | 8 | 162 | # | 56 | 48 | 50 | 3 | 1 |
| Coeff. of Variation | 0.016 | 0.011 | 0.027 | ** | 0.019 | 0.076 | # | 0.021 | 0.019 | 0.015 | 0.005 | 0.002 |
| Casting 6 | | | | | | | | | | | | |
| Av. Strength (psi) | 1,486 | 1,070 | 1,384 | 325 | 242 | --- | 975 | 2,040 | 1,467 | 1,797 | 539 | 349 |
| St. Deviation (psi) | 35 | 62 | 137 | 14 | 15 | --- | 100 | 12 | 33 | * | 2 | 1 |
| Coeff. of Variation | 0.024 | 0.058 | 0.099 | 0.042 | 0.064 | --- | 0.103 | 0.006 | 0.022 | * | 0.003 | 0.002 |

# Too weak to test  ** Data lost  --- Equipment being serviced  *Only one reading

**TABLE 4-6. RESULTS OF 3-DAY TESTS ON CASTINGS 4, 5, AND 6**

| | Cylinders | | | Beams | | | | Maturity Strengths Listed by Basis of Maturity Curve: Cylinders | | | Maturity Strengths Listed by Basis of Maturity Curve: Beams | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors | Pullout LOK-Test | Pen Resist Windsor Probe | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors |
| Casting 4 | | | | | | | | | | | | |
| Av. Strength (psi) | 2,443 | 1,809 | 2,128 | 460 | 403 | 1,568 | 1,500 | 2,366 | 1,890 | 2,136 | 471 | 341 |
| St. Deviation (psi) | 22 | 37 | 61 | 22 | 33 | 373 | 1,500 | 0 | 46 | 18 | 0 | 0 |
| Coeff. of Variation | 0.009 | 0.020 | 0.028 | 0.047 | 0.082 | 0.238 | 1 | 0 | 0.024 | 0.008 | 0 | 0 |
| Casting 5 | | | | | | | | | | | | |
| Av. Strength (psi) | 4,675 | 4,312 | 5,073 | 663 | 610 | 3,896 | --- | 4,117 | 3,781 | 4,656 | 660 | 567 |
| St. Deviation (psi) | 128 | 31 | 162 | 17 | 24 | 302 | --- | 31 | 55 | 38 | 0 | 1 |
| Coeff. of Variation | 0.027 | 0.007 | 0.032 | 0.026 | 0.040 | 0.077 | --- | 0.008 | 0.014 | 0.008 | 0.001 | 0.001 |
| Casting 6 | | | | | | | | | | | | |
| Av. Strength (psi) | 5,052 | 3,688 | 4,608 | 652 | 581 | 3,675 | 3,300 | 4,482 | 3,648 | 4,169 | 658 | 476 |
| St. Deviation (psi) | 36 | 168 | 119 | 26 | 29 | 404 | 50 | 3 | 42 | * | 0 | 0 |
| Coeff. of Variation | 0.007 | 0.045 | 0.026 | 0.040 | 0.050 | 0.110 | 0.015 | 0.001 | 0.011 | * | 0.001 | 0.001 |

--- Equipment being serviced  *Only one reading

## TABLE 4.7. RESULTS OF 7-DAY TESTS ON CASTINGS 4, 5, AND 6

| | Cylinders | | | Beams | | | Pen Resist | Maturity Strengths Listed by Basis of Maturity Curve | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Cylinders | | | Beams | |
| | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors | Pullout LOK-Test | Windsor Probe | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors |
| Casting 4 | | | | | | | | | | | | |
| Av. Strength (psi) | 3,766 | 3,657 | 3,574 | 573 | 553 | 2,583 | 2,263 | 3,479 | 3,118 | 3,204 | 555 | 468 |
| St. Deviation (psi) | 81 | 172 | 211 | 12 | 19 | 252 | 213 | 0 | 96 | 28 | 0 | 0 |
| Coeff. of Variation | 0.022 | 0.047 | 0.059 | 0.022 | 0.034 | 0.097 | 0.094 | 0 | 0.031 | 0 | 0 | 0 |
| Casting 5 | | | | | | | | | | | | |
| Av. Strength (psi) | 5,979 | 5,009 | 5,957 | 707 | 587 | 5,348 | --- | 5,337 | 4,683 | 5,686 | 721 | 659 |
| St. Deviation (psi) | 133 | 145 | 161 | 12 | 33 | 667 | --- | 14 | 46 | 41 | 0 | 1 |
| Coeff. of Variation | 0.022 | 0.029 | 0.027 | 0.018 | 0.056 | 0.125 | --- | 0.003 | 0.01 | 0 | 0 | 0.001 |
| Casting 6 | | | | | | | | | | | | |
| Av. Strength (psi) | 6,750 | 6,272 | 6,196 | 783 | 639 | 5,022 | 3,688 | 6,219 | 5,668 | 5,763 | 744 | 593 |
| St. Deviation (psi) | 74 | 102 | 290 | 30 | 48 | 759 | 113 | 1 | 94 | * | 0 | 3 |
| Coeff. of Variation | 0.011 | 0.016 | 0.047 | 0.038 | 0.075 | 0.151 | 0.031 | 0 | 0.017 | * | 0 | 0.005 |

--- Equipment being serviced *Only one reading

## TABLE 4-8. RESULTS OF 14-DAY TESTS ON CASTINGS 4, 5, AND 6

| | Cylinders | | | Beams | | | Pen Resist | Maturity Strengths Listed by Basis of Maturity Curve | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Cylinders | | | Beams | |
| | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors | Pullout LOK-Test | Windsor Probe | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors |
| Casting 4 | | | | | | | | | | | | |
| Av. Strength (psi) | 4,597 | 4,164 | 4,058 | 633 | 520 | --- | --- | 4,539 | 4,483 | 4,074 | 637 | 589 |
| St. Deviation (psi) | 112 | 227 | 196 | 9 | 22 | --- | --- | 0 | 3 | 12 | 0 | 6 |
| Coeff. of Variation | 0.024 | 0.055 | 0.048 | 0.015 | 0.042 | --- | --- | 0 | 0.001 | 0 | 0 | 0.01 |
| Casting 5 | | | | | | | | | | | | |
| Av. Strength (psi) | 6,058 | 5,175 | 6,669 | 786 | 762 | --- | --- | 6,330 | 5,417 | 6,540 | 770 | 731 |
| St. Deviation (psi) | 31 | 293 | 243 | 8 | 5 | --- | --- | 7 | 34 | 25 | 0 | 1 |
| Coeff. of Variation | 0.005 | 0.057 | 0.036 | 0.010 | 0.007 | --- | --- | 0.001 | 0.006 | 0.004 | 0 | 0.001 |
| Casting 6 | | | | | | | | | | | | |
| Av. Strength (psi) | 7,430 | 7,420 | 6,748 | 750 | 730 | --- | --- | 7,470 | 6,944 | 6,909 | 806 | 668 |
| St. Deviation (psi) | 204 | 245 | 190 | 39 | 33 | --- | --- | 1 | 159 | * | 0 | 7 |
| Coeff. of Variation | 0.027 | 0.033 | 0.028 | 0.052 | 0.045 | --- | --- | 0 | 0.023 | * | 0 | 0.011 |

--- Equipment being serviced *Only one reading

TABLE 4-9. RESULTS OF 28-DAY TESTS ON CASTINGS 4, 5, AND 6

| | Cylinders | | | Beams | | | | Maturity Strengths Listed by Basis of Maturity Curve: Cylinders | | | Maturity Strengths Listed by Basis of Maturity Curve: Beams | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors | Pullout LOK-Test | Pen Resist Windsor Probe | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors |
| Casting 4 | | | | | | | | | | | | |
| Av. Strength (psi) | 5,183 | 5,034 | 4,727 | 701 | 681 | 3,472 | 3,575 | 5,425 | 5,035 | 4,920 | 705 | 693 |
| St. Deviation (psi) | 85 | 230 | 61 | 18 | 43 | 685 | 0 | 0 | 0 | 3 | 0 | 0 |
| Coeff. of Variation | 0.016 | 0.046 | 0.013 | 0.026 | 0.063 | 0.197 | 0 | 0 | 0 | 0.001 | 0 | 0 |
| Casting 5 | | | | | | | | | | | | |
| Av. Strength (psi) | 7,008 | 6,106 | 7,023 | 805 | 815 | 6,501 | 5,100 | 7,345 | 6,276 | 7,423 | 821 | 811 |
| St. Deviation (psi) | 268 | 277 | 480 | 46 | 56 | 690 | 300 | 3 | 52 | 16 | 0 | 1 |
| Coeff. of Variation | 0.038 | 0.045 | 0.068 | 0.057 | 0.069 | 0.106 | 0.059 | 0 | 0.008 | 0.002 | 0 | 0.001 |
| Casting 6 | | | | | | | | | | | | |
| Av. Strength (psi) | 8,235 | 8,012 | 8,018 | 892 | 664 | 6,995 | 4,088 | 8,741 | 8,726 | 8,315 | 868 | 769 |
| St. Deviation (psi) | 102 | 249 | 50 | 30 | 29 | 663 | 213 | 0 | 67 | * | 0 | 2 |
| Coeff. of Variation | 0.012 | 0.031 | 0.006 | 0.033 | 0.043 | 0.095 | 0.052 | 0 | 0.008 | * | 0 | 0.003 |

*Only one reading

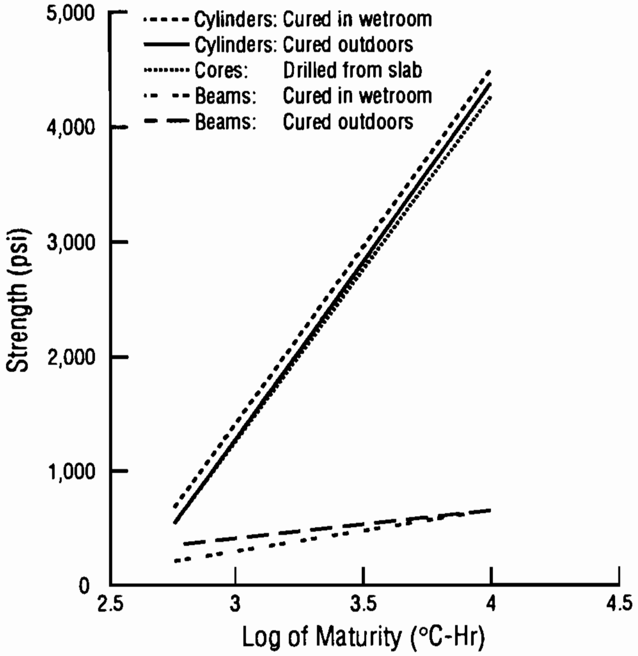


Fig 4-2. Maturity curves for casting 4.

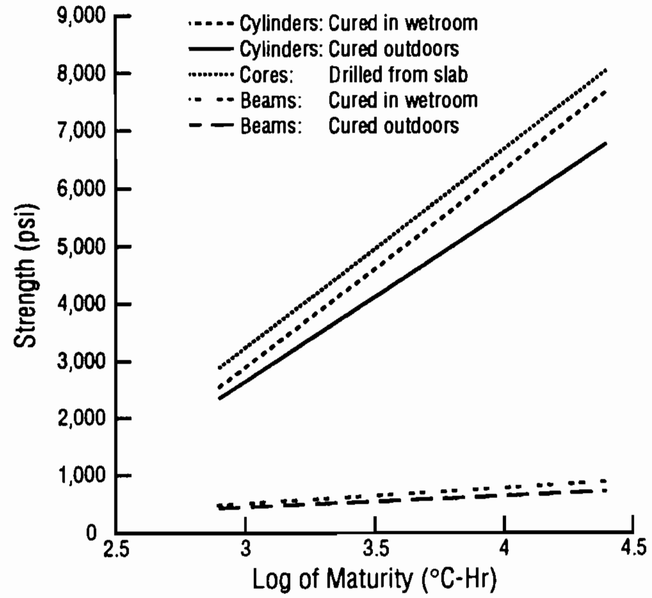


Fig 4-3. Maturity curves for casting 5.

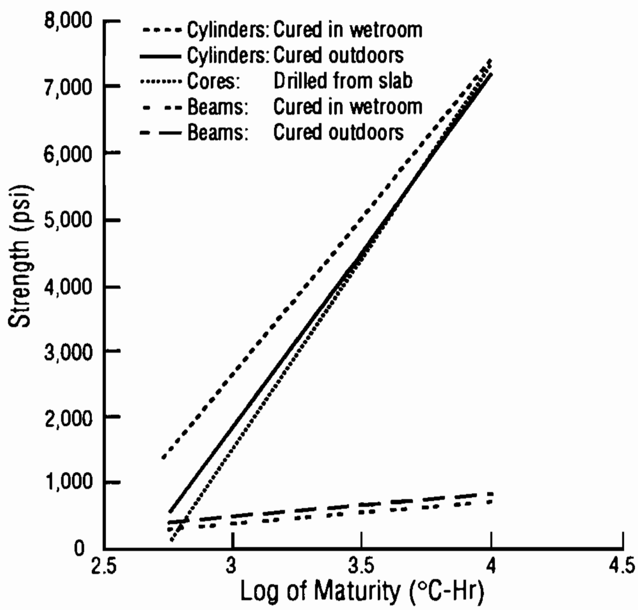


Fig 4-4. Maturity curves for casting 6.

## TABLE 4-10. RESULTS OF 1-DAY TESTS ON CASTINGS 7, 8, AND 9

| | Cylinders | | | Beams | | | | Maturity Strengths Listed by Basis of Maturity Curve: Cylinders | | | Maturity Strengths Listed by Basis of Maturity Curve: Beams | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors | Pullout LOK-Test | Pen Resist Windsor Probe | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors |
| Casting 4 | | | | | | | | | | | | |
| Av. Strength (psi) | 1,764 | 1,523 | 2,398 | 400 | 363 | 1,641 | 1,400 | 2,371 | 2,004 | 2,762 | 544 | 385 |
| St. Deviation (psi) | 149 | 52 | 20 | 22 | 26 | 195 | 100 | 4 | 16 | 0 | 2 | 2 |
| Coeff. of Variation | 0.084 | 0.034 | 0.008 | 0.054 | 0.072 | 0.119 | 0.071 | 0.002 | 0.008 | 0 | 0.004 | 0.004 |
| Casting 5 | | | | | | | | | | | | |
| Av. Strength (psi) | 357 | 376 | 0 | 130 | 130 | 116 | 0 | 516 | 528 | 0 | 189 | 171 |
| St. Deviation (psi) | 34 | 25 | 0 | 3 | 4 | 116 | 0 | 5 | 8 | 0 | 1 | 2 |
| Coeff. of Variation | 0.095 | 0.067 | 0.000 | 0.021 | 0.029 | 1.000 | 0 | 0.095 | 0.067 | 0 | 0.021 | 0.029 |
| Casting 6 | | | | | | | | | | | | |
| Av. Strength (psi) | 1,324 | 1,250 | 1,443 | 301 | 294 | ** | ** | 1,691 | 1,573 | ** | 449 | 342 |
| St. Deviation (psi) | 15 | 65 | 33 | 9 | 9 | ** | ** | 17 | 23 | ** | 1 | 2 |
| Coeff. of Variation | 0.012 | 0.052 | 0.023 | 0.029 | 0.031 | ** | ** | 0.01 | 0.015 | ** | 0.003 | 0.005 |

**No data

## TABLE 4-11. RESULTS OF 3-DAY TESTS ON CASTINGS 7, 8, AND 9

| | Cylinders | | | Beams | | | | Maturity Strengths Listed by Basis of Maturity Curve: Cylinders | | | Maturity Strengths Listed by Basis of Maturity Curve: Beams | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors | Pullout LOK-Test | Pen Resist Windsor Probe | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors |
| Casting 4 | | | | | | | | | | | | |
| Av. Strength (psi) | 4,727 | 3,894 | 4,941 | 633 | 617 | 4,235 | 3,250 | 4,237 | 3,734 | 4,695 | 651 | 563 |
| St. Deviation (psi) | 76 | 52 | 37 | 9 | 21 | 458 | 50 | 2 | 21 | 0 | 1 | 2 |
| Coeff. of Variation | 0.016 | 0.013 | 0.007 | 0.015 | 0.033 | 0.108 | 0.015 | 0 | 0.006 | 0 | 0.001 | 0.003 |
| Casting 5 | | | | | | | | | | | | |
| Av. Strength (psi) | 1,305 | 1,146 | 1,188 | 300 | 323 | 932 | 750 | 1,097 | 1,037 | 1,198 | 293 | 297 |
| St. Deviation (psi) | 21 | 39 | 17 | 14 | 5 | 71 | 250 | 2 | 8 | 8 | 0 | 1 |
| Coeff. of Variation | 0.016 | 0.034 | 0.015 | 0.047 | 0.015 | 0.076 | 0.333 | 0.095 | 0.067 | 0 | 0.021 | 0.029 |
| Casting 6 | | | | | | | | | | | | |
| Av. Strength (psi) | 3,515 | 3,348 | 3,677 | 498 | 527 | ** | ** | 3,132 | 3,092 | ** | 536 | 456 |
| St. Deviation (psi) | 58 | 15 | 275 | 10 | 43 | ** | ** | 5 | 22 | ** | 0 | 1 |
| Coeff. of Variation | 0.017 | 0.004 | 0.075 | 0.019 | 0.081 | ** | ** | 0.002 | 0.007 | ** | 0.001 | 0.003 |

**No data

## TABLE 4-12. RESULTS OF 7-DAY TESTS ON CASTINGS 7, 8, AND 9

| | Cylinders | | | Beams | | | | Maturity Strengths Listed by Basis of Maturity Curve | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Cylinders | | | Beams | |
| | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors | Pullout LOK-Test | Pen Resist Windsor Probe | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors |
| Casting 4 | | | | | | | | | | | | |
| Av. Strength (psi) | 6,483 | 6,123 | 6,777 | 782 | 653 | 5,669 | 3,650 | 6,001 | 5,433 | 6,240 | 748 | 722 |
| St. Deviation (psi) | 137 | 96 | 156 | 50 | 24 | 911 | 50 | 1 | 27 | 0 | 0 | 2 |
| Coeff. of Variation | 0.021 | 0.016 | 0.023 | 0.064 | 0.037 | 0.161 | 0.014 | 0 | 0.005 | 0 | 0 | 0.002 |
| Casting 5 | | | | | | | | | | | | |
| Av. Strength (psi) | 1,648 | 1,476 | 1,510 | 365 | 433 | 1,030 | 800 | 1,545 | 1,329 | 1,483 | 371 | 380 |
| St. Deviation (psi) | 125 | 12 | 75 | 22 | 6 | 316 | 100 | 1 | 6 | 8 | 0 | 0 |
| Coeff. of Variation | 0.076 | 0.008 | 0.050 | 0.059 | 0.014 | 0.306 | 0.125 | 0 | 0.005 | 0 | 0 | 0.002 |
| Casting 6 | | | | | | | | | | | | |
| Av. Strength (psi) | 4,551 | 4,272 | 4,716 | 668 | 531 | ** | ** | 4,238 | 4,331 | ** | 604 | 548 |
| St. Deviation (psi) | 45 | 69 | 133 | 42 | 18 | ** | ** | 2 | 3 | ** | 0 | 1 |
| Coeff. of Variation | 0.010 | 0.016 | 0.028 | 0.062 | 0.034 | ** | ** | 0 | 0.001 | ** | 0 | 0.002 |

**No data

## TABLE 4-13. RESULTS OF 14-DAY TESTS ON CASTINGS 7, 8, AND 9

| | Cylinders | | | Beams | | | | Maturity Strengths Listed by Basis of Maturity Curve | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Cylinders | | | Beams | |
| | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors | Pullout LOK-Test | Pen Resist Windsor Probe | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors |
| Casting 4 | | | | | | | | | | | | |
| Av. Strength (psi) | 7,606 | 6,945 | 7,580 | 810 | 900 | ** | ** | 7,355 | 6,597 | 7,334 | 822 | 839 |
| St. Deviation (psi) | 34 | 281 | 382 | 8 | 8 | ** | ** | 0 | 32 | 0 | 0 | 1 |
| Coeff. of Variation | 0.005 | 0.040 | 0.050 | 0.010 | 0.009 | ** | ** | 0 | 0.005 | 0 | 0 | 0.001 |
| Casting 5 | | | | | | | | | | | | |
| Av. Strength (psi) | 1,788 | 1,708 | 1,704 | 417 | 455 | ** | ** | 1,886 | 1,618 | 1,725 | 433 | 455 |
| St. Deviation (psi) | 84 | 19 | 124 | 5 | 5 | ** | ** | 0 | 7 | 5 | 0 | 1 |
| Coeff. of Variation | 0.047 | 0.011 | 0.073 | 0.011 | 0.011 | ** | ** | 0 | 0.005 | 0.003 | 0 | 0.001 |
| Casting 6 | | | | | | | | | | | | |
| Av. Strength (psi) | 5,022 | 5,982 | 5,138 | 655 | 646 | ** | ** | 5,047 | 5,123 | ** | 653 | 614 |
| St. Deviation (psi) | 229 | 1,575 | 61 | 14 | 42 | ** | ** | 1 | 48 | ** | 0 | 1 |
| Coeff. of Variation | 0.046 | 0.263 | 0.012 | 0.021 | 0.065 | ** | ** | 0 | 0.009 | ** | 0 | 0.002 |

**No data

### TABLE 4-14. RESULTS OF 28-DAY TESTS ON CASTINGS 7, 8, AND 9

| | Cylinders | | | Beams | | | | Maturity Strengths Listed by Basis of Maturity Curve: Cylinders | | | Maturity Strengths Listed by Basis of Maturity Curve: Beams | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors | Pullout LOK-Test | Pen Resist Windsor Probe | Cured in Wetroom | Cured Outdoors | Cores | Cured in Wetroom | Cured Outdoors |
| Casting 4 | | | | | | | | | | | | |
| Av. Strength (psi) | 8,119 | 7,197 | 8,004 | 893 | 943 | ** | 4,400 | 8,753 | 7,915 | 8,414 | 899 | 967 |
| St. Deviation (psi) | 75 | 232 | 96 | 5 | 29 | ** | 283 | 0 | 47 | 0 | 0 | 1 |
| Coeff. of Variation | 0.009 | 0.032 | 0.012 | 0.005 | 0.030 | ** | 0.067 | 0 | 0.006 | 0 | 0 | 0.001 |
| Casting 5 | | | | | | | | | | | | |
| Av. Strength (psi) | 2,176 | 1,731 | 1,993 | 486 | 496 | 1,867 | ** | 2,230 | 1,925 | 1,988 | 470 | 536 |
| St. Deviation (psi) | 77 | 134 | 204 | 35 | * | 134 | ** | 0 | 1 | 5 | 0 | 0 |
| Coeff. of Variation | 0.035 | 0.077 | 0.102 | 0.071 | * | 0.072 | ** | 0 | 0.001 | 0.003 | 0 | 0 |
| Casting 6 | | | | | | | | | | | | |
| Av. Strength (psi) | 5,549 | 5,321 | 5,430 | 675 | 654 | ** | ** | 5,853 | 6,044 | ** | 703 | 692 |
| St. Deviation (psi) | 138 | 174 | * | 39 | 8 | ** | ** | 0 | 133 | ** | 0 | 8 |
| Coeff. of Variation | 0.025 | 0.033 | * | 0.058 | 0.012 | ** | ** | 0 | 0.022 | ** | 0 | 0.011 |

**No data *Only one reading

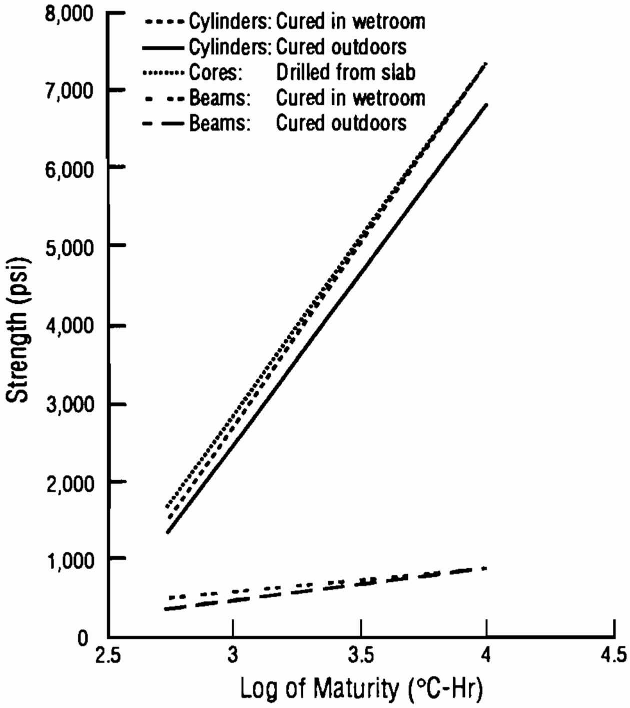


**Fig 4-5. Maturity curves for casting 7.**

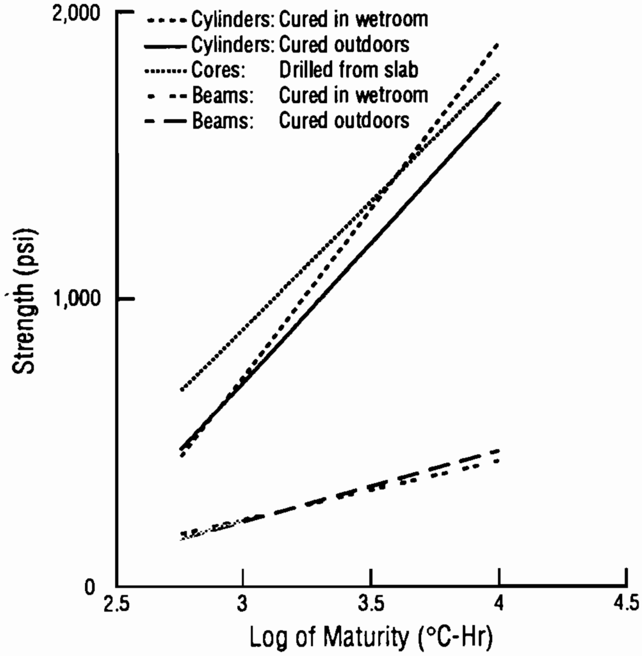


**Fig 4-6. Maturity curves for casting 8.**

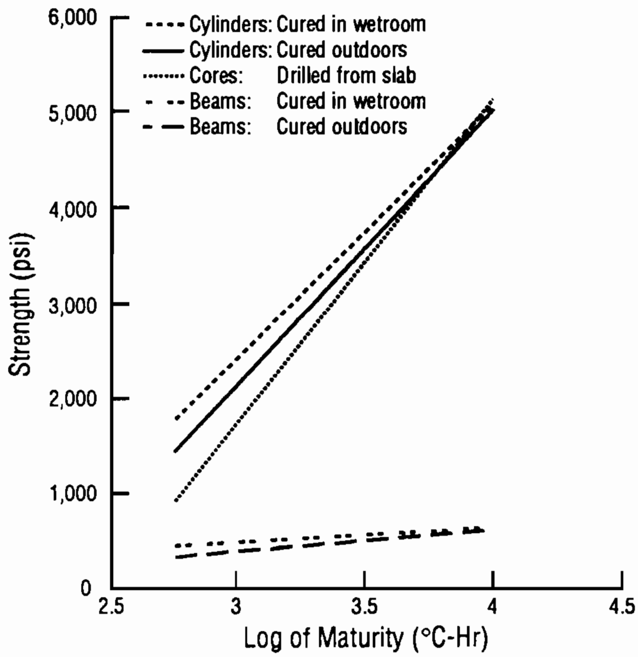


**Fig 4-7. Maturity curves for casting 9.**

**TABLE 4-15. RESULTS OF TESTS ON CASTING 10**

| | 73 °F Cure | | | 110 °F Cure | | |
|---|---|---|---|---|---|---|
| Age (hrs) | Maturity (°C/hr) | Predicted Strength (psi) | Actual Strength (psi) | Maturity (°C/hr) | Predicted Strength (psi) | Actual Strength (psi) |
| 20 | 724 | 1,000 | 760 | 965 | 1,300 | 1,480 |
| 69 | 2,373 | 2,340 | 2,300 | 2,612 | 2,450 | 2,350 |

**TABLE 4-16. RESULTS OF TESTS ON CASTING 11**

| | 73 °F Cure | | | 110 °F Cure | | |
|---|---|---|---|---|---|---|
| Age (hrs) | Maturity (°C/hr) | Predicted Strength (psi) | Actual Strength (psi) | Maturity (°C/hr) | Predicted Strength (psi) | Actual Strength (psi) |
| 21 | 748 | 1,030 | 710 | 1,156 | 1,500 | 1,730 |
| 45 | 1,554 | 1,840 | 1,800 | 1,964 | 2,120 | 2,110 |
| 69 | 2,324 | 2,310 | 2,410 | 2,755 | 2,510 | 2,390 |

# CHAPTER 5. DATA ANALYSIS

## 5.1 INTRODUCTION

Results of the tests conducted in Phase I and II were tabulated and calculated in three ways. Phase III is supplemental to the rest of the investigation and was considered separately.

In the first analysis, the mean value, standard deviation, and coefficient of variation (COV) were calculated for the results of each test method on each test day. These data were presented in Chapter 4. In addition, the data are also presented graphically for discussion in this chapter.

In the second analysis, mean values of each test method at each test age were compared with corresponding mean values at the same age for cylinders and beams. For example, the mean strength found by the Lok-Test pullout method at 3 days on casting 1 was compared to the strength of wetroom-cured cylinders made during the same casting and tested at the same age. This approach allowed convenient comparison of any method with any other method, and it also allowed comparisons between castings which used different concrete mixes. Strengths by the various testing methods were compared with those of cylinders cured in the wetroom, cylinders cured outdoors, beams cured in the wetroom, and beams cured outdoors. The ratios were tabulated and presented graphically.

In the third analysis, results from the first two analyses were examined for notable trends and tendencies with regard to the usefulness of particular tests in the concrete construction industry.

## 5.2 ANALYSIS OF PHASE I RESULTS

The results of the concrete tests corresponding to the first analysis performed on Phase I castings were given in Chapter 4, Tables 4-1 through 4-4. In the second analysis, strengths determined by each test method were first compared to strengths by two well-known standard methods: wetroom-cured cylinders and wetroom-cured beams. Ratios of each test strength to these standard strengths were tabulated at each test age. The tables of strength ratios can be found in Appendix A. The tables contain data that were used to develop graphs showing how the ratios varied for each test at different ages. These graphs are presented in Appendix A, and selected individual graphs are also included in this chapter as they are discussed.

In the third analysis, trends and indicators were found in the results of the previous two analyses. Results of the third analysis of Phase I data are discussed below under five headings: pullout, rebound number, maturity method, penetration resistance, and flexural beams.

### *5.2.1 PULLOUT*

Figures 5-1 and 5-2 illustrate that the CAPO pullout test results do not agree with those of standard wetroom-cured cylinder compression tests as consistently as do the Lok-Test pullout results, especially at early ages. The CAPO test results are generally higher than Lok-Test results, especially at 1 day. At 3 and 7 days, Lok-Tests still tend to be in closer agreement with the cylinder tests, and the Lok-Test results are not scattered as widely as are the results of the CAPO test. Finally, at 28 days, the ratios of CAPO test results to cylinder test results and the ratios of Lok-Test results to cylinder test results both converge to a range between 0.80 to 1.20.

Figures 5-3 through 5-5 reveal that the coefficient of variation for the CAPO test is somewhat higher than that of the Lok-Test, and both, generally, are considerably higher than that of the standard cylinder. In Fig 5-3 it is

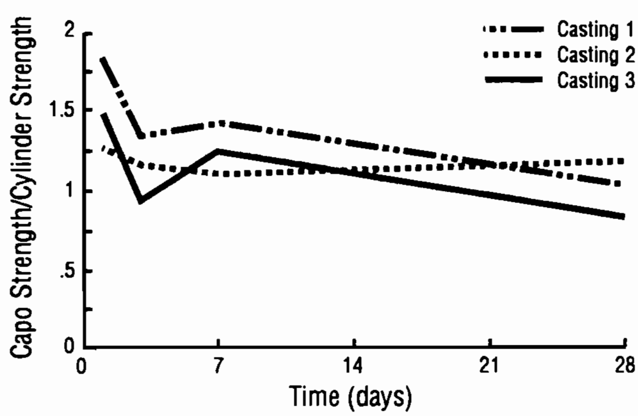


**Fig 5-1. Ratio of "CAPO test" strength to strength of wetroom-cured cylinders for castings 1, 2, and 3.**

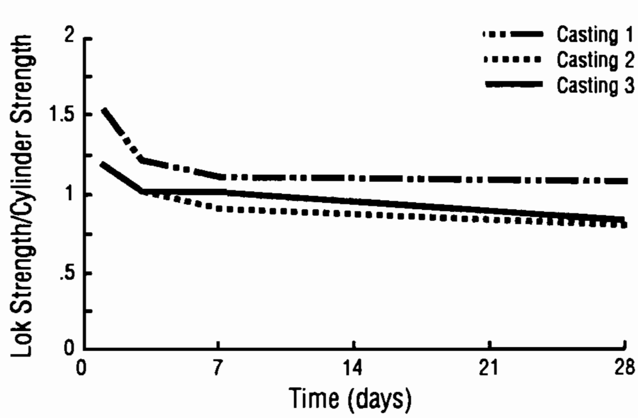


**Fig 5-2. Ratio of "Lok-Test" pullout strength to strength of wetroom-cured cylinders of castings 1, 2, and 3.**

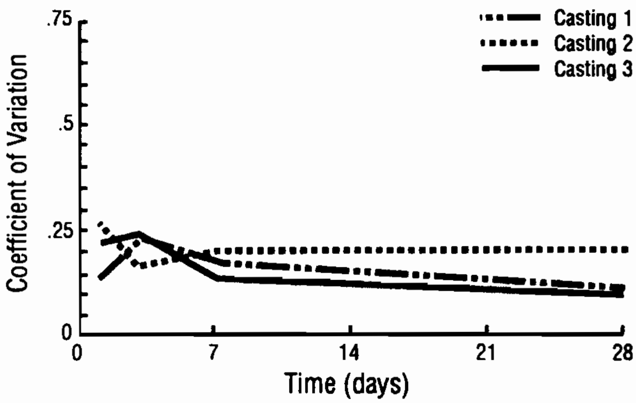


**Fig 5-3. Coefficient of variation of "CAPO test" pullout strengths for castings 1, 2, and 3.**

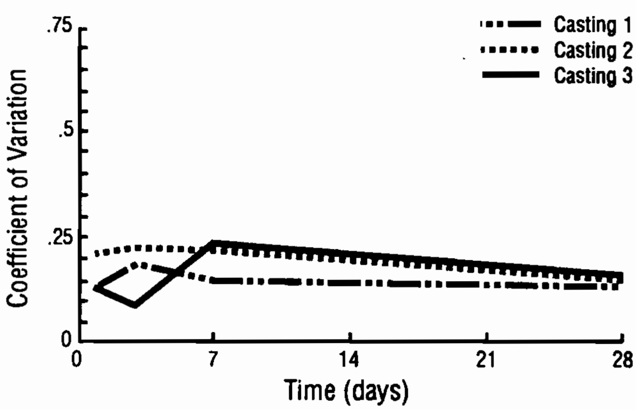


**Fig 5-4. Coefficient of variation of "Lok-Test" pullout strengths for castings 1, 2, and 3.**

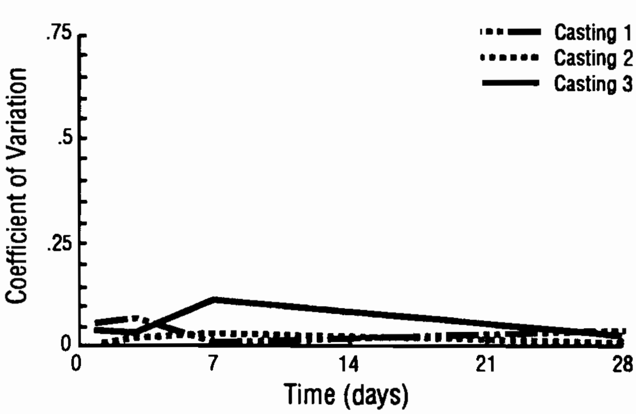


**Fig 5-5. Coefficient of variation of wetroom-cured cylinder strengths for castings 1, 2, and 3.**

apparent that the coefficient of variation of the CAPO test was always above 10 percent, and at early ages it was frequently in the range of 20 to 30 percent. This is a sign of wide variations in the test results. At later ages, there was a general trend toward lower coefficients, although in one case the coefficient remained around 20 percent. For the Lok-Test, Fig 5-4 shows that the coefficient of variation was, at early ages, mostly between 10 and 25 percent. Later, it settled at about 15 percent. The COV of the cylinder compression test, in contrast, reached as high as 10 percent just once (at 7 days), and at 28 days was between 2 and 4 percent, as shown in Fig 5-5.

### *5.2.2 REBOUND NUMBER*

Figures 5-6 and 5-7 show no consistent trends when the results of the rebound number test are compared with cylinder strength or beam modulus of rupture. Figure 5-6 shows that the strengths determined by the rebound hammer begin at levels from 30 to 70 percent above their companion cylinder strengths, but at 28 days they may be 60 percent lower or 15 percent higher. Compared with beam strengths, as in Fig 5-7, the rebound hammer strengths are again scattered. At early ages, they range between three and six times the beam strength. At later ages, they range from two to eight times the beam strength. With such a wide range, it would be difficult to assess the beam strength based on knowledge of the rebound strength.

Figure 5-8 compared with Figs 5-5 and 5-9 shows that the rebound number is consistently far more variable than either the wetroom cylinder strength or the wetroom beam strength. The rebound number coefficient of variation was always above 15 percent and once was over 50 percent. On the other hand, cylinders and beams cured in the wetroom were much more consistent between tests and between slabs. As was previously pointed out in Fig 5-5, cylinder tests had coefficients of variation that were always below 10 percent and usually less than 5 percent. The beam COV was always below 15 and several times reached below 5 percent, as shown in Fig 5-9.

### *5.2.3 MATURITY METHOD*

The maturity method is based on the principle that, for a given concrete mix design, the strength is primarily a function of the curing time and temperature. This principle would indicate that the three slabs of Phase I, which were all made of concrete with the same mix design, should all have the same strength versus maturity relationship. To test this hypothesis, the strength and maturity values from these three slabs were used to develop the maturity curves which were presented in Fig 4-1. These curves appear to be quite similar, and two methods of analysis confirm this impression.

The first way the similarity was confirmed was with a statistical method. Appendix D contains the details of a statistical F-test of variances, which demonstrate that there is a 95 percent probability that there are no significant differences among the three data groups. Thus, a curve suitable for one slab is considered suitable for all three. The strength versus maturity relationship is the

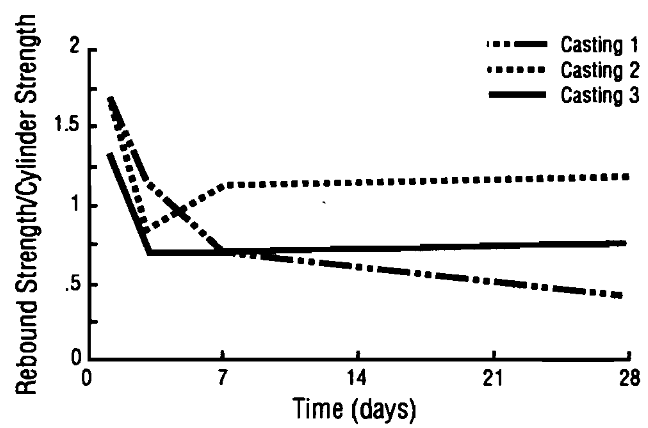


**Fig 5-6. Ratio of rebound hammer strengths to strengths of wetroom-cured cylinders for castings 1, 2, and 3.**

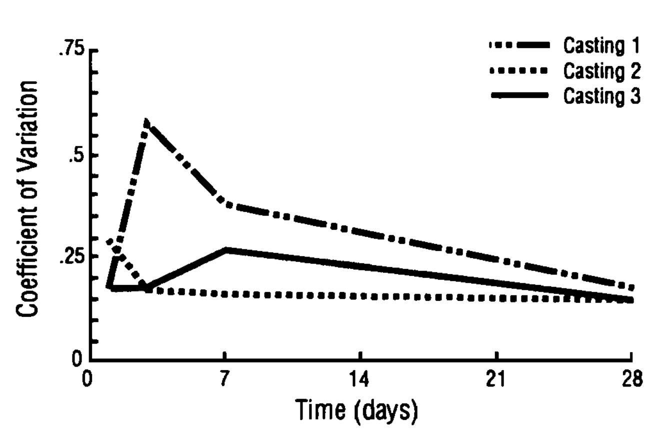


**Fig 5-8. Coefficient of variation of rebound number strengths for castings 1, 2, and 3.**

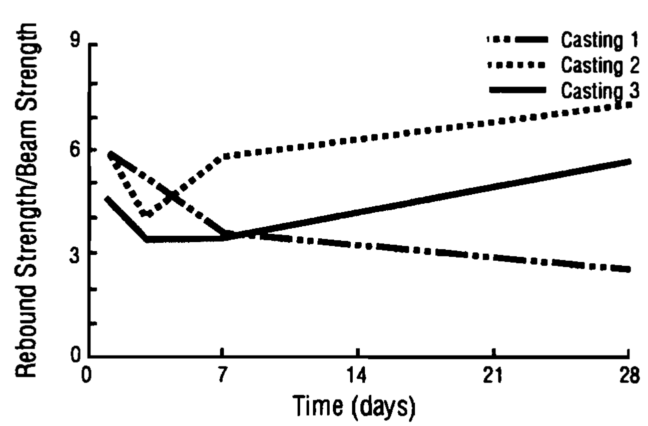


**Fig 5-7. Ratio of rebound number strength to flexural strength of wetroom-cured beams for castings 1, 2, and 3.**

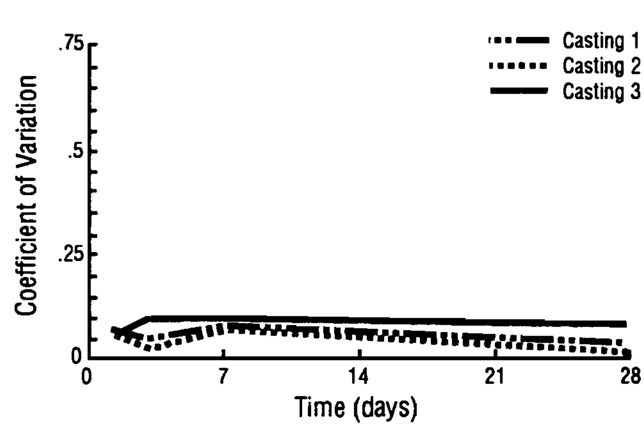


**Fig 5-9. Coefficient of variation of flexural strength of wetroom-cured beams for castings 1, 2, and 3.**

same for all three slabs made from the same mix design, just as the maturity theory predicts.

The second method of confirming the similarity was a more empirical approach in which maturity predictions were compared. In Fig 4-1 three maturity curves were shown for three separate batches of the same mix design produced over a period of two months. The equations of these curves for castings 1, 2 and 3 are listed here:

(Casting 1) y = 2,090 (x) – 4,766
(Casting 2) y = 1,868 (x) – 4,157
(Casting 3) y = 2,182 (x) – 4,954

where x is the log of maturity in degree C-hours and y is strength in psi. In Table 5-1, these three equations were used to calculate strengths for two representative maturity values. Typical values would be 1,000 degree C-hours in the first day or two after casting and 20,000 degree C-hours at 25 to 30 days. The table shows how the strength predictions vary depending on which equation is used. There is a 9 percent difference in the strength predicted between the highest value and the lowest value at the early age. At the later age, there is a 13 percent difference between the highest and lowest values. Thus, an empirical approach shows that the strength versus maturity relationship is reasonably close to being the same for all three slabs, verifying the maturity theory.

Maturity method results were consistently very close to the compressive cylinder strengths, as can be seen from Fig 5-10; and from Fig 5-11 it is apparent they were also very consistent, having very low coefficients of variation. The general pattern evident from Fig 5-10 is that the maturity method strengths at 1 day are somewhere around 25 percent higher than the cylinder strengths, but that by 3 days and later they fall within about 10 percent of the cylinder strengths. The maturity method coefficient of variation, shown in Fig 5-11, is always below 1 percent. This is far lower than the cylinder COV's shown in Fig 5-5.

**TABLE 5-1. COMPARISON OF STRENGTH PREDICTIONS FROM VARIOUS MATURITY EQUATIONS**

| Casting Number | Maturity Equation Based on Wetroom Cylinders | Strength by Equation at 1,000 °C – Hr | Strength by Equation at 20,000 °C – Hr |
|---|---|---|---|
| 1 | y = 2090(log x) - 4766 | 1,500 | 4,220 |
| 2 | y = 1868(log x) - 4157 | 1,450 | 3,880 |
| 3 | y = 2182(log x) - 4954 | 1,590 | 4,430 |

### *5.2.4 PENETRATION RESISTANCE*

The Windsor probe penetration test was a better and less variable predictor of cylinder strength at later ages than it was at early ages. As shown in Fig 5-12, the relationship with the cylinder strength started out with no clear tendencies or trends, but at 7 days it converged on unity. At 28 days, the relationship remained at about unity. Figure 5-13 shows that the coefficient of variation, although somewhat unpredictable, does generally decrease with age. At less than 7 days it could be extremely variable, with values over 65 percent, but by 28 days the COV falls to around 5 percent.

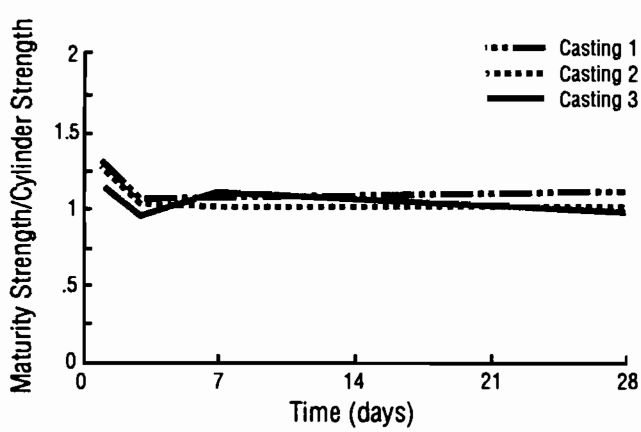


**Fig 5-10. Ratio of maturity method strength to strength of wetroom-cured cylinders for castings 1, 2, and 3.**

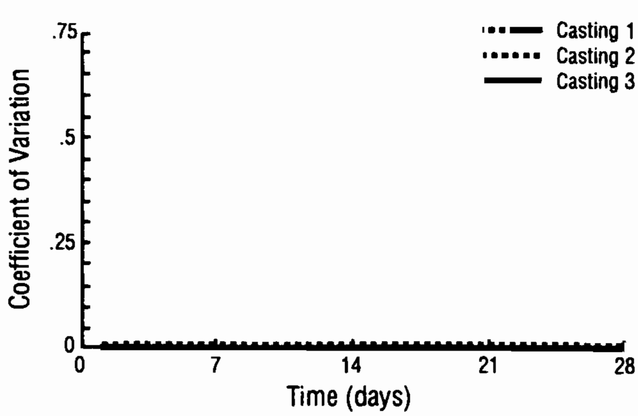


**Fig 5-11. Coefficient of variation of maturity method strengths for castings 1, 2, and 3.**

### *5.2.5 FLEXURAL BEAMS*

Comparisons made with the beam test revealed that there was little correlation between the modulus of rupture and any other test. Figure 5-14 is typical of the normalizations onto beam modulus: a lower ratio at early ages and a higher ratio at later ages. This figure is for the Lok-Test pullout test, but others follow this pattern. In Fig 5-15 it can be seen that the COV of the beam test is usually below 10 percent.

## 5.3 TEST REGIMEN MODIFICATIONS AFTER PHASE I

Based on the results of Phase I, the testing regimen developed for Phase I was modified before Phase II work proceeded. Rebound and CAPO testing were discontinued and maturity testing was expanded, as explained in the following paragraphs.

### *5.3.1 REBOUND AND CAPO*

The rebound number test was the most variable test and had the poorest comparison with cylinders and beams, so it was discontinued. Also, of the two pullout tests that were considered, the CAPO test showed poorer correlation with beams or cylinders and greater within-test variability. Therefore the CAPO test, too, was discontinued, and the Lok-Test was the only pullout test considered in further investigations.

### *5.3.2 MATURITY*

In Section 5.2 it was noted that the maturity method had a very low coefficient of variation at all ages and showed good agreement with the cylinder strengths, especially after 1 day. Based on this observation, it was determined that additional testing was warranted, and the scope of the maturity method investigation was significantly expanded.

In Phase I, maturity curves were developed using the standards mentioned in Chapter 2, which require that cylinders be cured in controlled environments. In Phase II, the curing conditions and the specimens tested would be systematically varied to see what effect this would have on the use of the maturity method. Curves would be developed using the same general principles, but with five variations. First, as a baseline, the maturity curves would be developed just as they were in Phase I, using cylinders stored in the controlled environment of the wetroom. Second, curves would be developed using cylinders stored in the field until testing, exposed to the weather just as the slab was exposed. Third, curves would be developed based on maturities and strengths from cylindrical cores drilled from the slab and tested immediately after drilling. Fourth, curves would be developed using wetroom-cured beams rather than cylinders. Fifth, curves would be developed using strengths from beams cured in the field.

## 5.4 ANALYSIS OF PHASE II RESULTS

The results of the concrete tests performed on the first three castings of Phase II, castings 4, 5, and 6, were given in Chapter 4, Tables 4-5 through 4-9. The results for Phase II castings 7, 8, and 9 were given in Chapter 4, Tables 4-10 through 4-14. This was the first analysis of Phase II results, which included calculation of the mean, standard deviation, and coefficient of variation (COV) for each set of companion tests at each age.

In the second analysis, strengths determined by each test method were compared first to strengths of wetroom-cured cylinders, outdoor-cured cylinders, wetroom-cured beams, and outdoor-cured beams. Ratios of strength to standard strength were tabulated for each test age. These tables of strength ratios can be found in Appendix B for castings 4, 5, and 6, and in Appendix C for castings 7, 8, and 9. The tables contain data that were used to develop graphs showing how the ratios varied for each test at different ages. These graphs are also presented in Appendices B and C, and selected individual graphs are included in this chapter as they are discussed.

In the third analysis, trends and indicators were found in the results of the previous two analyses. Results

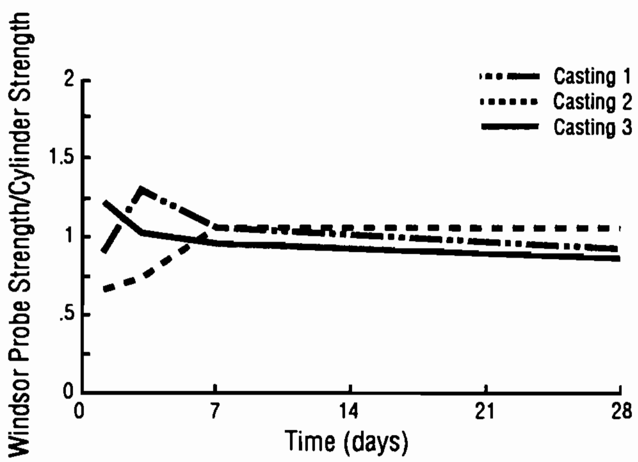


**Fig 5-12. Ratio of Windsor probe penetration resistance strength to strength of wetroom-cured cylinders for castings 1, 2, and 3.**

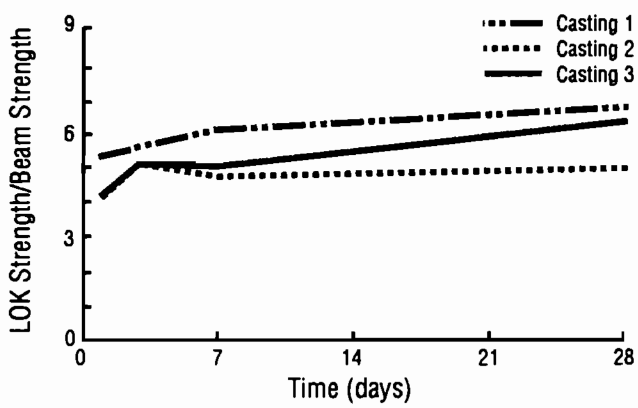


**Fig 5-14. Ratio of "Lok-Test" pullout strength to strength of wetroom-cured flexural beams for castings 1, 2, and 3.**

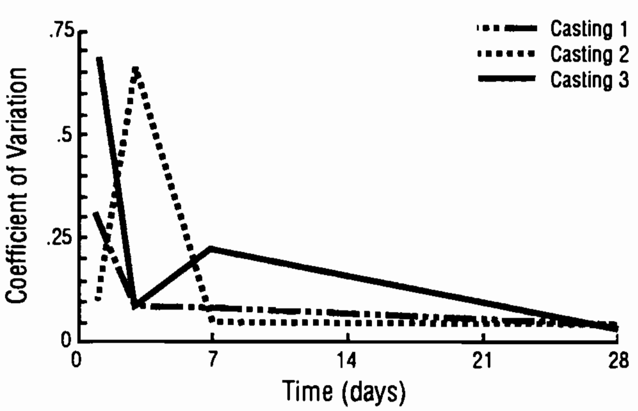


**Fig 5-13. Coefficient of variation of Windsor probe penetration resistance strengths for castings 1, 2, and 3.**

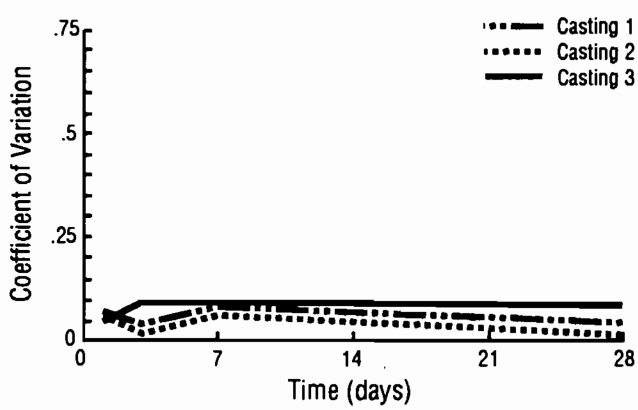


**Fig 5-15. Coefficient of variation of wetroom-cured flexural beam strengths for castings 1, 2, and 3.**

of the third analysis of this Phase II data are discussed below under five headings: outdoor-cured cylinders, cores, maturity, pullout, and penetration resistance.

### 5.4.1 OUTDOOR-CURED CYLINDERS AND CORES

Figure 5-16 shows the ratio of the strengths of outdoor cylinders to the strengths of wetroom cylinders. There are considerable differences from unity, especially at early ages, but also at 28 days. Note that the strengths for the outdoor cylinders for casting 5 were slightly higher than those of the wetroom cylinders at early ages but were lower at later ages. This trend is an expected consequence of high-temperature field-curing at early ages.

Figure 5-17 shows that castings 7, 8, and 9 indicated no strong trends in the relationship between outdoor cylinders and wetroom cylinders. In some castings, the relationship is closer to unity at earlier ages, while in other castings, the relationship is closer to unity at later ages.

Figures 5-18 and 5-19 show that there is a significant variability at early ages for compression tests on cores. These cores were drilled, prepared, and tested within several hours on the same day. Comparing Fig 5-16 with Fig 5-18 and Fig 5-17 with Fig 5-19, it is apparent that the outdoor cylinders are not following the same trends as the cores. Strengths from core tests and field cure tests both vary from the strength of wetroom cylinders, but they do not show ratios similar to that of wetroom cylinder strength. In other words, they are different both from wetroom cylinders and from each other.

### 5.4.2 MATURITY

Figures 5-20, 5-21, 5-22 and 5-23 show that the various maturity curves are all useful for predicting strengths

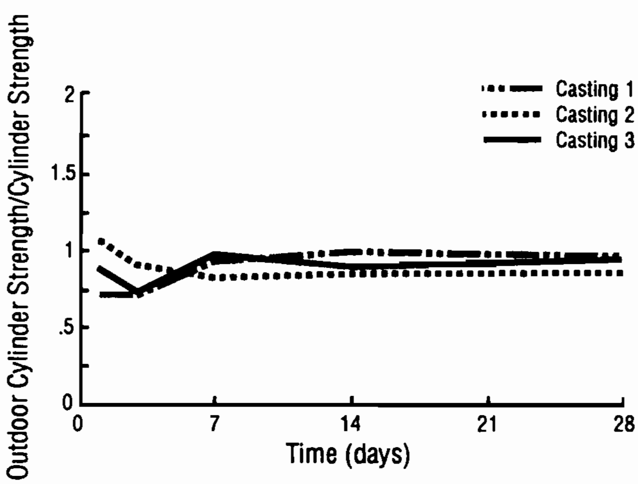


**Fig 5-16. Ratio of outdoor-cured cylinder strength to strength of wetroom-cured cylinders for castings 4, 5, and 6.**

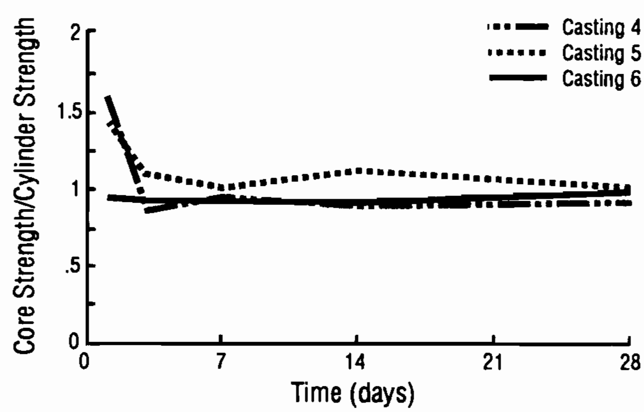


**Fig 5-18. Ratio of strength of cores drilled from slabs to strength of wetroom-cured cylinders for castings 4, 5, and 6.**

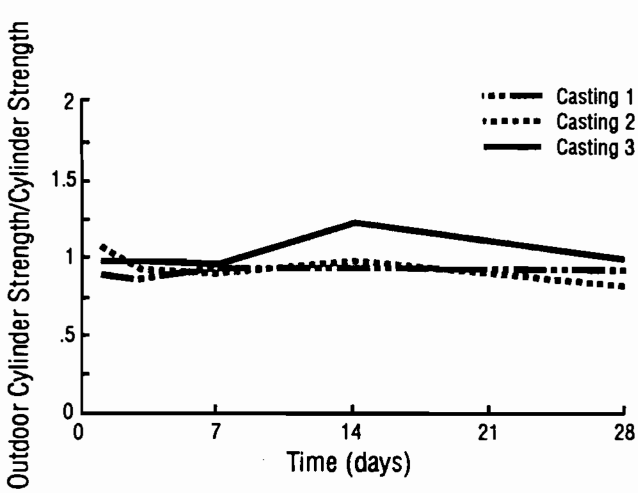


**Fig 5-17. Ratio of outdoor-cured cylinder strength to strength of wetroom-cured cylinders for castings 7, 8, and 9.**

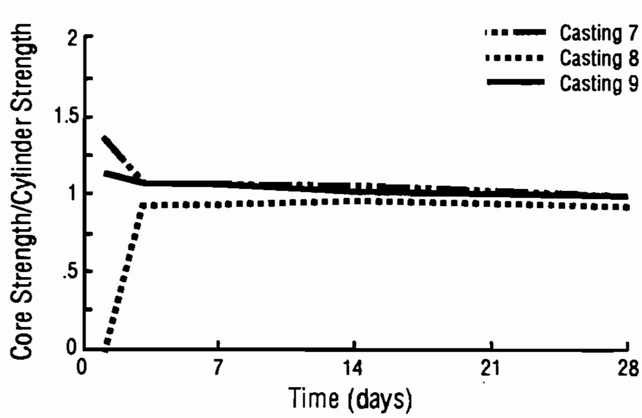


**Fig 5-19. Ratio of strength of cores drilled from slabs to strength of wetroom-cured cylinders for castings 7, 8, and 9.**

of their respective specimens. That is, in Fig 5-20 it is evident that if wetroom-cured cylinders are used to develop the maturity curve, then the curve can give very close estimates of wetroom-cured cylinder strength after about 1 day. Similarly, in Fig 5-21, if outdoor-cured cylinders are used to develop the maturity curve, then the curve can give close estimates of the strength of outdoor-cured cylinders. However, the estimates are not quite as close as those in the previous example. Similar results are shown in Figs 5-22 and 5-23 for flexural beams stored in the wetroom and outdoors, respectively.

In addition, a statistical evaluation shows that the maturity curves developed were almost identical regardless of how the specimens were cured. Curves based on strengths of wetroom-cured cylinders, outdoor-cured cylinders, and cores drilled from the slab all passed the F-test and were thus shown to be the same curve. These calculations are summarized in Appendix D. The results are not surprising, considering the similarity of the curves which were shown in Figs 4-2 through 4-7.

Typically, the curves used to calculate the strengths for Figs 5-20 through 5-23 predict considerably higher than actual strengths at 1 day and slightly lower than actual strengths at 3 days, but they are very close at 7, 14, and 28 days. This is a consistent trend in all the maturity curve work on this project, and the inaccuracy at early ages is related to the inaccuracy of the semi-logarithmic curve-fitting technique employed. This is explained in Section 5.5, which contains a discussion of curve-fitting techniques for the maturity method.

Figures 5-24 and 5-25 show typical coefficients of variation for maturity method testing. As in Phase I, they are very low. Even at early ages, the variation is far less than 3 percent, which is considered very good in cylinder testing at 28 days. Figure 5-24 shows the COV for the maturity method based on beams stored outdoors next to

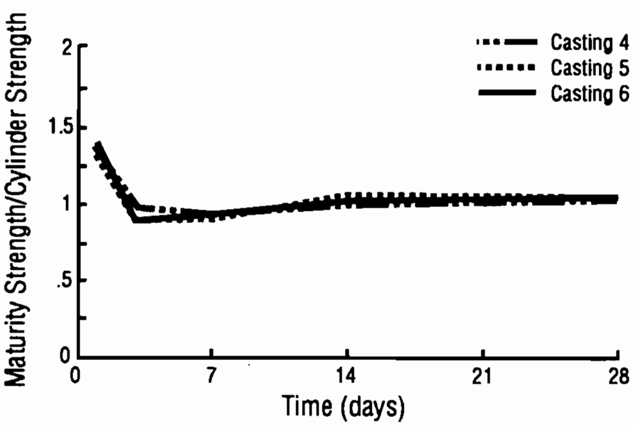


**Fig 5-20. Ratio of strength by maturity method to strength of wetroom-cured cylinders for castings 4, 5, and 6. Maturity curve based on wetroom-cured cylinders.**

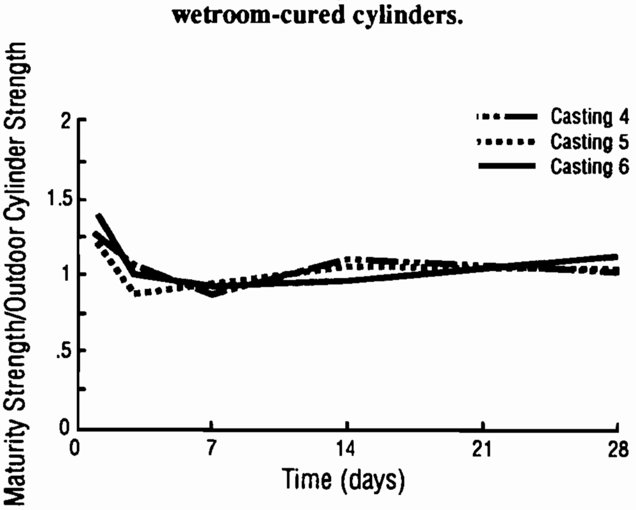


**Fig 5-21. Ratio of strength by maturity method to strength of outdoor-cured cylinders for castings 4, 5, and 6. Maturity curve based on outdoor-cured cylinders.**

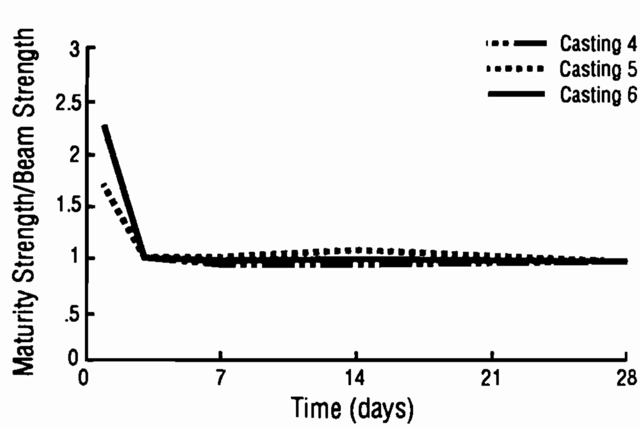


**Fig 5-22. Ratio of strength by maturity method to strength of wetroom-cured flexural beams for castings 4, 5, and 6. Maturity curve based on wetroom-cured beams.**

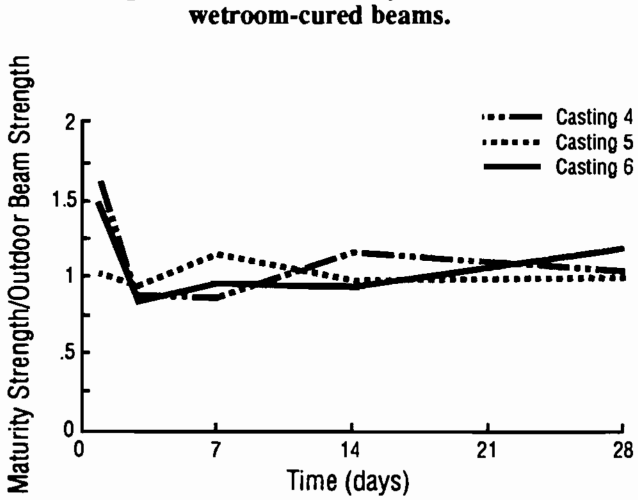


**Fig 5-23. Ratio of strength by maturity method to strength of outdoor-cured beams for castings 4, 5, and 6. Maturity curve based on outdoor-cured beams.**

the slab, and it peaks at about 1 percent. The same is true of the COV for the maturity method based on cores, shown in Fig 5-25. No equation was developed for casting 9 because of maturity data collection equipment failure.

### *5.4.3 PULLOUT AND PENETRATION RESISTANCE*

Comparing Figs 5-24 and 5-25 (the two figures of maturity method data) with Figs 5-26 and 5-27, one relationship is clear: there is a significant difference between the low-variability maturity method testing and the highly-variable Lok-Test pullout method. Lok-Test results are consistently above 10 percent and sometimes as high as 30 percent. At most test ages, casting 4 has the worst agreement of the three sets of data shown in Fig 5-26, but at one age it has the best. The other two slabs were both richer mixes, but the slab of casting 5 had 3/4-inch rounded river rock for coarse aggregate while the slab of casting 6 used crushed limestone aggregate. Patterns are not readily apparent. The Lok-Test variability increases seem to be more closely related to chance than to mix proportions or materials used. The extremely high variability of the slab of casting 8 is related to its low strength; the 1-day strengths were at or below 500 psi, so a relatively small absolute value of variation caused a very large coefficient of variation—hence the COV of about 100 percent in Fig 5-27.

Similarly, comparing Figs 5-24 and 5-25 with Fig 5-28 shows a higher variability for the Windsor probe penetration resistance method than for the maturity method. While the maturity method coefficient of variation is below 1 percent, the Windsor probe tests are frequently in the range of 10 percent, and they occasionally reach 100 percent. As in Phase I, the highest variabilities were

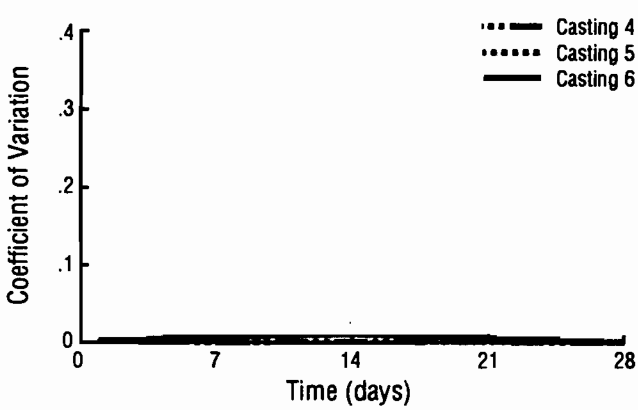


**Fig 5-24. Coefficient of variation of maturity method strengths for castings 4, 5, and 6. Maturity curve based on outdoor-cured beams.**

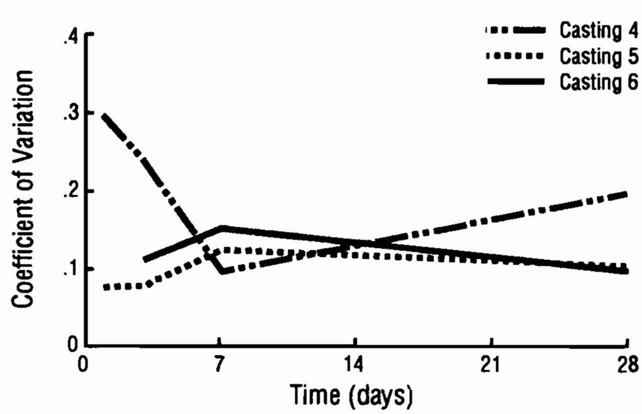


**Fig 5-26. Coefficient of variation of "Lok-Test" pullout strengths for castings 4, 5, and 6.**

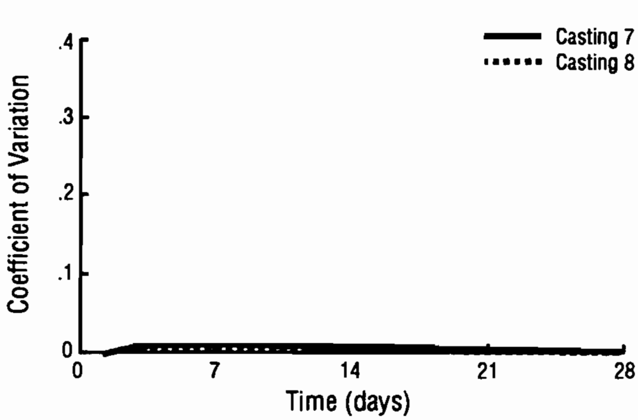


**Fig 5-25. Coefficient of variation of maturity method strengths for castings 7 and 8. Maturity curves based on cores drilled from slabs.**

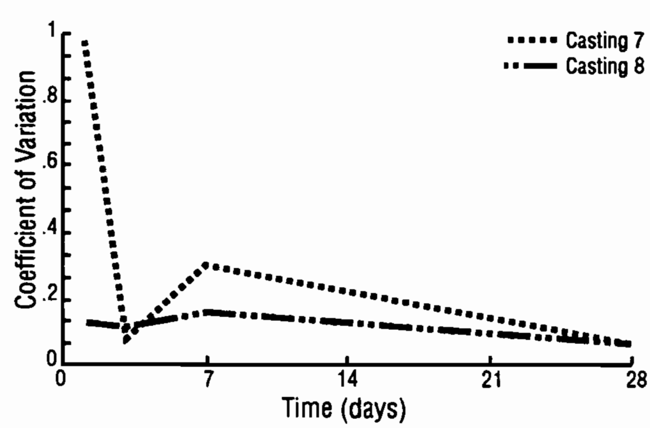


**Fig 5-27. Coefficient of variation of "Lok-Test" pullout strengths for castings 7 and 8.**

found at early ages, and by 28 days they were consistently below 10 percent.

## 5.5 ANALYSIS OF MATURITY CALCULATION PROCEDURES

Following the completion of Phase II testing and data calculation, an alternative maturity calculation procedure was investigated, with the aim of increasing the accuracy of the maturity method at ages below 3 days. This procedure, which was mentioned in Section 2.5.2 and is presented in this section, was found to improve the performance of the maturity method at these early ages. The procedure involves development and use of the hyperbolic strength prediction equation. The remainder of this section is divided into two parts. The first is a discussion of the procedure used in most of the calculations done for this research program, which is based on a logarithmic strength prediction equation. The second part is an explanation of the hyperbolic strength prediction procedure.

### *5.5.1 LOGARITHMIC STRENGTH PREDICTION EQUATION*

The ASTM Standard C1074, Estimating Strength by the Maturity Method,[7] calls for the compressive strength of the concrete to be plotted as a function of the maturity. This typically looks like the graph in Fig 5-29, with the strength on the vertical axis and the maturity on the horizontal axis. Once it is plotted, a best-fit curve is then determined for the data. One convenient way to fit a curve to the data is to plot the logarithm of the maturity rather than the maturity and then to draw a straight line through the data points. Plowman[72] discovered in the 1950's that the data points will usually line up quite well and that fitting a curve is relatively simple. In this testing program, the maturity method was employed with the aid of such a "logarithmic best-fit curve" procedure. To demonstrate, in Fig 5-30 the data points of Fig 5-29 have been plotted on a graph where strength is again on the vertical y-axis, but this time the logarithm of the maturity is on the horizontal x-axis. The points line up quite well, and a best-fit line has been drawn through them. The equation of that line is

$$y = 3{,}052x - 7{,}733$$

where y is the strength and x is the logarithm of the maturity. For any given maturity, the strength can be quickly estimated using this equation.

However, analysis of the data has shown that the logarithmic best-fit procedure shows some deficiencies at very early ages. Several of the figures in this chapter show that this method typically overestimates the strength of the concrete at 1 day. In Fig 5-10, for example, the maturity method strength was within 10 or 15 percent of the cylinder compressive strength at ages 3, 7,

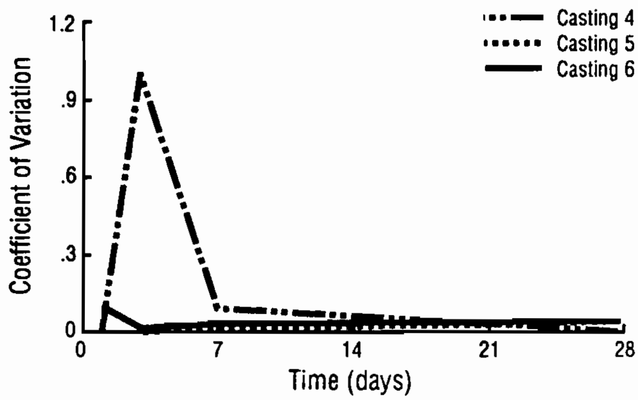


**Fig 5-28. Coefficient of variation of Windsor probe penetration resistance strength for castings 4, 5, and 6.**

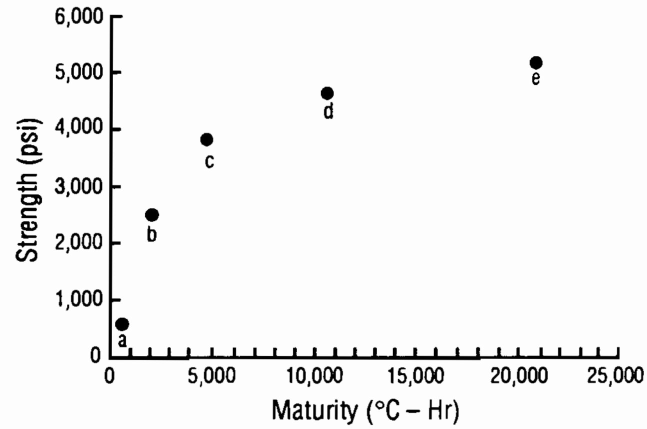


**Fig 5-29. Relationship between maturity and strength of wetroom-cured cylinders for casting 4.**

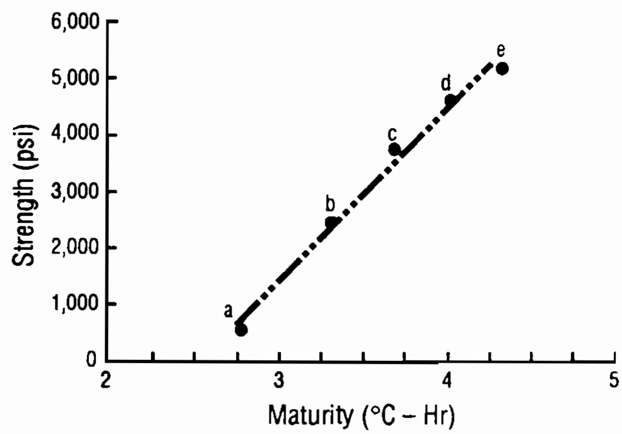


**Fig 5-30. Relationship between logarithm of maturity and strength of wetroom-cured cylinders for casting 4.**

and 28 days, but at 1 day, the difference was between 20 and 40 percent. For insight into why this occurs, observe that the best-fit line in Fig 5-30 passes above the point associated with 1-day strength, which is point "a." It also passes below the point associated with 3-day strength, which is point "b" in Fig 5-30. This is because the strength gain actually follows a slight curve on the semi-logarithmic plot, rather than a straight line. This causes an inherent inaccuracy in the logarithmic procedure. In other words, using this best-fit curve, a significant error in strength is made at 1 day. There is also an error at 3 days, but it is usually not as significant, because the error is smaller relative to the actual strength at 3 days than it is at 1 day.

### *5.5.2 HYPERBOLIC STRENGTH PREDICTION EQUATION*

Review of the data throughout the first two phases showed that the built-in error of the logarithmic curve fitting procedure was insignificant for all but 1-day strength predictions. When the logarithmic curve fitting procedure is used, the maturity method appeared quite useful and workable, yet there was clearly a weakness in the method. At this stage, another procedure was investigated in an effort to produce better-fitting maturity curves. The procedure was first described in two papers published in the early 1980's by Carino and Lew[19] and by Carino.[18] The following discussion is based on those two papers.

The hyperbolic curve-fitting procedure is a way to develop an equation of the form

$$S = (M - M_o) / [(1/A) + ((M - M_o) / S_u)]$$

where S is strength, M is maturity, A is a constant representing the initial rate of strength gain, $S_u$ is the ultimate strength that can be reached by the concrete, and $M_o$ is the offset maturity, a constant which accounts for the fact that some maturity is gained during the dormant period before the concrete begins to gain strength. The equation and the parameters are identified in Fig 5-31. The variables $M_o$, $S_u$, and A depend on the particular mix design, materials, and curing conditions being used, and they must be determined by following a trial and error technique.

The first step of the trial and error technique is to determine $S_u$. A graphical representation of this step appears in Fig 5-32. The strength and maturity data are the same ones required by the ASTM standards, but in this case their inverses are plotted. This inversion reverses the graph. The points labeled "a" through "e" represent that same data shown in Figs 5-29 and 5-30, only this time the points representing higher strength and greater maturity are in the lower left-hand part of the graph, and points of lower strength and lesser maturity are in the upper right-hand part of the graph. In Fig 5-32, the inverse of strength is on the vertical axis and the inverse of maturity is on the horizontal axis.

Various straight lines, such as Line 1 and Line 2, could be considered representative of these points. Note that at the point where the line intersects the vertical axis, the value of the inverse of the maturity is zero, which is the same as saying that the value of the maturity is infinite. The strength associated with this infinite maturity is the ultimate strength, $S_u$. The value determined for $S_u$ depends on what line was drawn through the data points. In this figure, Line 1 was a least squares linear regression. It represents all the points, but it does not account for the trend at the lower left-hand side of the graph, where the curve is relatively flat. Line 1 intersects the vertical axis at 0.0000667. The inverse of that value is 14,800 psi, which is a possible but not a very probable value for the ultimate strength, $S_u$, of a mix design that is made to reach 3,500 psi in 28 days.

During the calculation of Line 2, the point on the right end of the graph was ignored. Line 2 is a least squares linear regression using only the four latest points, and it is more representative of the later-age trend in the lower left-hand part of the graph. Line 2 intersects the vertical axis at 0.000168; the inverse of that is 5,940 psi, which is another possible value of $S_u$. Here the trial-and-error nature of the procedure begins to show. Although it might be reasonable to assume that 14,800 psi is too high, there are other lines that could be used besides the two mentioned so far, and so other values of $S_u$ could be found. Theoretically, any two of the data points could be used to determine a different line, and thus a different value of $S_u$. Some may not be as obviously wrong as the 14,800 psi value. One practical option at this point is to proceed with calculations in an orderly way, using all the values found for $S_u$ and then determining later which is the best one. This increases the certainty that the best answer will be found, and this is also the best way to computerize the procedure. Carino[18] mentions determining a series of values by first using all the data points to develop a best fit, as was done with Line 1 in the figure, then eliminating the earliest point to determine another value of $S_u$, then eliminating the second-earliest point to get another value, and so on. In the figure, this would amount to simply ignoring successive points on the upper right end of the graph.

That is the procedure which was employed in calculations made during the analysis of Phase II results. The first trial value of $S_u$ was based on a best-fit line, similar to Line 1, through all five data points. The second trial value of $S_u$ was based on a best-fit line through the last four data points, similar to Line 2. Finally, the third value was based on a best fit line through the last three data points. In the calculations for Phase II, no lines were actually drawn. The values were determined mathematically on a personal computer spreadsheet.

After several values of $S_u$ have been determined, the second step of the procedure is to determine values for $M_o$ and A. Figure 5-33 presents a graphical representation of this step. In the figure, $S/(S_u–S)$ is plotted on the vertical axis and maturity is plotted on the horizontal axis. The earliest-age data points are in the lower left-hand part of the graph. Once again, several best-fit lines can be drawn through the points shown in the figure. The maturity at the point where such a line intersects the horizontal axis determines the offset maturity, $M_o$, and the slope of the best-fit line sets the rate constant, $k_T$. The initial rate of strength gain, A, is then found by multiplying $k_T$ by $S_u$. Again, there are several lines that could be drawn through the data points, some of which might improve the estimates of $M_o$, $k_T$, and A, by ignoring later-age data points.

The only way to tell which is the best line is to calculate them all and determine which produces the most appropriate constants. For each of several values of $S_u$, there will be several values of A and $M_o$. Together, all these combinations of values describe numerous hyperbolic curves, curves of the form

$$S = (M - M_o) / ((1/A) + ((M - M_o) / S_u .$$

The best-fit hyperbolic curve is the one which compares most closely to the actual data points. That is, the data points determined by measuring maturity and cylinder compressive strength will all be close to or coincident with the calculated best-fit hyperbolic curve, developed by trial and error.

Table 5-2 shows the results of using data from casting 4 to develop a best-fit hyperbolic curve for the relationship of cylinder compressive strength to maturity.

There are several combinations of points that yield negative values of $M_o$ and ultimate strengths of nearly three times the 28-day strength. A negative offset maturity is meaningless, so these combinations of points are not the correct ones to find the best-fit curve. For the combinations remaining, a review of the column "Ratio of Estimated to Actual" shows which one is the closest in all cases. The case in which the last three points are used to determine $S_u$ and the first three points are used to determine A and $M_o$ is the one which best matches the actual relationship of strength and maturity. Using the hyperbolic equation with the constants A = 2.47 psi/degree C-Hr, $S_u$ = 5,770 psi, and $M_o$ = 338 psi, the strength predictions are within 4 percent of the actual strengths at all ages. This is shown in Fig 5-34, where the hyperbolic curve closely matches the data points. The hyperbolic

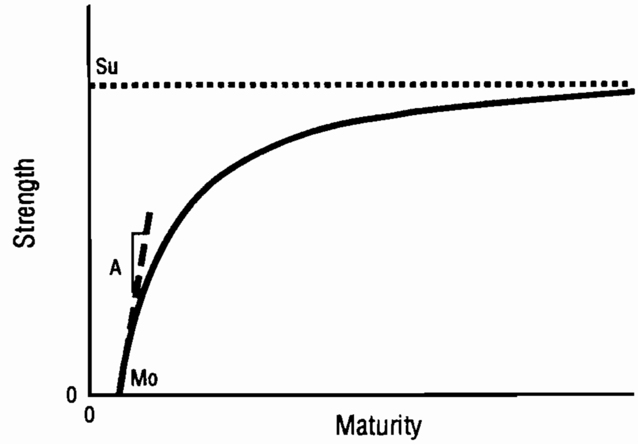


**Fig 5-31. Typical relationship between maturity and strength with hyperbolic constants defined.**

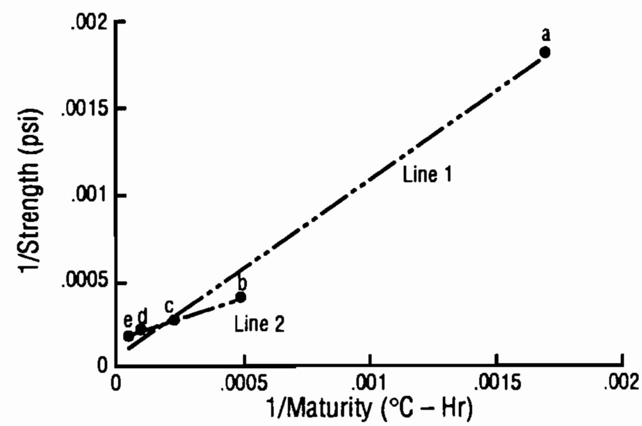


**Fig 5-32. Relationship between maturity and strength for casting 4, used to determine $S_u$.**

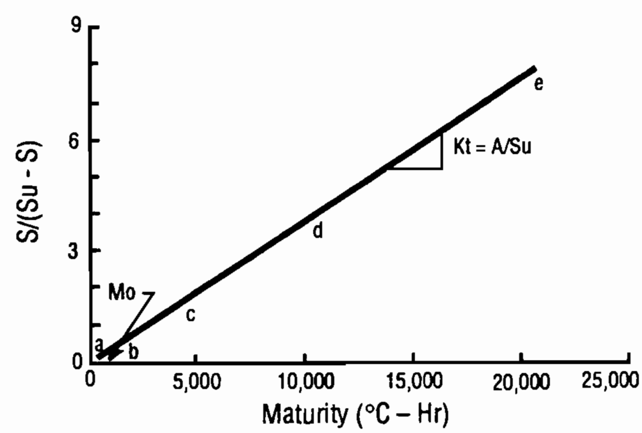


**Fig 5-33. Relationship between maturity and S/Su-S used to determine $K_t$, A and $M_o$.**

**TABLE 5-2. TRIAL-AND-ERROR DETERMINATION OF CONSTANTS FOR HYPERBOLIC MATURITY EQUATION**

| Number of Points used to find Su | Number of Points used to find A and Mo | Constants of the Hyperbolic Equation | | | Wetroom Maturity of Casting 4 | Strength Estimate Using Hyperbolic Curve | Actual Strength of Wetroom Cylinders | Ratio of Estimate to Actual |
|---|---|---|---|---|---|---|---|---|
| | | Su (psi) | A (psi/°C-Hr) | Mo (°C-Hr) | (deg C-Hr) | (psi) | (psi) | |
| 5 | 5 | 14,740 | 0.325 | -6,607 | 592 | 2,016 | 549 | 3.672 |
| | | | | | 2,040 | 2,357 | 2,443 | 0.965 |
| | | | | | 4,735 | 2,945 | 3,766 | 0.782 |
| | | | | | 10,555 | 4,042 | 4,597 | 0.879 |
| | | | | | 20,635 | 5,526 | 5,183 | 1.066 |
| 5 | 4 | 14,740 | 0.560 | -2,322 | 592 | 1,469 | 549 | 2.676 |
| | | | | | 2,040 | 2,095 | 2,443 | 0.858 |
| | | | | | 4,735 | 3,116 | 3,766 | 0.827 |
| | | | | | 10,555 | 4,842 | 4,597 | 1.053 |
| | | | | | 20,635 | 6,867 | 5,183 | 1.325 |
| 5 | 3 | 14,740 | 1.046 | -271 | 592 | 850 | 549 | 1.548 |
| | | | | | 2,040 | 2,077 | 2,443 | 0.850 |
| | | | | | 4,735 | 3,864 | 3,766 | 1.026 |
| | | | | | 10,555 | 6,405 | 4,597 | 1.393 |
| | | | | | 20,635 | 8,806 | 5,183 | 1.699 |
| 4 | 5 | 5,942 | 1.963 | -24 | 592 | 1,005 | 549 | 1.831 |
| | | | | | 2,040 | 2,409 | 2,443 | 0.986 |
| | | | | | 4,735 | 3,632 | 3,766 | 0.964 |
| | | | | | 10,555 | 4,620 | 4,597 | 1.005 |
| | | | | | 20,635 | 5,183 | 5,183 | 1.000 |
| 4 | 4 | 5,942 | 1.956 | -38 | 592 | 1020 | 549 | 1.858 |
| | | | | | 2,040 | 2,413 | 2,443 | 0.988 |
| | | | | | 4,735 | 3,631 | 3,766 | 0.964 |
| | | | | | 10,555 | 4,618 | 4,597 | 1.005 |
| | | | | | 20,635 | 5,180 | 5,183 | 0.999 |
| 4 | 3 | 5,942 | 2.329 | 303 | 592 | 604 | 549 | 1.100 |
| | | | | | 2,040 | 2,407 | 2,443 | 0.985 |
| | | | | | 4,735 | 3,771 | 3,766 | 1.001 |
| | | | | | 10,555 | 4,758 | 4,597 | 1.035 |
| | | | | | 20,635 | 5,279 | 5,183 | 1.019 |
| 3 | 5 | 5,767 | 2.495 | 530 | 592 | 151 | 549 | 0.275 |
| | | | | | 2,040 | 2,279 | 2,443 | 0.933 |
| | | | | | 4,735 | 3,721 | 3,766 | 0.988 |
| | | | | | 10,555 | 4,686 | 4,597 | 1.019 |
| | | | | | 20,635 | 5,172 | 5,183 | 0.998 |
| 3 | 4 | 5,767 | 2.201 | 122 | 592 | 876 | 549 | 1.596 |
| | | | | | 2,040 | 2,437 | 2,443 | 0.998 |
| | | | | | 4,735 | 3,678 | 3,766 | 0.977 |
| | | | | | 10,555 | 4,609 | 4,597 | 1.003 |
| | | | | | 20,635 | 5,113 | 5,183 | 0.986 |
| 3 | 3 | 5,767 | 2.471 | 338 | 592 | 566 | 549 | 1.031 |
| | | | | | 2,040 | 2,432 | 2,443 | 0.995 |
| | | | | | 4,735 | 3,767 | 3,766 | 1.000 |
| | | | | | 10,555 | 4,694 | 4,597 | 1.021 |
| | | | | | 20,635 | 5,172 | 5,183 | 0.998 |

equation could be used with great confidence because at all ages the equation reflects the actual strength versus maturity relationship developed in the laboratory.

At the beginning of this section, it was stated that the logarithmic best-fit equation was inaccurate at ages of less than 3 days. The hyperbolic best-fit equation brings improved accuracy to concrete less than 3 days old. Figure 5-35 is a revised version of Fig 5-10, based on the hyperbolic equation in addition to the logarithmic equation. Note that in Fig 5-35 there is only a slight deviation from unity at 1 day for the data found with the hyperbolic curve, while there is a significant deviation from unity for the data found with the logarithmic equation.

In summary, this investigation showed that the maturity method with the hyperbolic strength equation consistently gives an excellent estimate of in-place concrete strength without demanding a great deal of time, effort, or skill to employ.

## 5.6 PHASE III

Phase II showed that the maturity method was the most representative, accurate, and repeatable of all the tests considered in this investigation. After the completion of that phase and its data analysis, a third phase was begun. Phase III consisted of an investigation into the effect of high-temperature curing on the maturity method.

The Phase III testing regimen was described in detail in Section 3.5 but will be briefly summarized here. In each casting, two sets of cylinders were cast, instrumented, and immediately placed for curing in two separate 100-percent-humidity chambers. The first group was cured at a constant 73°F. The second group was cured at temperatures ranging from 73°F up to 110 or 130°F. Beginning the day after casting, strength predictions were made by the maturity method, and then cylinders were broken to verify the predictions.

Results are presented below for each casting in Phase III, and a summary follows the results.

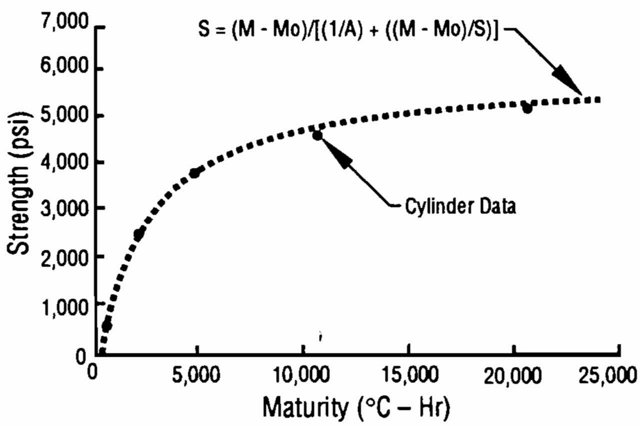


**Fig 5-34. Comparison of hyperbolic prediction curve with actual cylinder maturities and strength.**

### *5.6.1 CASTING 10*

Results for casting 10 were presented in Table 4-15 and are also plotted in Fig 5-36. In that figure, in addition, is a record of the curing temperature variations for the high-temperature cure. This figure shows that the high-temperature curing introduces a significant underestimation into the maturity method strength estimation. Phase II results showed that the maturity method usually overestimated strength at very early ages, and that is also the case in Phase III with the 73-degree group. The 20-hour maturity strength prediction was about 30 percent above the actual strength in the 73-degree group. For the 110-degree group, the maturity curve predicted a 20-hour strength about 10 percent below the actual strength. Thus, the graph shows that the 110-degree curing caused a low strength estimate, even though the prediction model is known to generate high 1-day strength estimates under ordinary conditions.

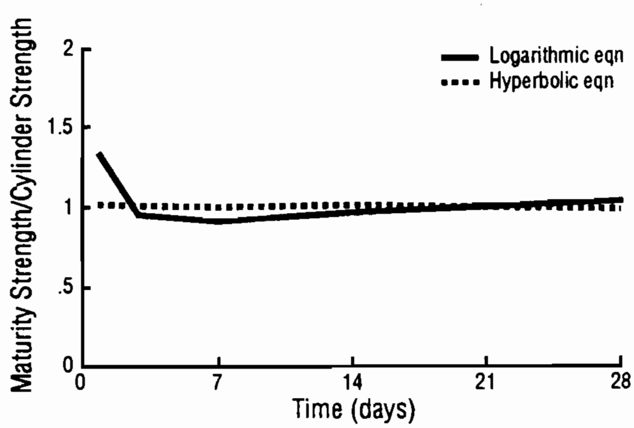


**Fig 5-35. Ratio of maturity method strength to strength of wetroom-cured cylinders using two different maturity relationships.**

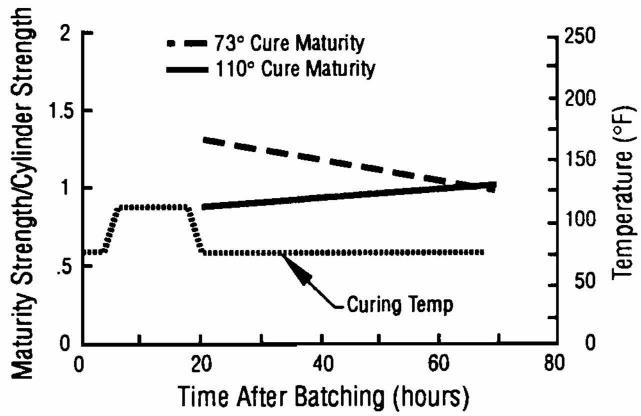


**Fig 5-36. Ratio of maturity method strength to cylinder strength for casting 10.**

### *5.6.2 CASTING 11*

The effect noted in the 110°F curing is also found at 130°F curing. In casting 11, one group of cylinders was cured at the standard 73°F, and the other group was cured at up to 130°F. The 21-hour maturity strength prediction was about 40 percent above the actual strength in the 73-degree group. For the 130-degree group, the maturity curve predicted a 21-hour strength almost 15 percent below the actual strength. As before, the numbers in Table 4-16 and the graph in Fig 5-37 show that high-temperature curing caused a low strength estimate, even though the prediction model is known to generate high 1-day strength estimates. This demonstrates that the maturity method will underestimate in-place strengths if it is used in a situation where the actual curing temperatures are significantly higher than those under which the maturity curve was developed.

### *5.6.3 PHASE III SUMMARY*

Phase III data show that if concrete is cured under significantly higher temperatures than those under which the maturity curve was developed, then the maturity method will be very conservative at ages of less than 3 days. A temperature 40 to 60°F higher may be considered significantly higher.

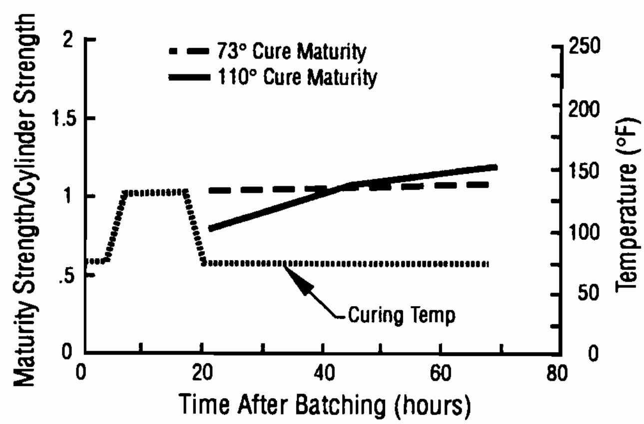


**Fig 5-37. Ratio of maturity method strength to cylinder strength for casting 11.**

# CHAPTER 6. SUMMARY AND CONCLUSIONS

## 6.1 SUMMARY

The primary goal of this study was to evaluate the currently-available concrete testing practices which might aid in evaluating and estimating the strength of the concrete in the field, particularly at early ages before the standard 28-day cylinder strength or 7-day beam strength is available. Special consideration was given to including both laboratory and field-cured specimens during hot and cold weather conditions.

The test methods used in this study included compressive cylinders, flexural beams, penetration resistance, rebound hammer, pullout, maturity, ultrasonic pulse velocity, and drilled cores.

Three different concrete mix designs were used, including cement plus fly ash contents ranging from 300 to 500 pounds per cubic yard, three maximum sizes of river gravels, and two maximum-size crushed limestone aggregates. Specified design concrete strengths were 3,500 and 5,000 psi, resulting in measured 28-day cylinder compressive strengths ranging from 3,700 to 8,700 psi. Full-size test slabs were cast and cured outdoors under simulated field conditions during the period from August through May. Test specimens were cured both under laboratory-controlled conditions and under field conditions adjacent to the slabs.

Test results from the different methods being evaluated were compared at concrete ages ranging from 1 to 28 days. Each test method was also evaluated for within-test variability among sets of companion specimens.

Of all the test methods studied, the maturity method exhibited the lowest variability and most consistent agreement with the generally-accepted standards for concrete testing, including compression cylinders and flexural beams for test ages after 1 day. The maturity method can also be applied for predicting the strength of concrete at 1 day; however, special considerations and curve-fitting techniques need to be applied to develop an accurate strength versus maturity relationship.

## 6.2 CONCLUSIONS

Based on the results from the research presented herein, the following conclusions can be drawn:

(1) Results from the rebound hammer were found to be the least reliable of those from all the test methods studied.

(2) Pullout test results were found to be significantly more variable than those from standard beam or cylinder tests. In addition, pullout tests require either preplanning when the Lok-Test is performed or a significant amount of labor at the time of testing when the CAPO test is used.

(3) Penetration resistance tests such as the Windsor Probe provide useful information regarding the relative strength of concrete, but the test variability is considerably high, especially at early ages.

(4) Of all the test methods evaluated, the maturity method was found to be the only one that provided reliable data comparable in variability and consistency to those obtained from standard quality control specimens.

(5) When the maturity method is used in conjunction with a hyperbolic curve-fitting procedure for developing the strength versus maturity curve, this method represents an excellent, reliable, and easy test procedure for evaluating the strength of concrete at early ages in the field.

# REFERENCES


1. ACI Committee 214, "Recommended Practice for Evaluation of Strength Test Results of Concrete," American Concrete Institute, Detroit, Michigan, 1989.
2. ACI Committee 228, "In-Place Methods for Determination of Strength of Concrete," ACI Materials Journal, pp 446-471, September-October 1988.
3. ACI Committee 306, "Cold Weather Concreting," ACI Materials Journal, pp 280-302, July-August 1988.
4. Akashi, T., and Amasaki, S., "Study of the Stress Waves in the Plunger of a Rebound Hammer at the Time of Impact," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 17-34.
5. Andersen, J., and Nerenst, P., "Wave Velocity in Concrete," ACI Journal, Vol 48, April 1952, pp 613-635. Discussion, Vol 48, December 1952, p 636.
6. Anderson, D. A., and Seals, R. K., "Pulse Velocity as a Predictor of 28- and 90-Day Strength," ACI Journal, Vol 78, pp 116-122, March-April 1981.
7. "Annual Book of ASTM Standards—Part 14," ASTM Committee C-9, American Society for Testing and Materials, Philadelphia, 1987.
8. Bartos, M. J., Jr., "Testing Concrete in Place," Civil Engineering, Vol 49, pp 66-69, October 1979.
9. Bickley, J. A., "The Evaluation and Acceptance of Quality Concrete by In-Place Testing," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 95-109.
10. Bickley, J. A., "Trinity Square: Commentary on Concrete Test Data," Cement, Concrete, and Aggregates, Vol 6, No. 1, Summer 1984, pp 28-32.
11. Bickley, J. A., and Fasullo, S., "Analysis of Pull-Out Test Data from Construction Sites," presented to the Sixtieth Annual Meeting of the Transportation Research Board, 1981, 43 pp.
12. Bloem, D. L., "Concrete Strength in Structures," ACI Journal, Vol 65, pp 176-187, March 1968.
13. Bloem, D. L., "Concrete Strength Measurement—Cores Versus Cylinders," Proceedings, American Society for Testing and Materials, Vol 65, 1965, pp 668-696.
14. Burmeister, R. A., "Tests of Paper Molds for Concrete Cylinders," ACI Journal, Vol 47, pp 17-24, September 1950.
15. Bungey, J. H., "The Influence of Reinforcement on Ultrasonic Pulse Velocity Testing," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 229-246.
16. Bungey, J. H., "The Testing of Concrete in Structures," Surrey University Press, London, 1982, 207 pp.
17. Carette, C. G., and Malhotra, V. M., "In Situ Tests: Variability and Strength Prediction of Concrete at Early Ages," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 111-141.
18. Carino, N. J., "The Maturity Method: Theory and Application," Cement, Concrete, and Aggregates, Vol 6, No. 2, Winter 1984, pp 61-73.
19. Carino, N. J., and Lew, H. S., "Temperature Effects on Strength-Maturity Relations of Mortar," ACI Journal, Vol 80, pp 177-182, May-June 1983.
20. Carino, N. J., Lew, H. S., and Volz, C. K., "Early Age Temperature Effects on Concrete Strength Prediction by the Maturity Method," ACI Journal, Vol 80, pp 93-101, March-April 1983.
21. Carrasquillo, P. M., and Carrasquillo, R. L., "Effect of Using Unbonded Capping Systems on the Compressive Strength of Concrete Cylinders," ACI Materials Journal, Vol 85, pp 141-7, May-June 1988. Discussion, ACI Materials Journal, Vol 86, pp 191-194, March-April 1989.
22. Carrasquillo, P. M., and Carrasquillo, R. L., "Guidelines for Use of High Strength Concrete in Texas Highways," Research Report 367-1F, Center for Transportation Research, The University of Texas at Austin, August 1986.
23. Carrasquillo, R. L., Nilson, A. H., and Slate, F. O., "The Production of High-Strength Concrete," Department Report No. 78-1, Structural Engineering Department, Cornell University, Ithaca, New York, May 1978, 91 pp.
24. Chin Fung Kee, "Relation Between Strength and Maturity of Concrete," ACI Journal, Vol 68, pp 196-203, March 1971.
25. Chung, H. W., and Law, K. S., "Diagnosing In-Situ Concrete by Ultrasonic Pulse Technique," Concrete International, Vol 5, October 1983, pp 42-49.
26. Cohen, E., "President's Memo: The Concrete Industry's Future," ACI Journal, Vol 69, p N-1, July 1972.

27. "Concrete Testing Scrutinized," ENR, December 5, 1985, pp 10-11.

28. Dilly, R. L., and Ledbetter, W. B., "Concrete Strength Based on Maturity and Pullout," ASCE Journal of Structural Engineering, Vol 110, pp 354-369, February 1984. Discussion, Vol 110, pp 482-485, February 1984.

29. Dilly, R. L., and Vogt, W. L., "Pullout Test, Maturity, and PC Spreadsheet Software," Nondestructive Testing, SP-112, American Concrete Institute, Detroit, 1988, pp 193-219.

30. Edwards, H. H., "Progress in Determining the Relation Between Test Cylinders and Concrete in the Structure," Proceedings, American Concrete Institute, Vol 26, 1930, pp 57-64.

31. Facaoaru, I., "Romanian Achievements in Nondestructive Strength Testing of Concrete," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 35-56.

32. Gaynor, R. D., Meininger, R. C., and Khan, T. S., "Effect of Temperature and Delivery Time on Concrete Proportions," Temperature Effects on Concrete, STP 858, American Society for Testing and Materials, Philadelphia, 1985, pp 68-87.

33. Goral, M. L., "Empirical Time-Strength Relations of Concrete," ACI Journal, Vol 53, pp 215-224, August 1956.

34. Gray, R. J., "Results of an Interlaboratory Concrete Testing Program: Part 1," Cement, Concrete and Aggregates, Vol 12, Summer 1990, pp 12-23.

35. Guo, Chengju, "Maturity of Concrete: Method for Predicting Early-Stage Strength," ACI Materials Journal, Vol 86, pp 341-353, July-August 1989. Discussion, Vol 87, pp 296-299, May-June 1990.

36. Halvorsen, G. T., and Farahmandnia, A., "The Willow Island Collapse: A Maturity Case Study," Temperature Effects on Concrete, STP 858, American Society for Testing and Materials, Philadelphia, 1985, pp 168-176.

37. Harrell, T. R., "Nondestructive Testing Use in Evaluating the Form Removal Time for a Large Diameter Tunnel Concrete Lining," Nondestructive Testing, SP-112, American Concrete Institute, Detroit, 1988, pp 153-164.

38. Hellier, A. K., Sansalone, M., Carino, N. J., Stone, W. C., and Ingraffea, A. R., "Finite-Element Analysis of the Pullout Test Using a Nonlinear Discrete Cracking Approach," Cement, Concrete, and Aggregates, Vol 9, Summer 1987, pp 20-29.

39. Horiguchi, T., Saeki, N., and Fujita, Y., "Evaluation of Pullout Test for Estimating Shear, Flexural, and Compressive Strength of Fiber Reinforced Silica Fume Concrete," ACI Materials Journal, Vol 85, pp 126-132, March-April 1988.

40. Hulshizer, A. J., Edgar, M. A., Daniels, R. E., Suminsby, J. D., and Myers, G. E., "Maturity Concept Proves Effective in Reducing Form Removal Time and Winter Curing Cost," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 351-376.

41. Jensen, B. C., and Braestrup, M. W., "Lok-Tests Determine the Compressive Strength of Concrete," Nordisk Betong (Stockholm), Vol 19, No. 2, pp 9-11.

42. Jones, R., "The Non-Destructive Testing of Concrete," Magazine of Concrete Research, Vol 11, No. 32, pp 67-84, July 1959.

43. Kaplan, M. F., "The Effects of Age and Water/Cement Ratio Upon the Relation Between Ultrasonic Pulse Velocity and Compressive Strength of Concrete," Magazine of Concrete Research, Vol 11, No. 32, pp 85-92, July 1959.

44. Kaplan, M. F., "The Relation Between Ultrasonic Pulse Velocity and the Compressive Strength of Concretes Having the Same Workability but Different Mix Proportions," Magazine of Concrete Research, Vol 12, No. 34, pp 3-8, March 1960.

45. Keiller, A. P., "Assessing the Strength of In-Situ Concrete," Concrete International, Vol 7, pp 15-21, February 1985.

46. Khoo, L. M., "Pullout Technique—An Additional Tool for In-Situ Concrete Strength Determination," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 143-159.

47. Klieger, P., "Effect of Mixing and Curing Temperature on Concrete Strength," ACI Journal, Vol 54, pp 1063-1081, June 1958.

48. Kotsovos, M. D., and Cheong, H. K., "Applicability of Test Specimen Results for the Description of the Behavior of Concrete in a Structure," ACI Journal, Vol 81, pp 358-363, July-August 1984.

49. Kriviak, G. J., and Scanlon, A., "Estimation of Compressive Strength of Concrete in Existing Structures Based on In-Situ Test Data," ACI Journal, Vol 84, pp 235-245, May-June 1987.

50. Leslie, J. R., and Cheesman, W. J., "An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures," ACI Journal, Vol 46, pp 17-36, September 1949.

51. Malhotra, V. M., "Concrete Strength Requirements—Cores Versus In-Situ Evaluation," ACI Journal, Vol 74, pp 163-172, April 1977. Discussion, Vol 74, pp 521-530, October 1977.
52. Malhotra, V. M., "In-Place Evaluation of Concrete," Journal of the Construction Division CO2, Proceedings, ASCE, Vol 101, No. 2, pp 345-357, June 1975.
53. Malhotra, V. M., "Testing Hardened Concrete: Nondestructive Methods," ACI Monograph No. 9, American Concrete Institute, Detroit, 1976, 188 pp.
54. Malhotra, V. M., and Carette, G. G., "Comparison of Pullout Strength of Concrete with Compressive Strength of Cylinders and Cores, Pulse Velocity, and Rebound Number," ACI Journal, Vol 77, pp 161-170, May-June 1980.
55. Mercer, L. B., "Sixty Contributory Causes to Concrete Strength Variations," ACI Journal, Vol 22, pp 745-747, May 1951.
56. Millstein, L., and Sabnis, G. M., "Determination of Concrete Strengths Using Ultrasonic Waves," Recent Advances in Engineering Mechanics and Their Impact on Civil Engineering Practice, American Society of Civil Engineers, New York, 1983, pp 561-564.
57. Mirza, S. A., Hatzinikolas, M., and MacGregor, J. G., "Statistical Descriptions of Strength of Concrete," Journal of the Structural Division, ASCE, Vol 105, pp 1021-1037, June 1979.
58. Mittelacher, M., "Effect of Hot Weather Conditions on the Strength Performance of Set-Retarded Field Concrete," Temperature Effects on Concrete, STP 858, American Society for Testing and Materials, Philadelphia, 1985, pp 89-106.
59. Naik, T. R., "Maturity Functions For Concrete Cured During Winter Conditions," Temperature Effects on Concrete, STP 858, American Society for Testing and Materials, Philadelphia, 1985, pp 107-117.
60. Nasser, K. W., and Al-Manaseer, A. A., "Comparison of Nondestructive Testers of Hardened Concrete," ACI Materials Journal, Vol 84, pp 374-80, September-October 1987. Discussion, ACI Materials Journal, Vol 85, pp 326-327, July-August 1988.
61. Nasser, K. W., and Al-Manaseer, A. A., "It's Time For a Change From 6 x 12- to 3 x 6-in. Cylinders," ACI Materials Journal, Vol 84, pp 213-216, May-June 1987.
62. Nasser, K. W., and Al-Manaseer, A. A., "New Non-Destructive Test," Concrete International, Vol 9, pp 41-44, January 1987.
63. Nurse, R. W., "Steam Curing of Concrete," Magazine of Concrete Research, No. 2, pp 79-88, June 1949.
64. Ottosen, N. S., "Nonlinear Finite Element Analysis of Pull-Out Test," Journal of the Structural Division, ASCE, Vol 107, pp 591-603, April 1981.
65. Parsons, T. J., "Early Concrete Strength Determination by Maturity," Nondestructive Testing, SP-112, American Concrete Institute, Detroit, 1988, pp 181-191.
66. Parsons, T. J., and Naik, T. R., "Early Age Concrete Strength Determination by Pullout Testing and Maturity," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 177-199.
67. Pauw, A., "Static Modulus of Elasticity of Concrete as Affected by Density," ACI Journal, Vol 57, pp 679-687, December 1960.
68. Peterman, M. B., and Carrasquillo, R. L., "Production of High Strength Concrete," Research Report 315-1F, Center for Transportation Research, The University of Texas at Austin, October 1983.
69. Petersen, C. G., "Determination of Strength of Concrete In-Situ with the Lok-Test, the CAPO Test and the COMA-Meter," presented at the 1988/89 Advanced Concrete Technology Course, Imperial College of Science and Technology, London, September 1988, p 9.
70. Philleo, R. E., "Lunatics, Liars, and Liability," ACI Journal, Vol 73, pp 181-183, April 1976.
71. Philleo, R. E., "President's Message: In-Place Heresy," ACI Journal, Vol 71, p N-1, March 1974.
72. Plowman, J. M., "Maturity and the Strength of Concrete," Magazine of Concrete Research, Vol 8, pp 13-22, March 1956. Discussion, Vol 8, pp 169-183, November 1956.
73. Price, W. H., "Factors Influencing Concrete Strength," ACI Journal, Vol 47, pp 417-432, February 1951.
74. Popovics, S., "Effect of Curing Method and Final Moisture Condition on Compressive Strength of Concrete," ACI Journal, Vol 83, pp 650-657, July-August 1986.
75. Ramakrishnan, V., "Past and Future of Concrete Quality Evaluation," Journal of the Construction Division, ASCE, Vol 102, pp 263-276, June 1976.

76. Ramsdell, S. D., "Interpreting Test Reports: Look Above the Bottom Line," Concrete Construction, Vol 33, pp 939-940, October 1988.

77. Richards, O., "Pullout Strength of Concrete," Reproducibility and Accuracy of Mechanical Tests, STP-626, American Society for Testing and Materials, Philadelphia, 1977, pp 32-40.

78. Rix, G. J., Bay, J. A., and Stokoe, K. H., "Assessing In-Situ Stiffness of Curing Portland Cement Concrete with Seismic Tests," Transportation Research Board Record, accepted for publication in 1990.

79. "Rx For Low Cylinder Tests," Concrete Construction, Vol 33, pp 898-899, September 1988.

80. Sabnis, G. M., and Mirza, S. A., "Size Effects in Model Concretes," Journal of the Structural Division, ASCE, Vol 105, pp 1007-1020, June 1979.

81. Samarin, A., and Dhir, R. K., "Determination of In-Situ Concrete Strength: Rapidly and Confidently by Nondestructive Testing," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 77-94.

82. Saul, A. G. A., "Principles Underlying the Steam Curing of Concrete at Atmospheric Pressure," Magazine of Concrete Research, Vol 2, pp 127-140, March 1951.

83. Shilstone, J. M., Sr., "The Cylinder Test—Reliable Informer or False Prophet," Concrete International, Vol 2, pp 63-68, July 1980.

84. Shilstone, J. M., Sr., "A Hard Look at Concrete," Civil Engineering, Vol 59, pp 47-49, January 1989. Discussion, Vol 59, p 29, May 1989.

85. Skramtajew, B. G., "Determining Concrete Strength for Control of Concrete in Structures," Proceedings, American Concrete Institute, Vol 34, January-February 1938, pp 285-304.

86. Snedecor, G. W., and Cochran, W. G., "Statistical Methods," 6th Edition, Iowa State University Press, Ames, 1967, 593 pp.

87. Stone, W. C., and Carino, N. J., "Deformation and Failure in Large-Scale Pullout Tests," ACI Journal, Vol 80, pp 501-513, November-December 1983. Discussion, Vol 81, pp 525-534, September-October 1984.

88. Stone, W. C., and Giza, B. J., "The Effect of Geometry and Aggregate on the Reliability of the Pullout Test," Concrete International, Vol 7, pp 27-36, February 1985.

89. Sturrup, V. R., Vecchio, F. J., and Caratin, H., "Pulse Velocity as a Measure of Concrete Compressive Strength," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 201-227.

90. Swamy, R. N., and Al-Hamad, A. H., "The Use of Pulse Velocity Measurements to Estimate Strength of Air-Dried Cubes and Hence In-Situ Strength of Concrete," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 247-276.

91. Swamy, R. N., and Al-Hamad, A. H., "Evaluation of Small Diameter Core Tests to Determine In-Situ Strength of Concrete," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 411-440.

92. Tanigawa, Y., Baba, K., and Mori, H., "Estimation of Concrete Strength by Combined Nondestructive Testing Method," In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 57-76.

93. Teodoru, G. V., "The Use of Simultaneous Nondestructive Tests to Predict the Compressive Strength of Concrete," Nondestructive Testing, SP-112, American Concrete Institute, Detroit, 1988, pp 137-152.

94. Tremper, B., "The Measurement of Concrete Strength by Embedded Pull-Out Bars," Proceedings, American Society for Testing and Materials, Vol 44, 1944, pp 880-887.

95. Tuthill, L. H., "Games People Play With Concrete," ACI Journal, Vol 73, pp 671-678, December 1976.

96. Vogt, W. L., Beizai, V., and Dilly, R. L., "In-Situ Pullout Strength of Concrete with Inserts Embedded by 'Finger Placing,'" In-Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, 1984, pp 161-175.

97. Yener, M., and Chen, W. F., "On In-Place Strength of Concrete and Pullout Tests," Cement, Concrete, and Aggregates, Vol 6, Winter 1984, pp 90-99.

98. Young, R. B., "The Field Testing of Concrete," Proceedings, American Society for Testing Materials, Philadelphia, Vol 27, 1927, pp 390-406.

99. Yun, C. H., Choi, K. R., Kim, S. Y., and Song, Y. C., "Comparative Evaluation of Nondestructive Test Methods for In-Place Strength Determination," Nondestructive Testing, SP-112, American Concrete Institute, Detroit, 1988, pp 111-136.

100. Zoldners, N. G., "Calibration and Use of Impact Test Hammer," ACI Journal, Vol 54, pp 161-165, November 1957.

# APPENDIX A

**TABLE A-1. DATA FOR WETROOM-CURED CYLINDER MATURITY CURVES OF CASTINGS 1, 2, AND 3**

| | STRENGTH (psi) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CASTING 1 | 1000 | 2290 | 3050 | 3580 | 4170 |
| CASTING 2 | 1000 | 2060 | 2690 | 3570 | 3710 |
| CASTING 3 | 1280 | 2500 | 2770 | 3670 | 4650 |

| | MATURITY (degree C–Hr) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CASTING 1 | 636 | 2106 | 5136 | 9981 | 20256 |
| CASTING 2 | 616 | 2011 | 5056 | 10096 | 20176 |
| CASTING 3 | 702 | 2100 | 4965 | 10035 | 20145 |

| | slope m (psi/log Mat) | intercept b (psi) |
|---|---|---|
| CASTING 1 | 2090 | –4766 |
| CASTING 2 | 1868 | –4157 |
| CASTING 3 | 2182 | –4954 |

NOTE: slope and intercept calculated using logarithmic curve fit. See chapter 5.

**TABLE A-2. CASTINGS 1, 2, AND 3, RATIOS OF VARIOUS STRENGTHS TO TWO STANDARDS, AT 1 DAY**

| | CYLINDERS Cured in wetroom | BEAMS Cured in wetroom | PULLOUT | | PEN. RESIST. Windsor Probe | REBOUND HAMMER | MATURITY from wetroom cylinders |
|---|---|---|---|---|---|---|---|
| | | | CAPO | LOK | | | |
| **CASTING 1** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.29 | 1.84 | 1.53 | 0.90 | 1.70 | 1.33 |
| ratio to strength of wetroom beams | 3.45 | 1.00 | 6.34 | 5.28 | 3.10 | 5.86 | 4.57 |
| **CASTING 2** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.29 | 1.28 | 1.16 | 0.65 | 1.70 | 1.29 |
| ratio to strength of wetroom beams | 3.45 | 1.00 | 4.41 | 4.00 | 2.24 | 5.86 | 4.44 |
| **CASTING 3** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.29 | 1.48 | 1.19 | 1.21 | 1.33 | 1.15 |
| ratio to strength of wetroom beams | 3.46 | 1.00 | 5.11 | 4.11 | 4.19 | 4.59 | 3.98 |

**TABLE A-3. CASTINGS 1, 2, AND 3, RATIOS OF VARIOUS STRENGTHS TO TWO STANDARDS, AT 3 DAYS**

| | CYLINDERS Cured in wetroom | BEAMS Cured in wetroom | PULLOUT | | PEN. RESIST. Windsor Probe | REBOUND HAMMER | MATURITY from wetroom cylinders |
|---|---|---|---|---|---|---|---|
| | | | CAPO | LOK | | | |
| **CASTING 1** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.22 | 1.35 | 1.22 | 1.31 | 1.14 | 1.07 |
| ratio to strength of wetroom beams | 4.58 | 1.00 | 6.18 | 5.58 | 6.00 | 5.20 | 4.89 |
| **CASTING 2** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.20 | 1.16 | 1.03 | 0.78 | 0.83 | 1.03 |
| ratio to strength of wetroom beams | 4.90 | 1.00 | 5.69 | 5.05 | 3.57 | 4.05 | 5.07 |
| **CASTING 3** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.20 | 0.94 | 1.01 | 1.02 | 0.68 | 0.96 |
| ratio to strength of wetroom beams | 5.00 | 1.00 | 4.68 | 5.04 | 5.10 | 3.40 | 4.78 |

**TABLE A-4. CASTINGS 1, 2, AND 3, RATIOS OF VARIOUS STRENGTHS TO TWO STANDARDS, AT 7 DAYS**

| | CYLINDERS | BEAMS | PULLOUT | | PEN. RESIST. | REBOUND | MATURITY |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured in wetroom | CAPO | LOK | Windsor Probe | HAMMER | from wetroom cylinders |
| **CASTING 1** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.18 | 1.43 | 1.11 | 1.05 | 0.66 | 1.07 |
| ratio to strength of wetroom beams | 5.45 | 1.00 | 7.77 | 6.05 | 5.71 | 3.57 | 5.82 |
| **CASTING 2** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.19 | 1.11 | 0.91 | 1.04 | 1.12 | 1.01 |
| ratio to strength of wetroom beams | 5.17 | 1.00 | 5.75 | 4.73 | 5.38 | 5.77 | 5.23 |
| **CASTING 3** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.20 | 1.26 | 1.00 | 0.94 | 0.68 | 1.10 |
| ratio to strength of wetroom beams | 5.04 | 1.00 | 6.33 | 5.02 | 4.73 | 3.41 | 5.52 |

TABLE A-5. CASTINGS 1, 2, AND 3, RATIOS OF VARIOUS STRENGTHS TO TWO STANDARDS, AT 28 DAYS

| | CYLINDERS Cured in wetroom | BEAMS Cured in wetroom | PULLOUT | | PEN. RESIST. Windsor Probe | REBOUND HAMMER | MATURITY from wetroom cylinders |
|---|---|---|---|---|---|---|---|
| | | | CAPO | LOK | | | |
| **CASTING 1** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.16 | 1.05 | 1.08 | 0.90 | 0.41 | 1.11 |
| ratio to strength of wetroom beams | 6.22 | 1.00 | 6.52 | 6.72 | 5.60 | 2.54 | 6.89 |
| **CASTING 2** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.16 | 1.20 | 0.80 | 1.04 | 1.17 | 1.01 |
| ratio to strength of wetroom beams | 6.29 | 1.00 | 7.54 | 5.05 | 6.53 | 7.37 | 6.33 |
| **CASTING 3** | | | | | | | |
| ratio to strength of wetroom cylinder | 1.00 | 0.13 | 0.83 | 0.82 | 0.84 | 0.73 | 0.96 |
| ratio to strength of wetroom beams | 7.69 | 1.00 | 6.38 | 6.33 | 6.45 | 5.62 | 7.41 |

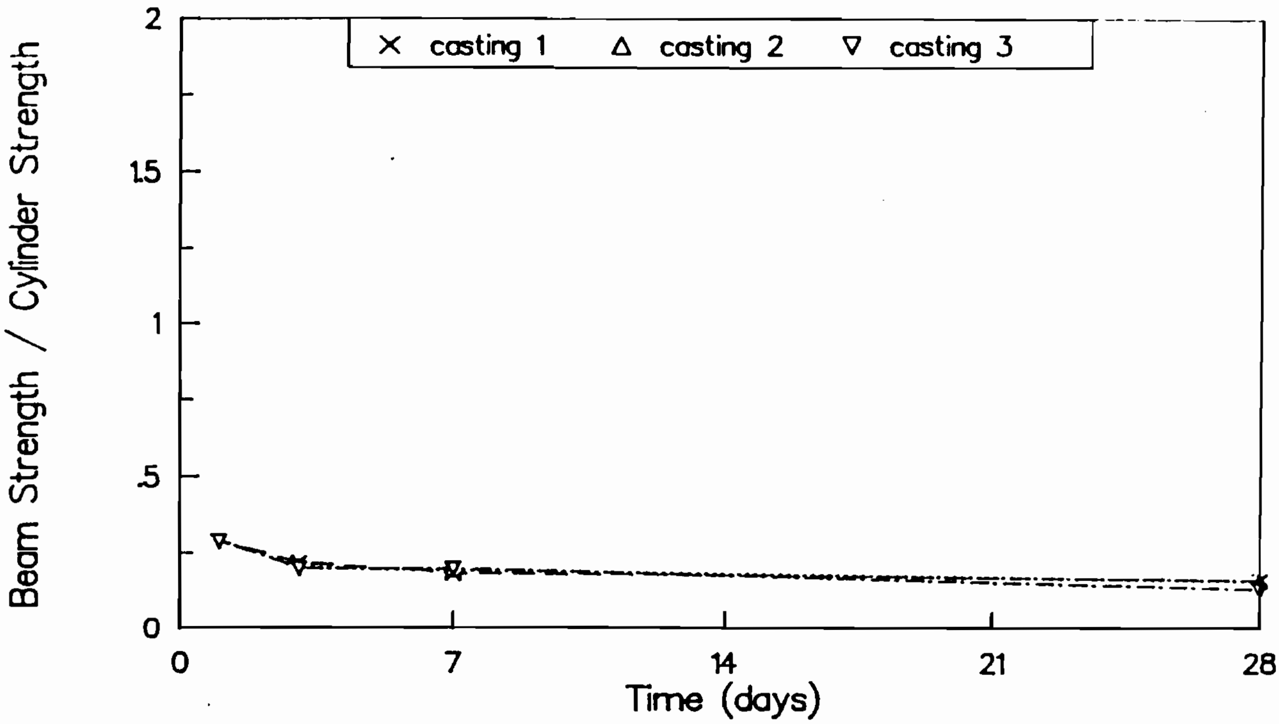


**Fig A-1. Ratio of flexural strengths of wetroom-cured beams to strengths of wetroom-cured cylinders for castings 1, 2, and 3.**

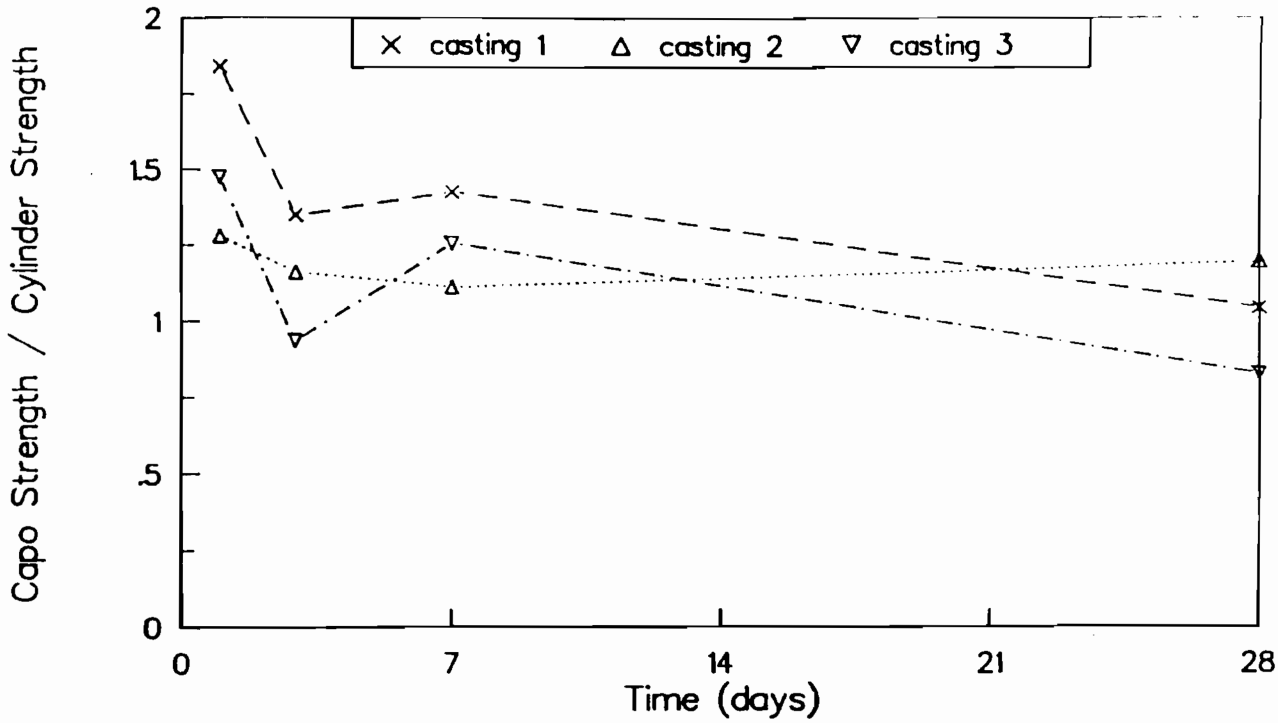


**Fig A-2. Ratio of "CAPO-test" pullout strengths to strengths of wetroom-cured cylinders for castings 1, 2, and 3.**

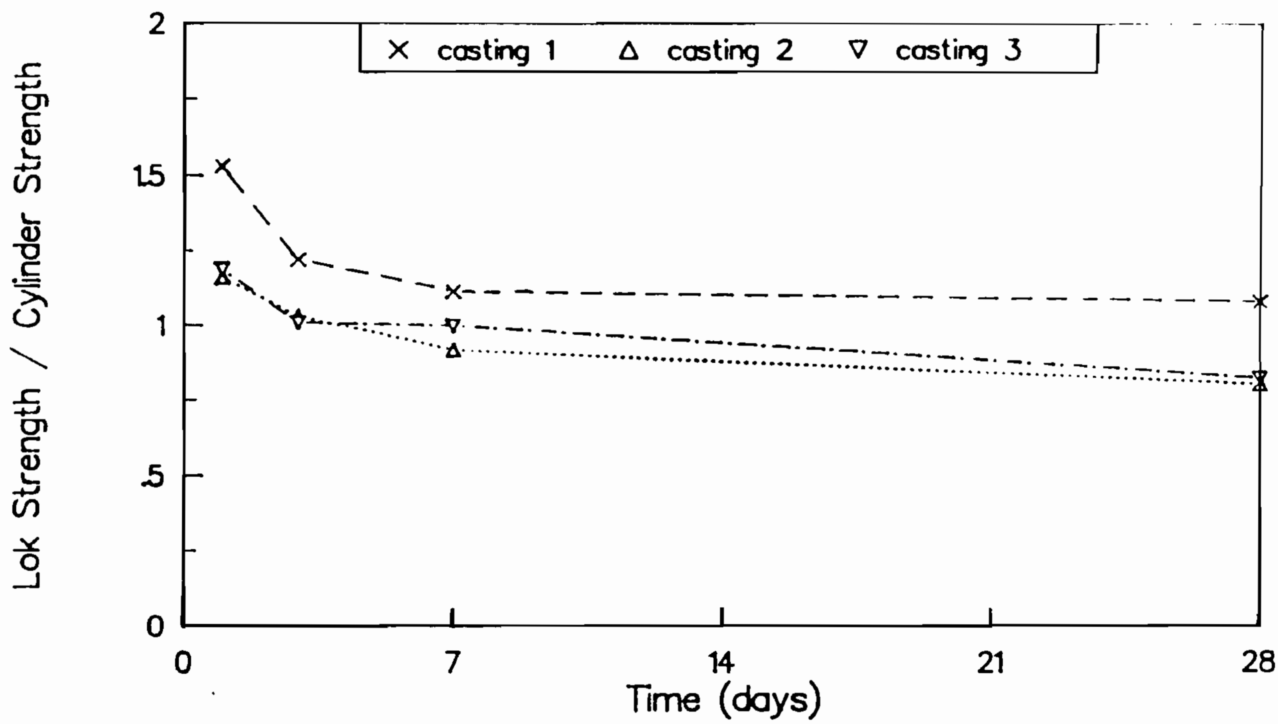


**Fig A-3. Ratio of "Lok-Test" pullout strengths to strengths of wetroom-cured cylinders for castings 1, 2, and 3.**

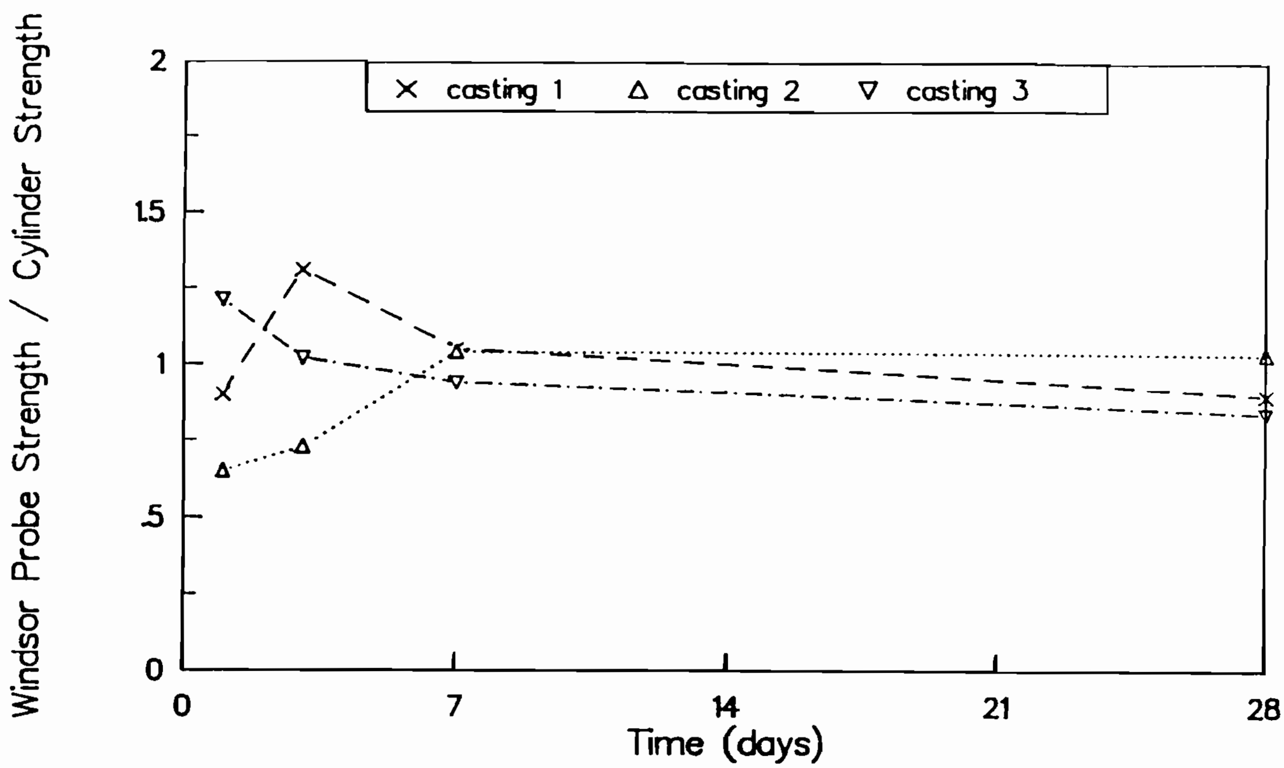


**Fig A-4. Ratio of Windsor probe penetration resistance strengths to strengths of wetroom-cured cylinders for castings 1, 2, and 3.**

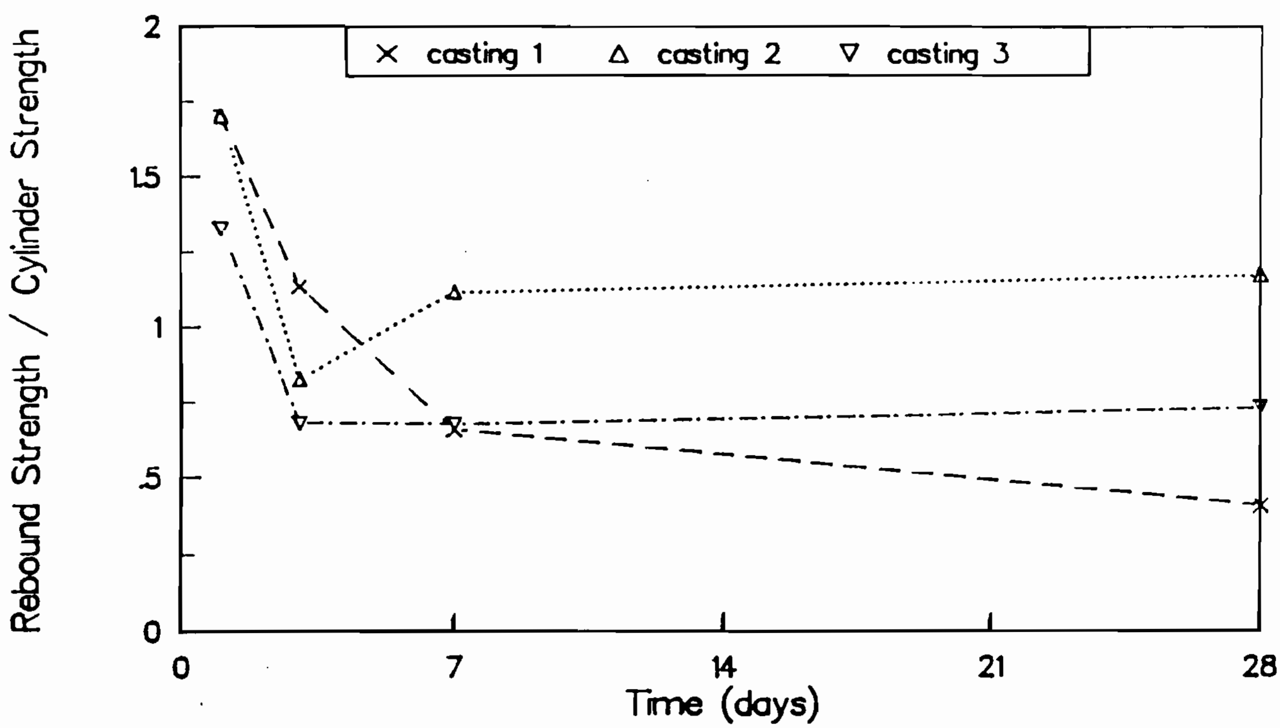


**Fig A-5. Ratio of rebound number strengths to strengths of wetroom-cured cylinders for castings 1, 2, and 3.**

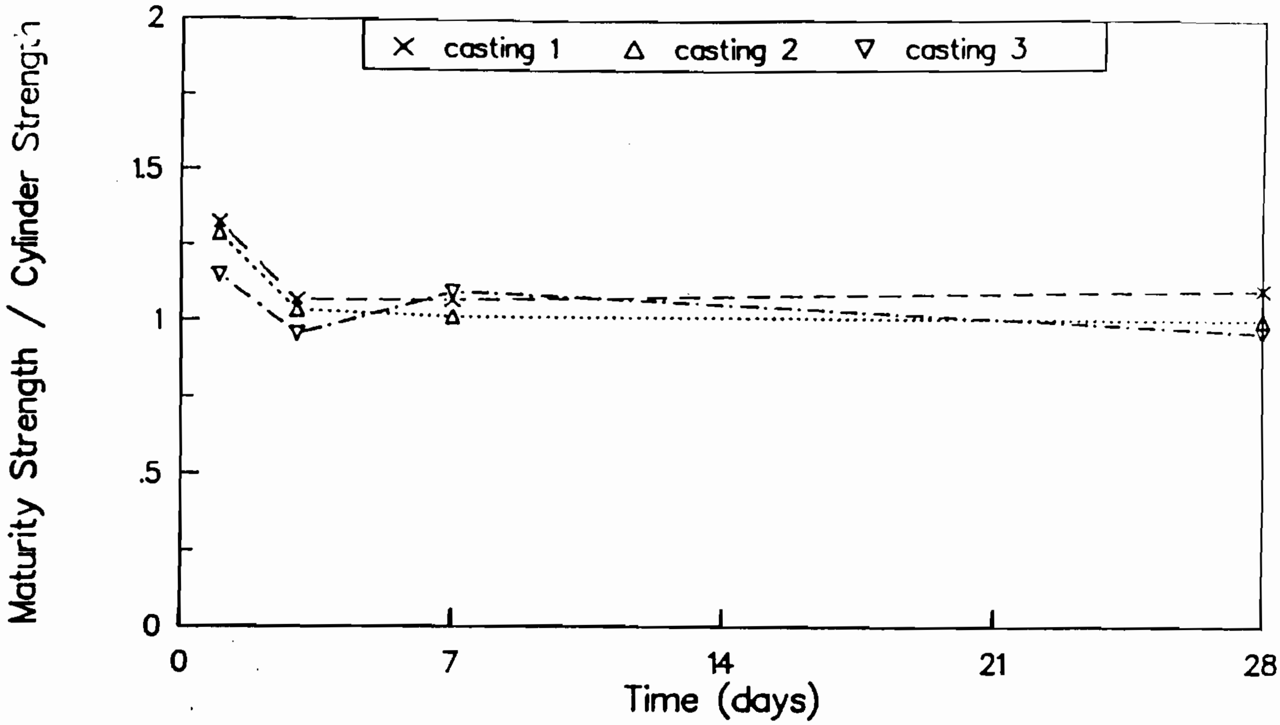


**Fig A-6. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 1, 2, and 3.**

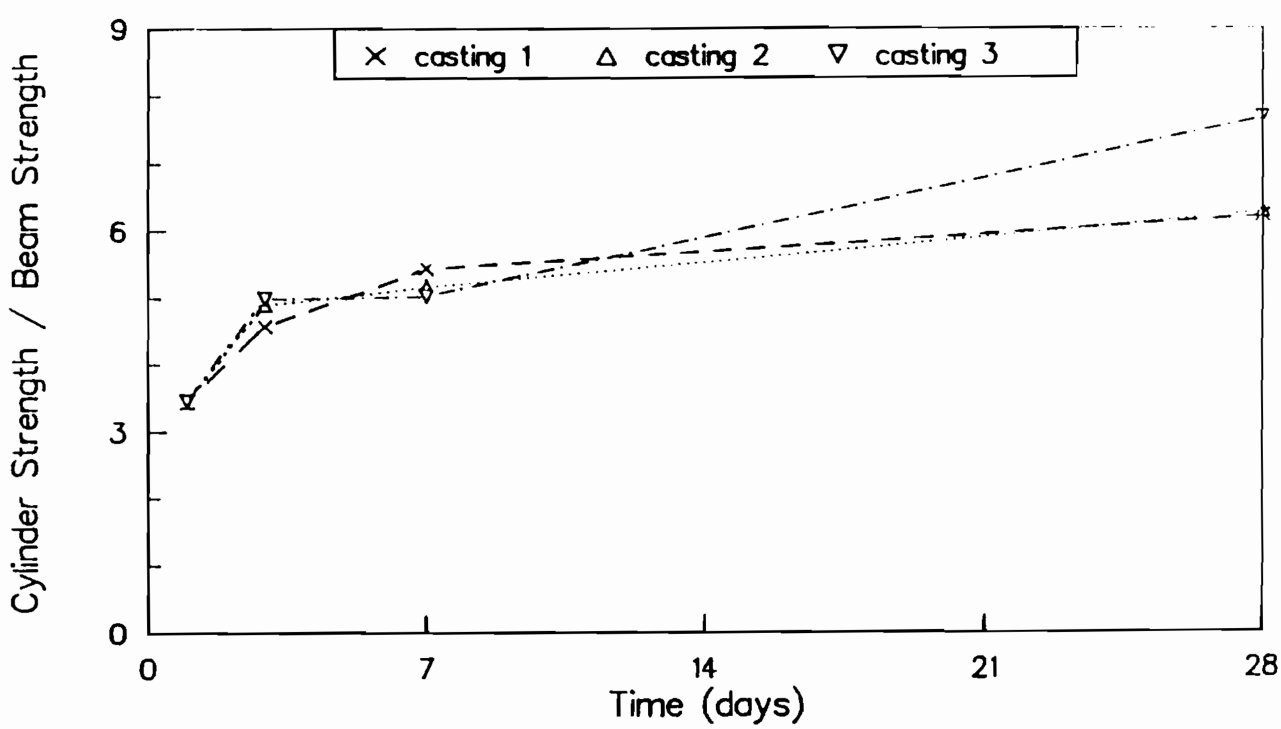


**Fig A-7. Ratio of strengths of wetroom-cured cylinders to flexural strengths of wetroom-cured beams for castings 1, 2, and 3.**

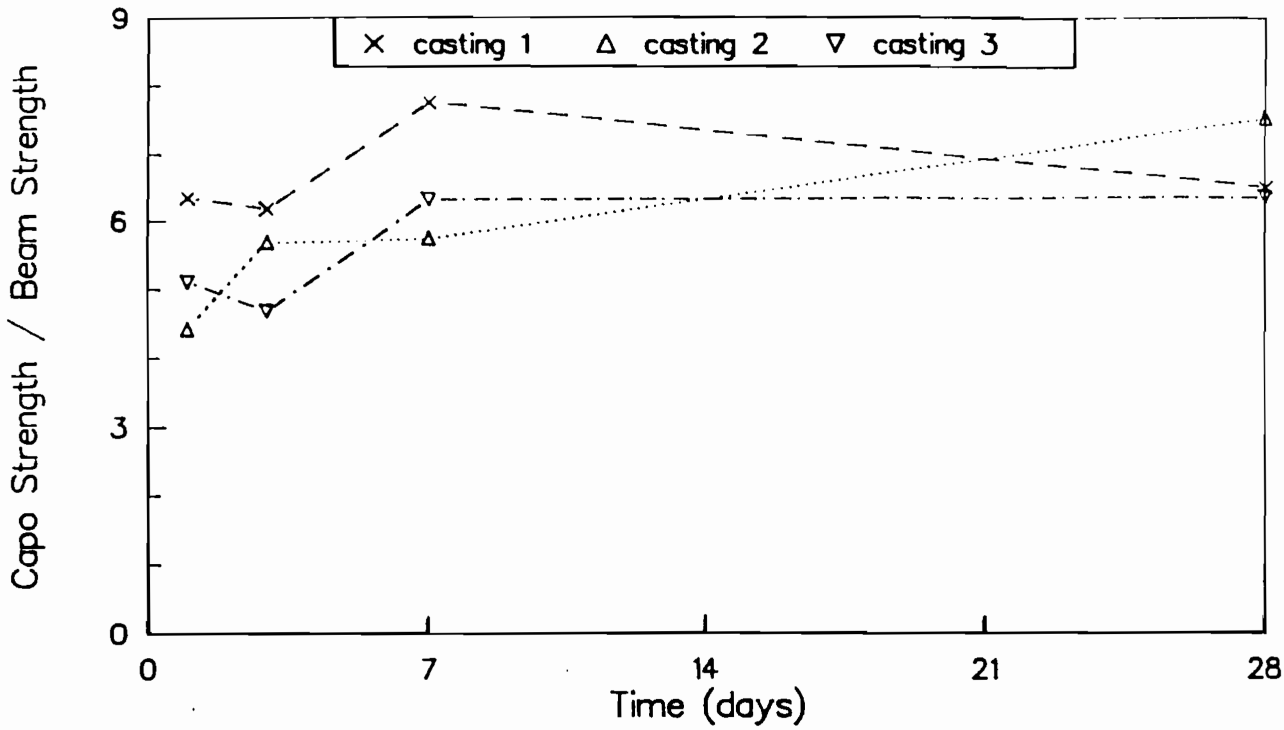


**Fig A-8. Ratio of "CAPO-test" pullout strengths to flexural strengths of wetroom-cured beams for castings 1, 2, and 3.**

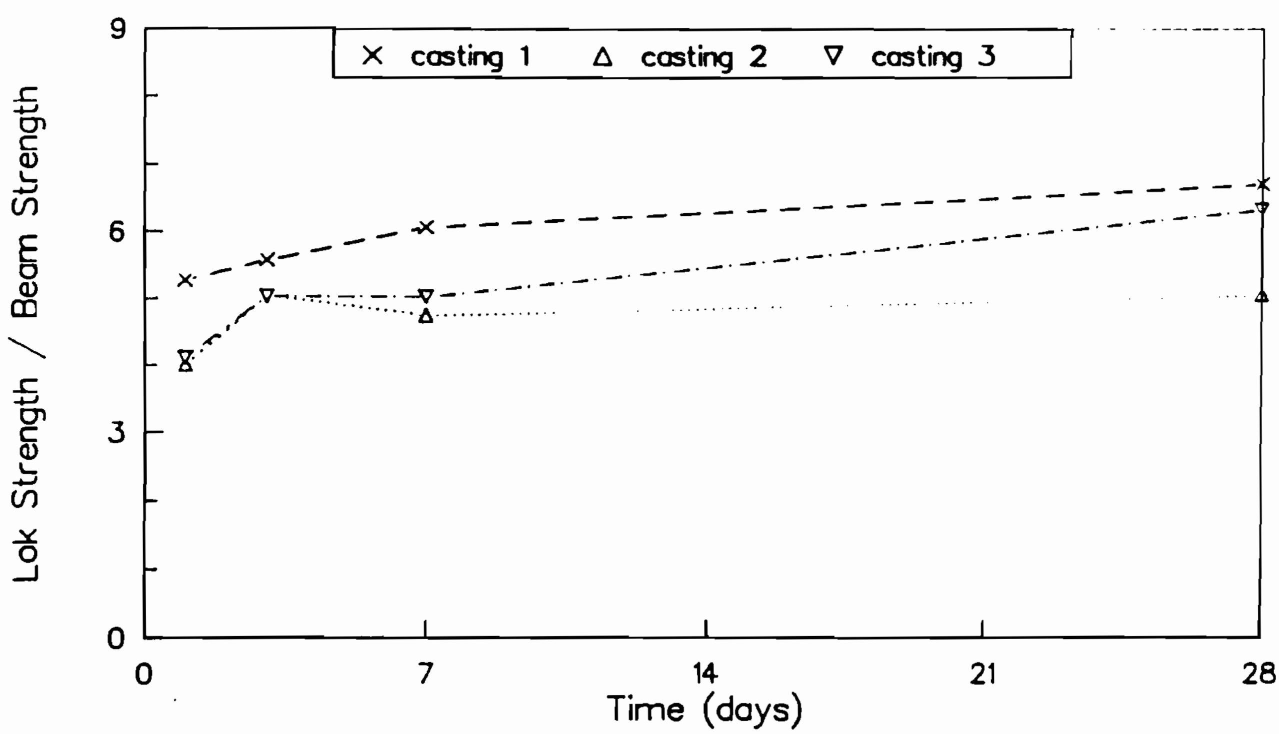


**Fig A-9. Ratio of "Lok-Test" pullout strengths to flexural strengths of wetroom-cured beams for castings 1, 2, and 3.**

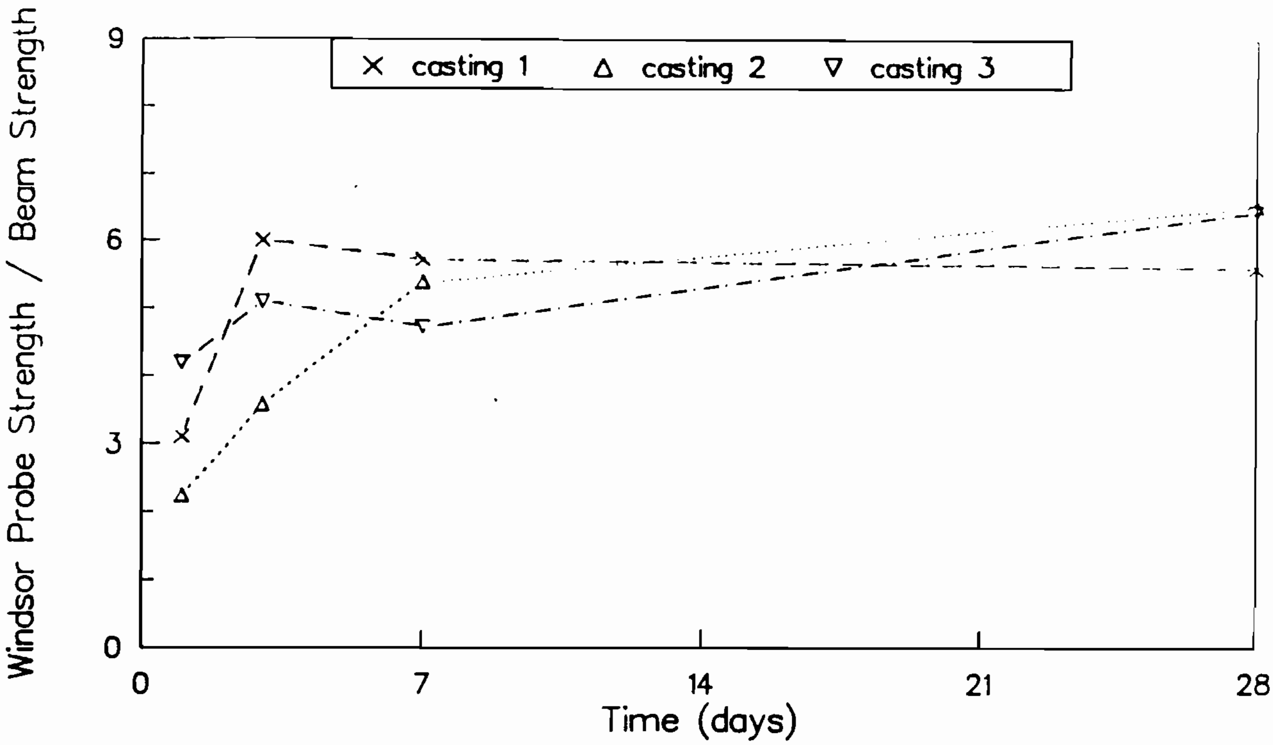


**Fig A-10. Ratio of Windsor probe penetration resistance strengths to flexural strengths of wetroom-cured beams for castings 1, 2, and 3.**

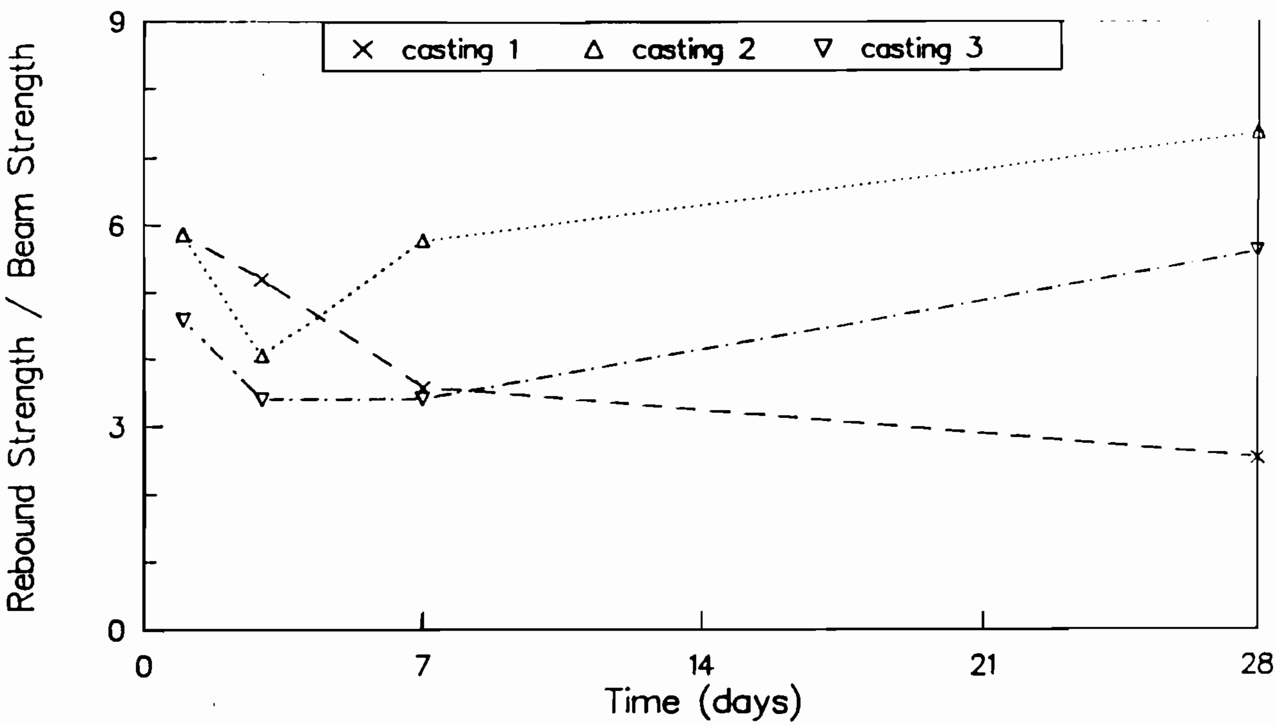


**Fig A-11. Ratio of rebound number strengths to flexural strengths of wetroom-cured beams for castings 1, 2, and 3.**

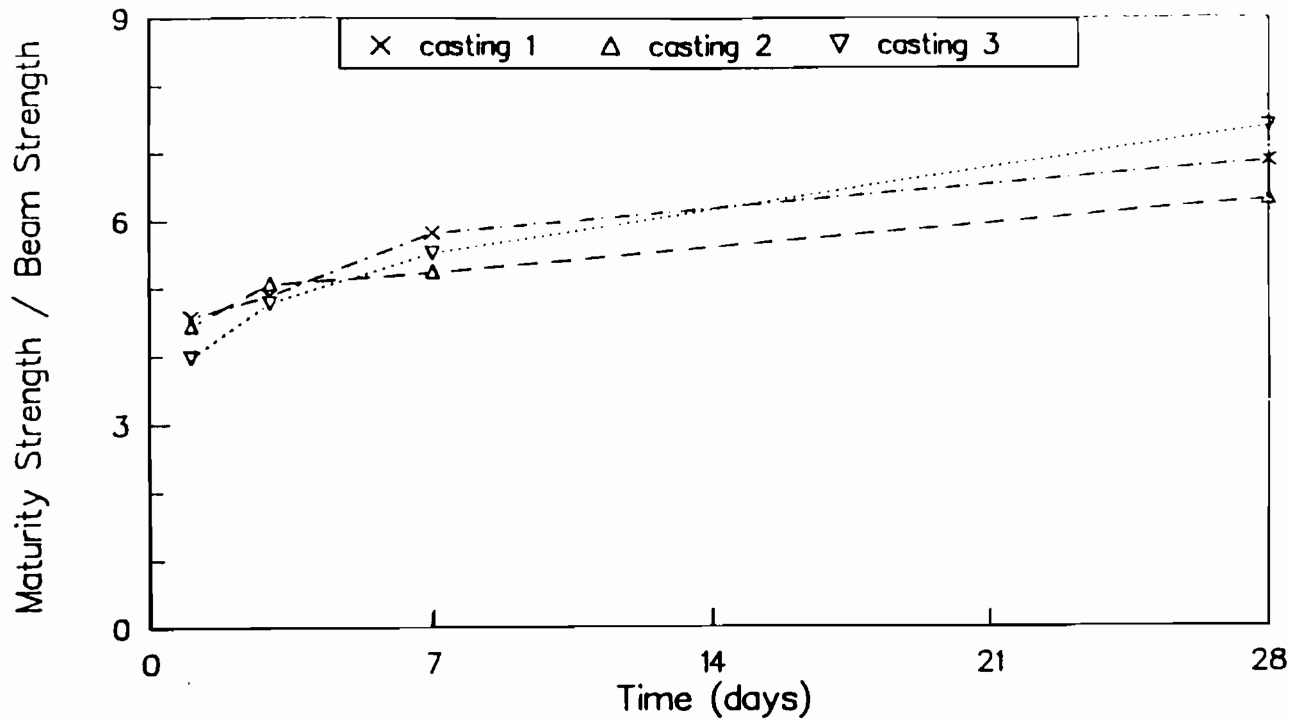


**Fig A-12. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 1, 2, and 3.**

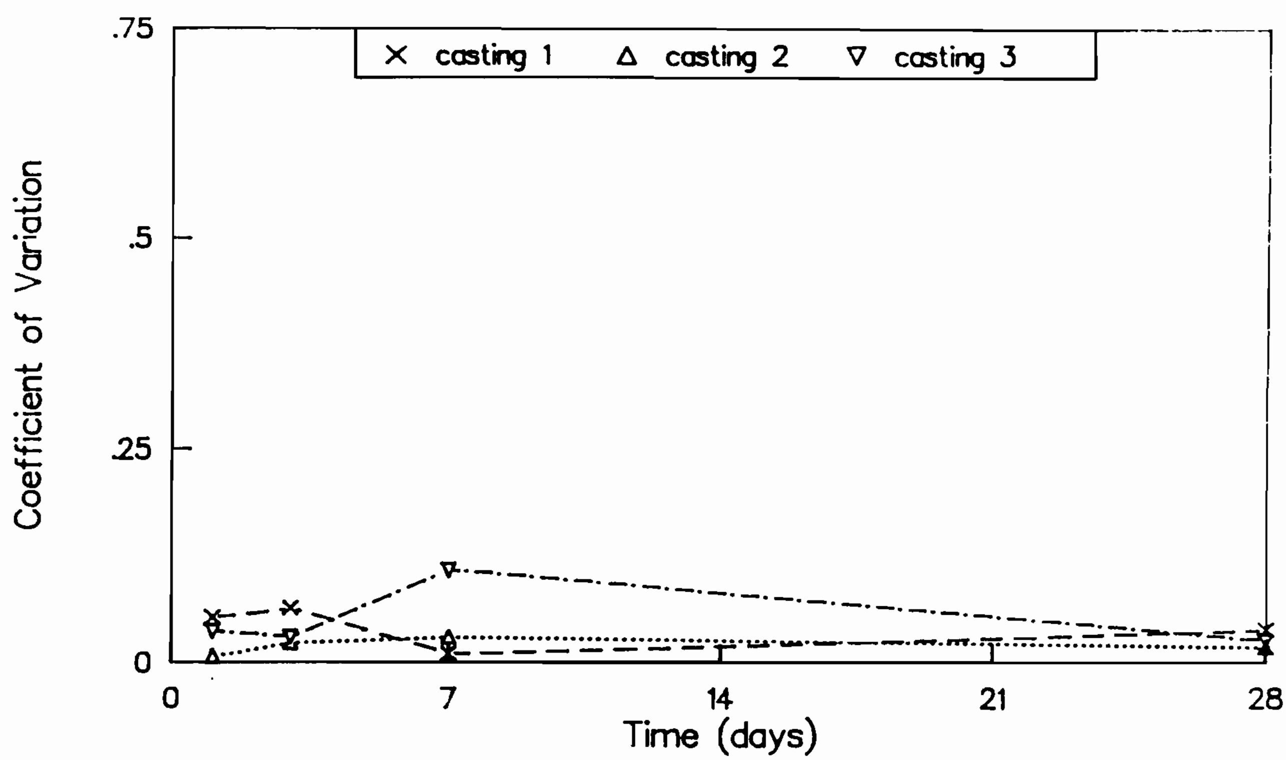


**Fig A-13. Coefficient of variation of wetroom-cured cylinder strengths for castings 1, 2, and 3.**

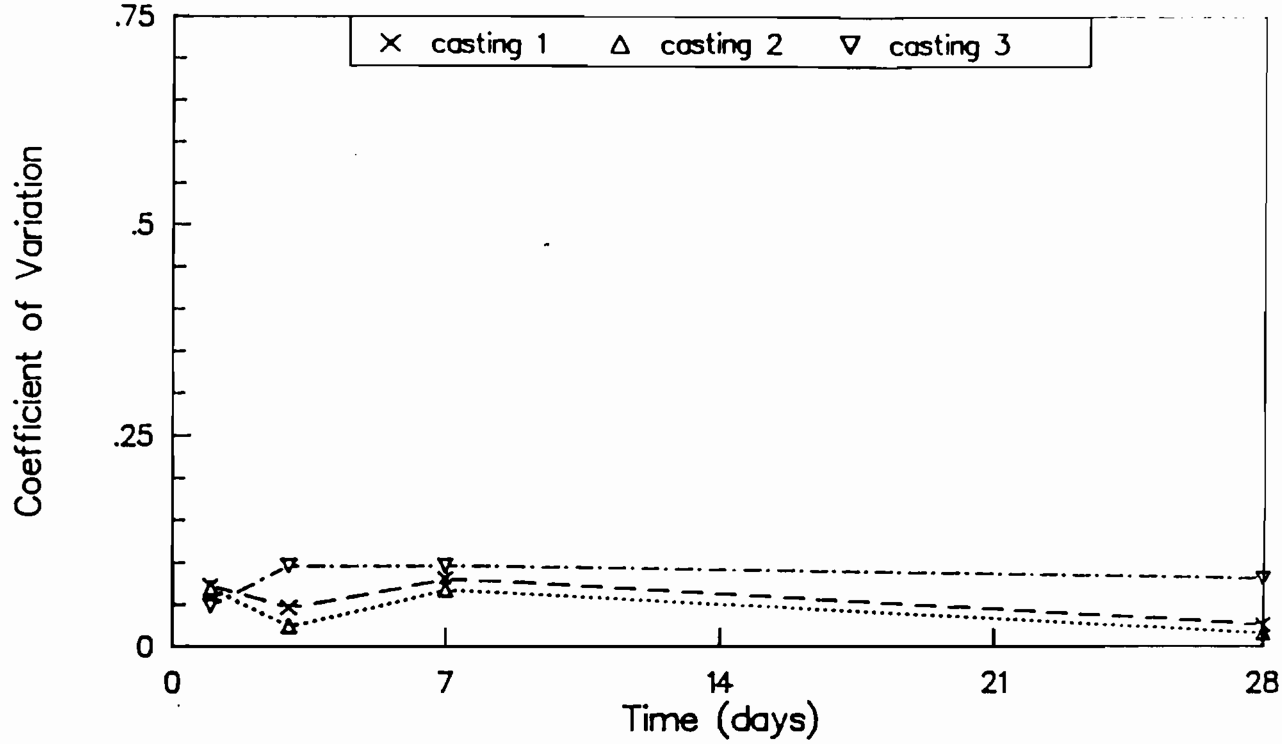


**Fig A-14. Coefficient of variation of flexural strength of wetroom-cured beams for castings 1, 2, and 3.**

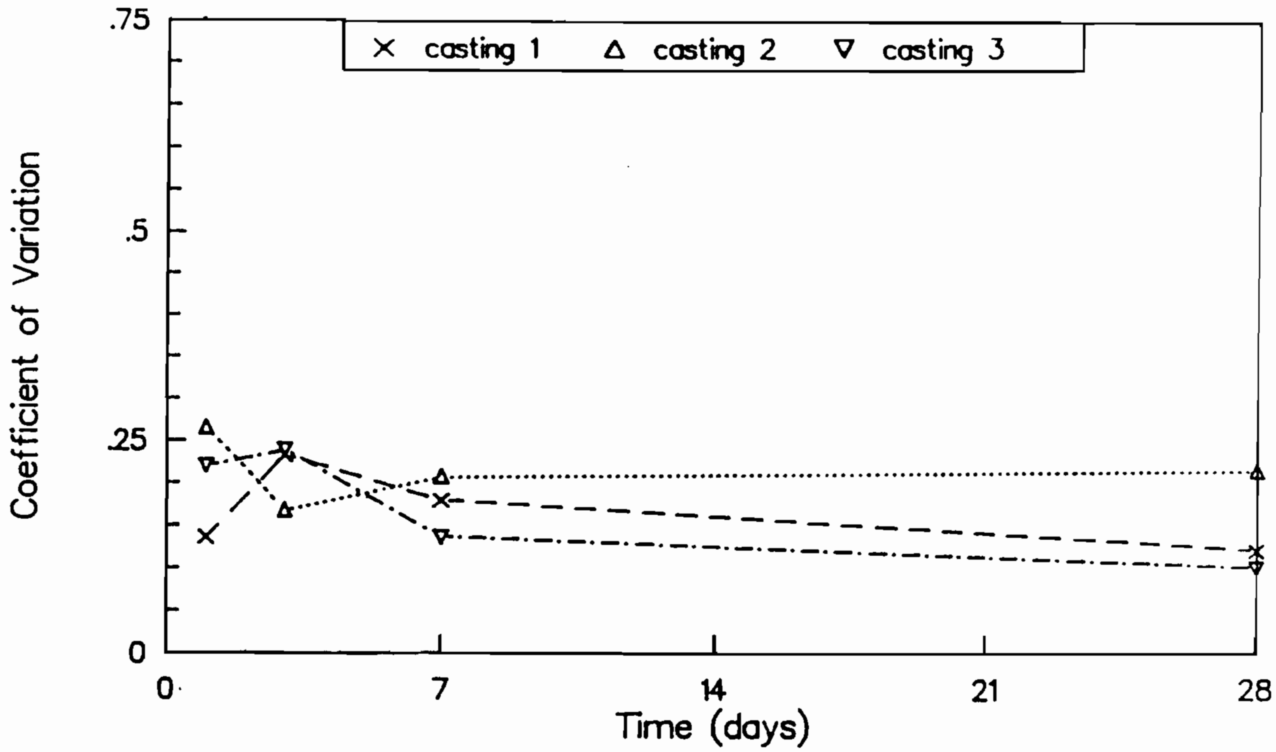


**Fig A-15. Coefficient of variation of "CAPO-test" pullout strengths for castings 1, 2, and 3.**

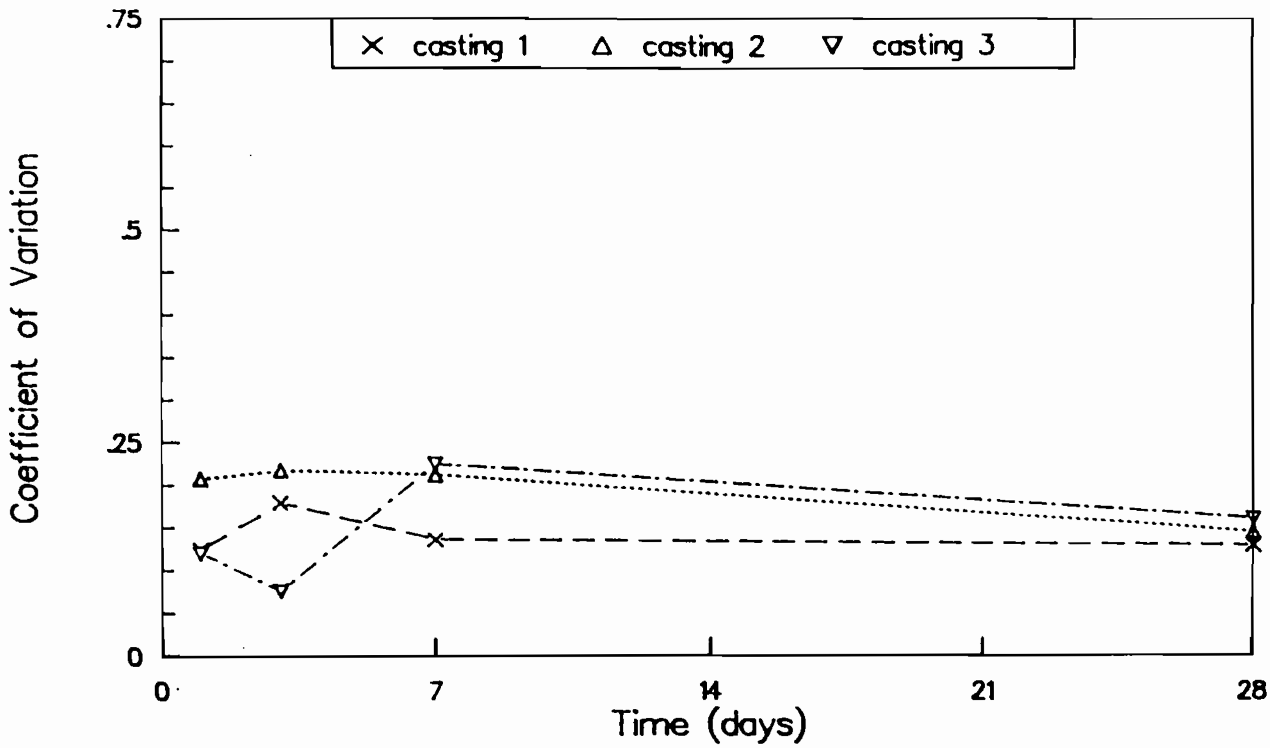


**Fig A-16. Coefficient of variation of "Lok-test" pullout strengths for castings 1, 2, and 3.**

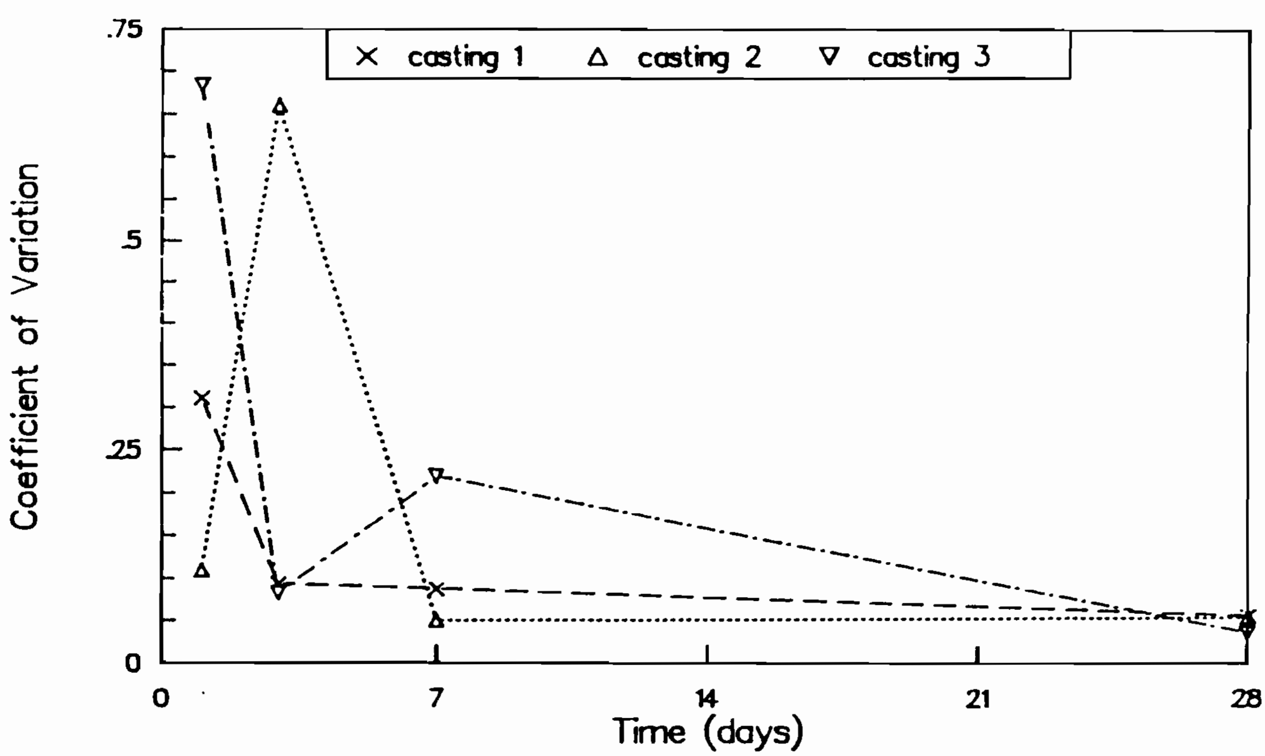


**Fig A-17. Coefficient of variation of Windsor probe penetration resistance strengths for castings 1, 2, and 3.**

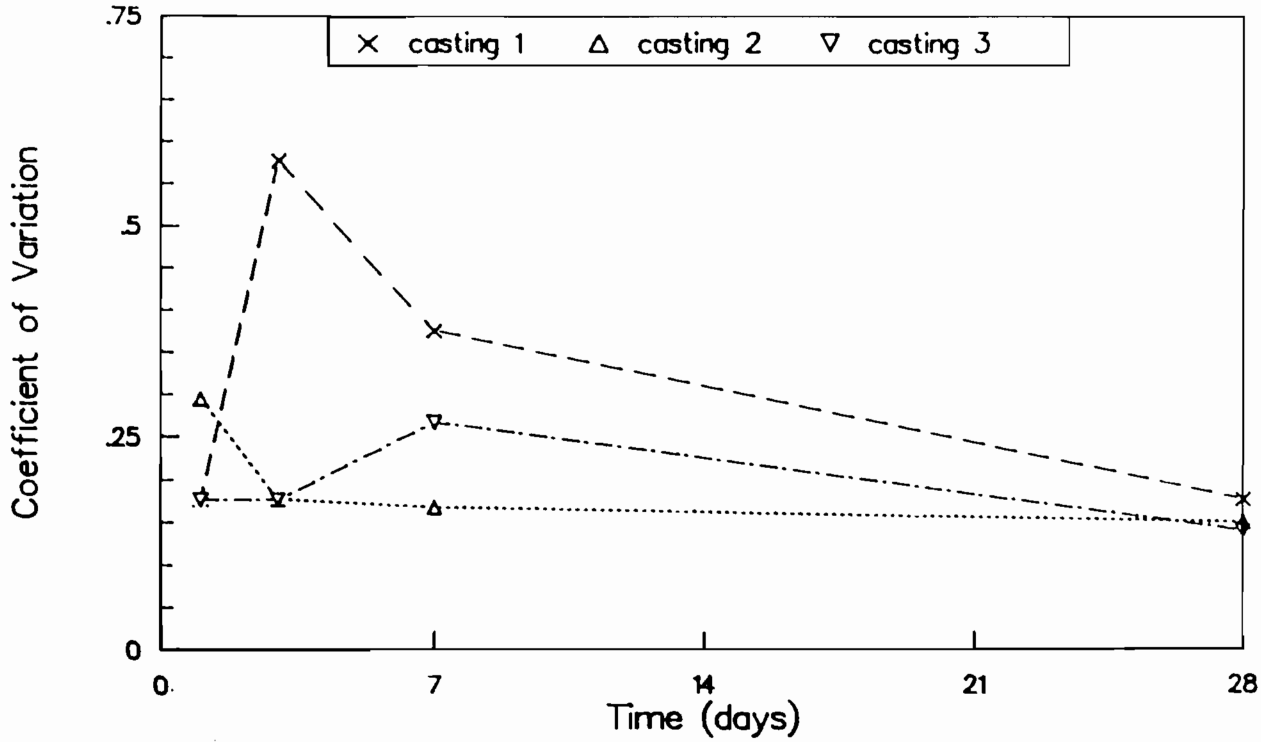


**Fig A-18. Coefficient of variation of rebound number strengths for castings 1, 2, and 3.**

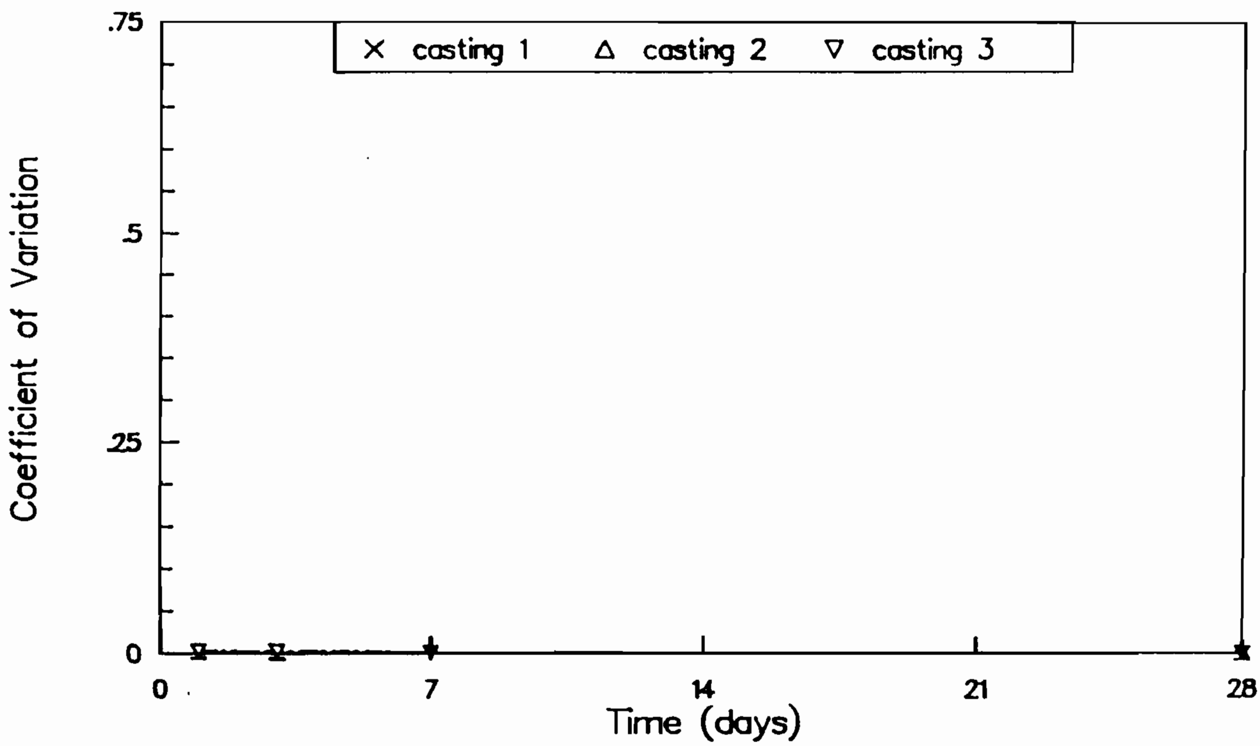


**Fig A-19. Coefficient of variation of maturity method strengths for castings 1, 2, and 3.**

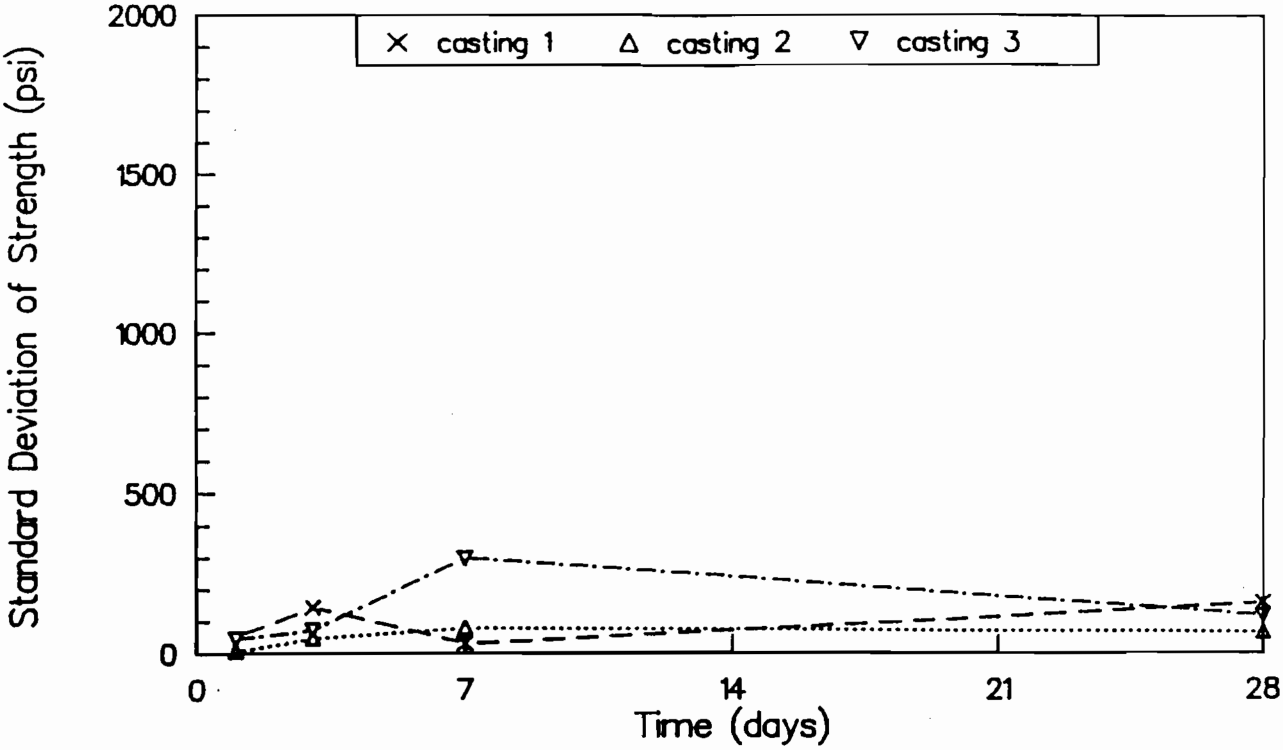


Fig A-20. Standard deviation of wetroom-cured cylinder strengths for castings 1, 2, and 3.

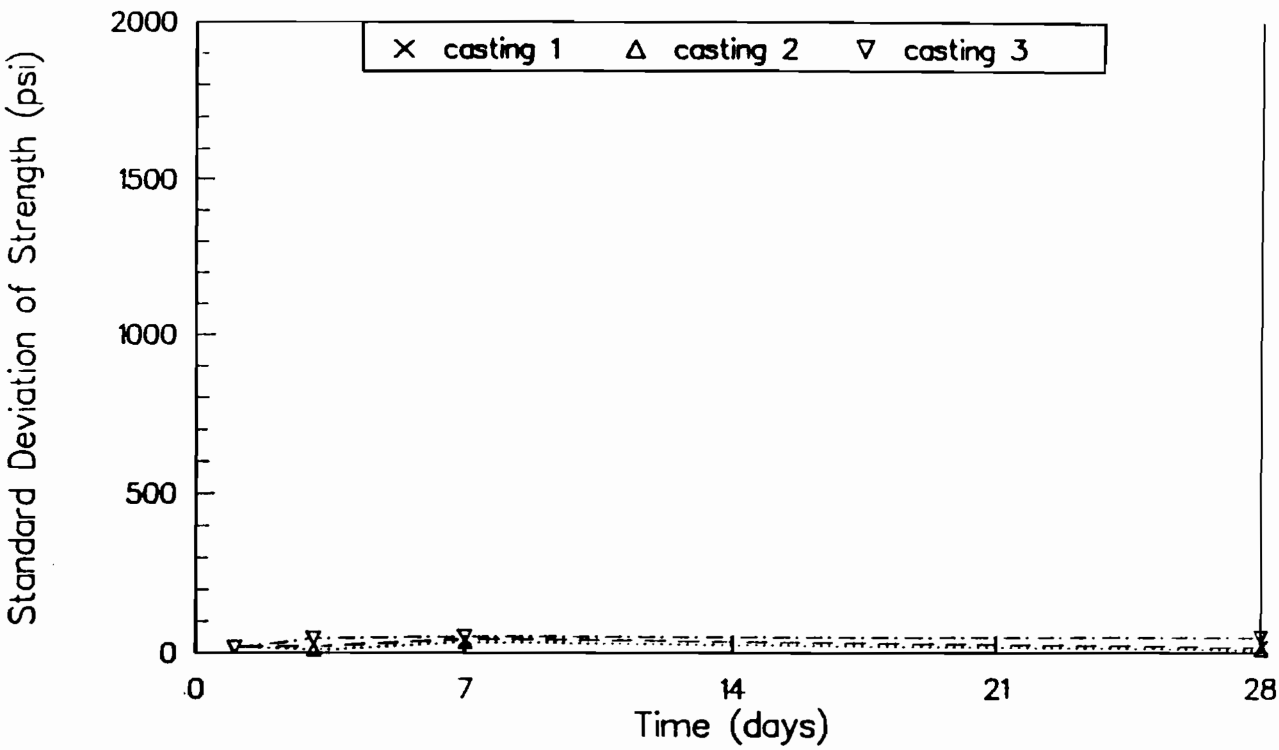


Fig A-21. Standard deviation of flexural strength of wetroom-cured beams for castings 1, 2, and 3.

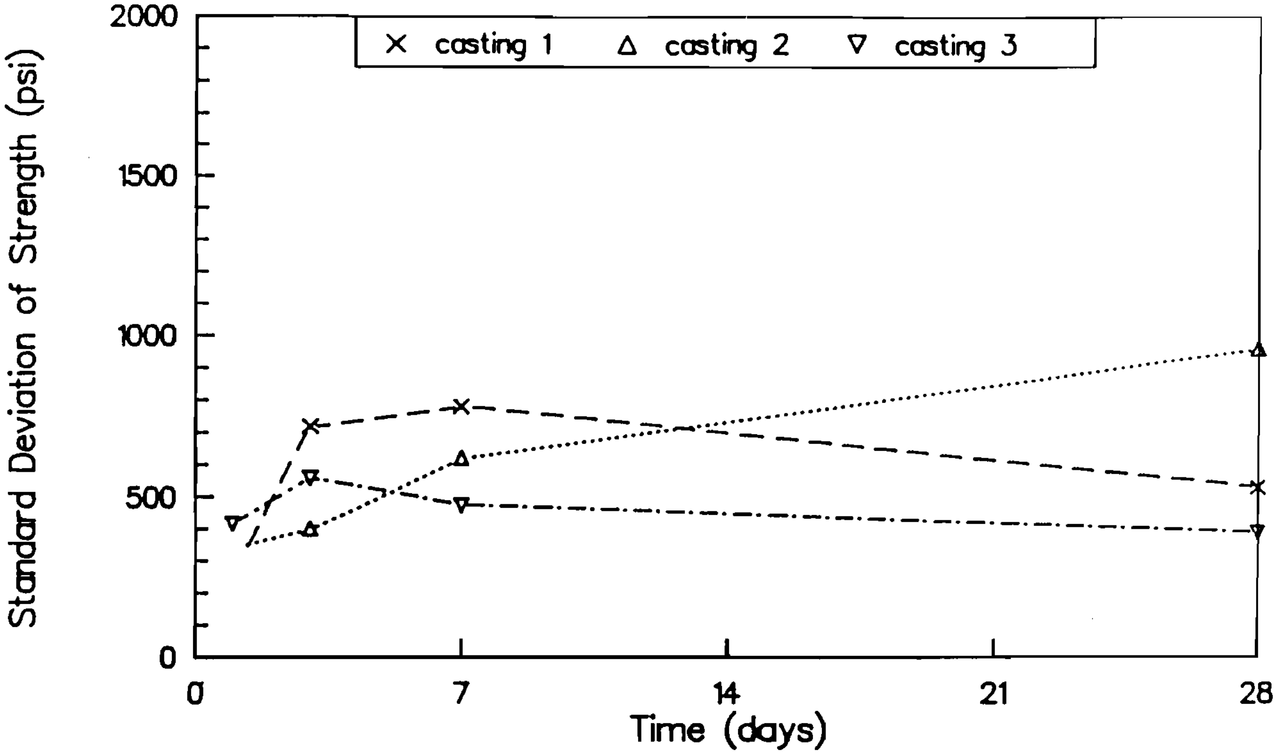


**Fig A-22. Standard deviation of "CAPO-test" pullout strengths for castings 1, 2, and 3.**

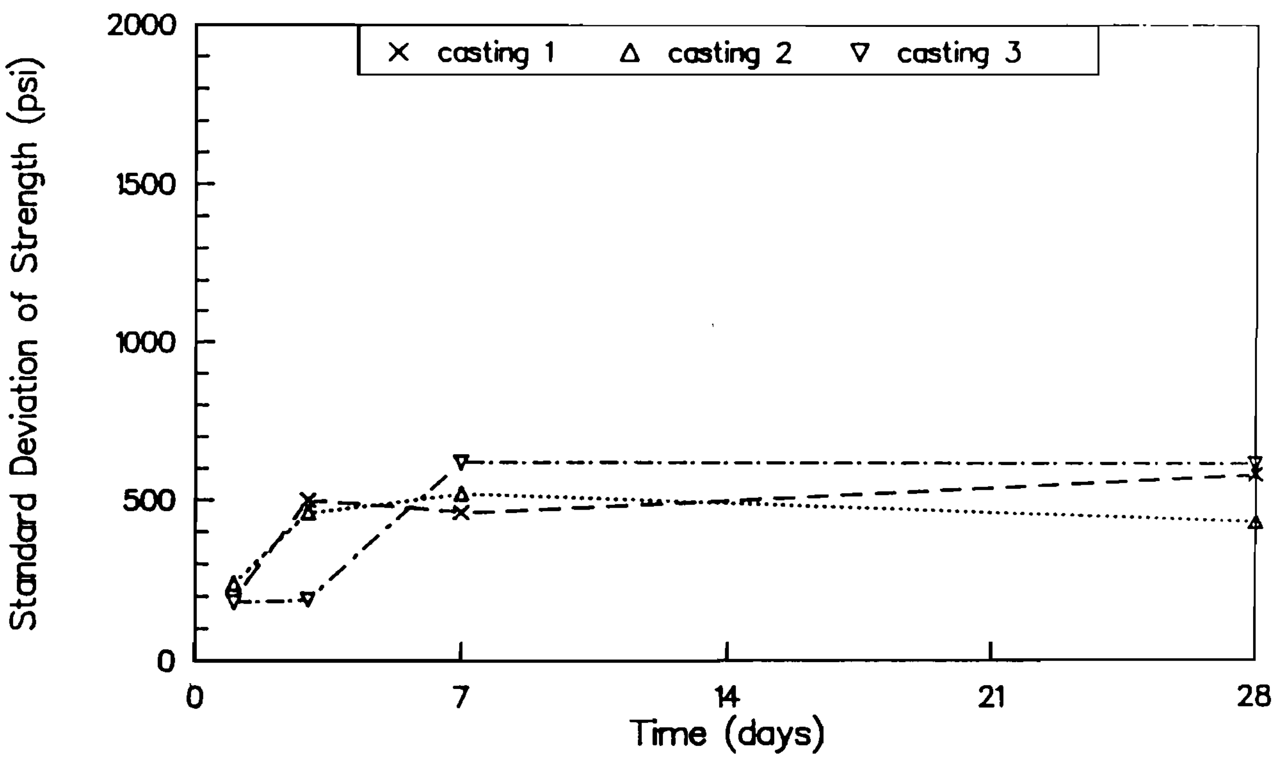


**Fig A-23. Standard deviation of "Lok-Test" pullout strengths for castings 1, 2, and 3.**

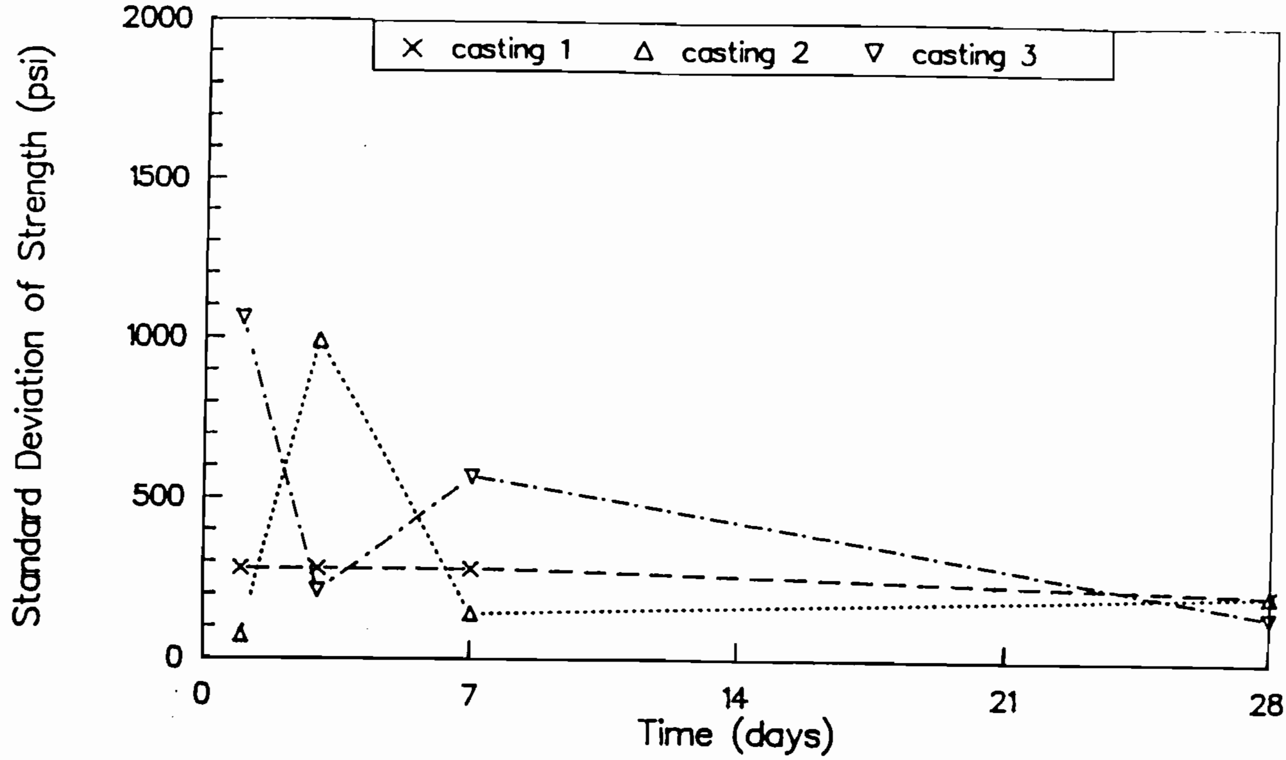


**Fig A-24. Standard deviation of Windsor probe penetration resistance strengths for castings 1, 2, and 3.**

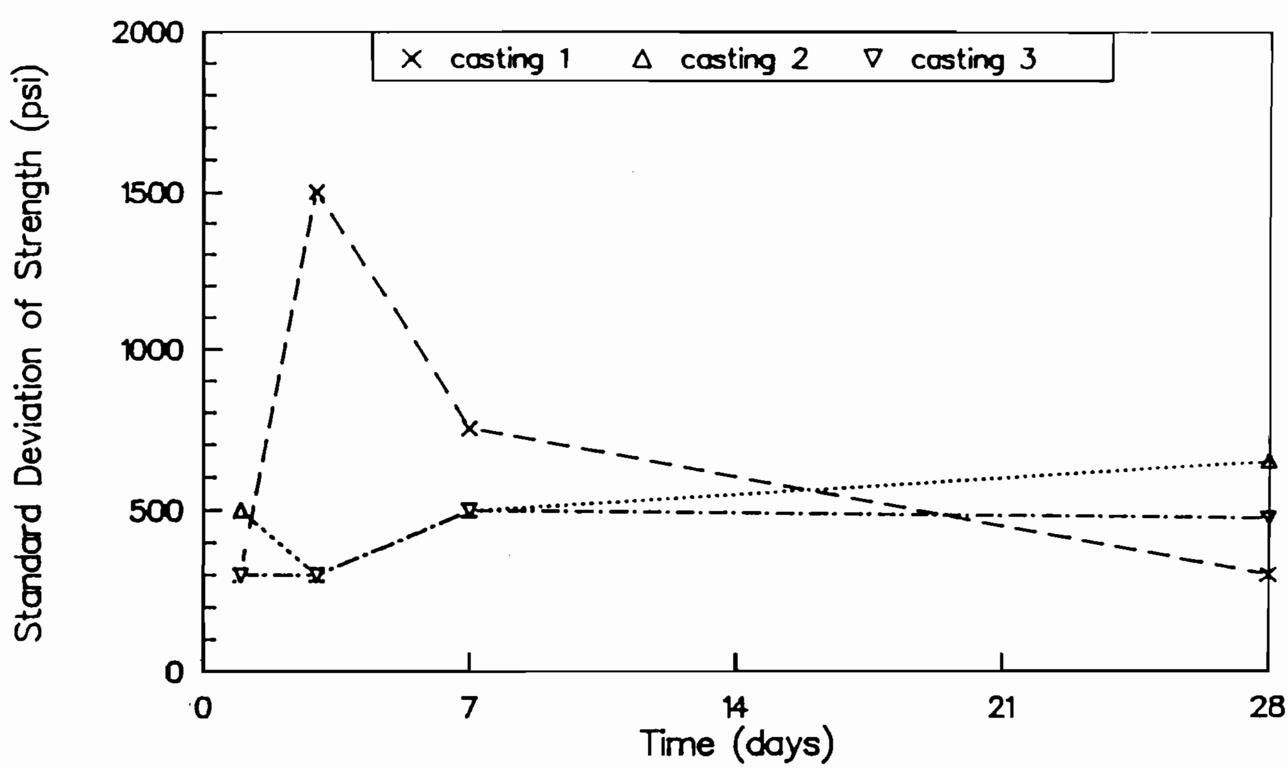


**Fig A-25. Standard deviation of rebound number strengths for castings 1, 2, and 3.**

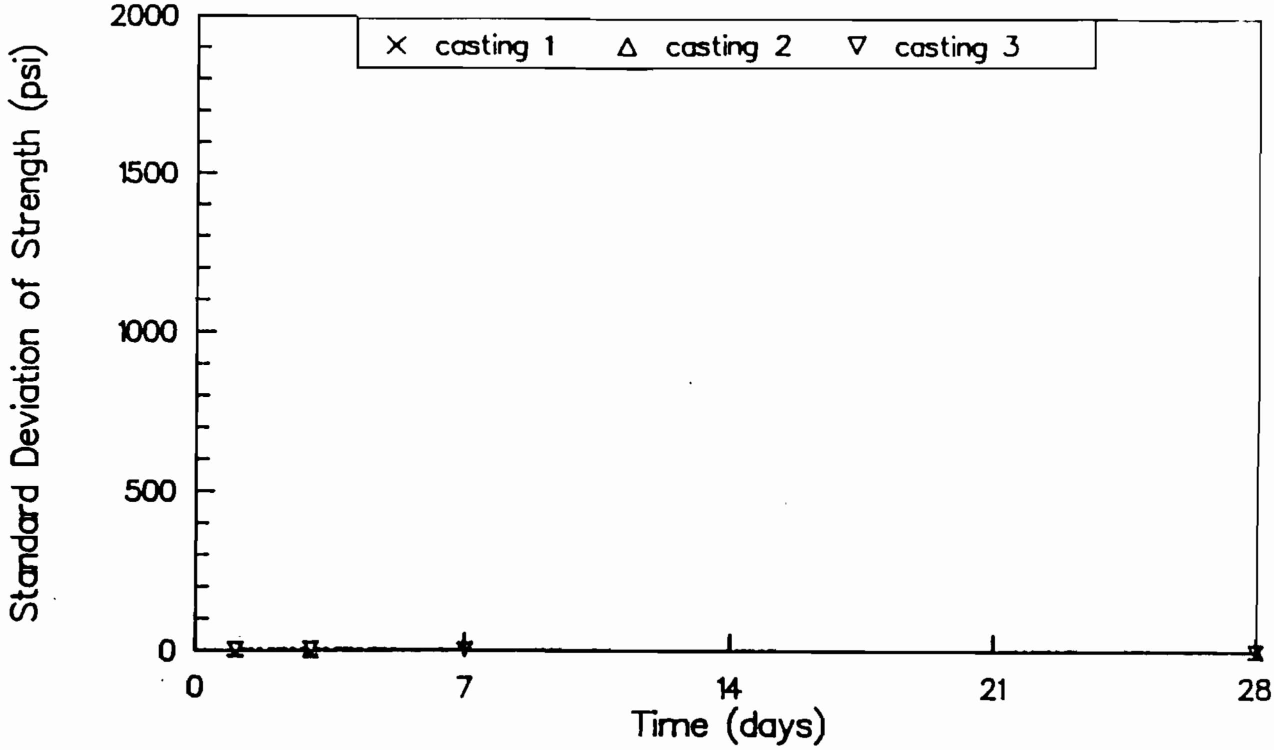


**Fig A-26. Standard deviation of maturity method strengths for castings 1, 2, and 3.**

# APPENDIX B

**TABLE B-1. DATA FOR MATURITY CURVES OF CASTING 4**

| | STRENGTH (psi) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 549 | 2443 | 3766 | 4597 | 5183 |
| CYLINDERS cured outdoors | 489 | 1809 | 3657 | 4164 | 5034 |
| CORES | 846 | 2128 | 3574 | 4058 | 4727 |
| BEAMS cured in wetroom | 153 | 460 | 573 | 633 | 701 |
| BEAMS cured outdoors | 113 | 403 | 553 | 520 | 681 |

| | MATURITY (degree C-Hr) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 592 | 2040 | 4735 | 10555 | 20635 |
| CYLINDERS cured outdoors | 604 | 1560 | 3913 | 10842 | 16373 |
| CORES | 808 | 1945 | 4435 | 8680 | 16675 |
| BEAMS cured in wetroom | 603 | 2062 | 4732 | 10552 | 20632 |
| BEAMS cured outdoors | 509 | 1506 | 3524 | 7921 | 15850 |

| | slope m (psi/log Mat) | intercept b (psi) |
|---|---|---|
| CYLINDERS cured in wetroom | 3045 | -7711 |
| CYLINDERS cured outdoors | 3079 | -7939 |
| CORES | 2983 | -7674 |
| BEAMS cured in wetroom | 234 | -305 |
| BEAMS cured outdoors | 344 | -752 |

NOTE: slope and intercept calculated using logarithmic curve fit. See chapter 5.

**TABLE B-2. DATA FOR MATURITY CURVES OF CASTING 5**

| | STRENGTH (psi) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 2000 | 4675 | 5979 | 6058 | 7008 |
| CYLINDERS cured outdoors | 2138 | 4312 | 5009 | 5175 | 6106 |
| CORES | 2832 | 5073 | 5957 | 6669 | 7023 |
| BEAMS cured in wetroom | --- | 663 | 707 | 786 | 815 |
| BEAMS cured outdoors | 437 | 610 | 598 | 762 | 815 |

| | MATURITY (degree C-Hr) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 835 | 2324 | 5264 | 10244 | 20233 |
| CYLINDERS cured outdoors | 986 | 2503 | 5028 | 8931 | 17241 |
| CORES | 1033 | 2648 | 5264 | 9307 | 16779 |
| BEAMS cured in wetroom | 780 | 2280 | 5190 | 10110 | 20163 |
| BEAMS cured outdoors | 790 | 2204 | 4665 | 8393 | 16173 |

| | slope m (psi/log Mat) | intercept b (psi) |
|---|---|---|
| CYLINDERS cured in wetroom | 3427 | -7419 |
| CYLINDERS cured outdoors | 2962 | -6281 |
| CORES | 3448 | -7135 |
| BEAMS cured in wetroom | 172 | 83 |
| BEAMS cured outdoors | 282 | -374 |

NOTE: slope and intercept calculated using logarithmic curve fit. See chapter 5.

## TABLE B-3. DATA FOR MATURITY CURVES OF CASTING 6

| | STRENGTH (psi) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 1486 | 5052 | 6750 | 7430 | 8235 |
| CYLINDERS cured outdoors | 1070 | 3688 | 6272 | 7420 | 8012 |
| CORES | 1384 | 4608 | 6196 | 6748 | 8018 |
| BEAMS cured in wetroom | 325 | 652 | 783 | 750 | 892 |
| BEAMS cured outdoors | 242 | 581 | 639 | 730 | 664 |

| | MATURITY (degree C-Hr) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 835 | 2324 | 5264 | 10244 | 20233 |
| CYLINDERS cured outdoors | 986 | 2503 | 5028 | 8931 | 17241 |
| CORES | 1033 | 2648 | 5264 | 9307 | 16779 |
| BEAMS cured in wetroom | 780 | 2280 | 5190 | 10110 | 20163 |
| BEAMS cured outdoors | 790 | 2204 | 4665 | 8393 | 16173 |

| | slope m (psi/log Mat) | intercept b (psi) |
|---|---|---|
| CYLINDERS cured in wetroom | 4762 | -11667 |
| CYLINDERS cured outdoors | 5839 | -16054 |
| CORES | 5299 | -14047 |
| BEAMS cured in wetroom | 369 | -669 |
| BEAMS cured outdoors | 328 | -612 |

NOTE: slope and intercept calculated using logarithmic curve fit. See chapter 5.

## TABLE B-4. CASTINGS 4, 5, AND 6, RATIOS OF VARIOUS STRENGTHS TO FOUR STANDARDS, AT 1 DAY

| | CYLINDERS | | CORES | BEAMS | | PULLOUT | PEN. RESIST. |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors | LOK-Test | Windsor Probe |
| **CASTING 4** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.89 | 1.54 | 0.28 | 0.21 | 0.83 | 0.00 |
| ratio to strength of outdoor cylinders | 1.12 | 1.00 | 1.73 | 0.31 | 0.23 | 0.93 | 0.00 |
| ratio to strength of wetroom beams | 3.58 | 3.19 | 5.52 | 1.00 | 0.74 | 2.98 | 0.00 |
| ratio to strength of outdoor beams | 4.84 | 4.32 | 7.47 | 1.35 | 1.00 | 4.03 | 0.00 |
| **CASTING 5** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 1.07 | 1.42 | --- | 0.22 | 1.07 | 0.00 |
| ratio to strength of outdoor cylinders | 0.94 | 1.00 | 1.32 | --- | 0.20 | 1.00 | 0.00 |
| ratio to strength of wetroom beams | --- | --- | --- | --- | --- | --- | --- |
| ratio to strength of outdoor beams | 4.58 | 4.90 | 6.49 | --- | 1.00 | 4.92 | 0.00 |
| **CASTING 6** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.72 | 0.93 | 0.22 | 0.16 | 0.00 | 0.66 |
| ratio to strength of outdoor cylinders | 1.39 | 1.00 | 1.29 | 0.30 | 0.23 | 0.00 | 0.91 |
| ratio to strength of wetroom beams | 4.57 | 3.29 | 4.26 | 1.00 | 0.74 | 0.00 | 3.00 |
| ratio to strength of outdoor beams | 6.15 | 4.43 | 5.73 | 1.34 | 1.00 | 0.00 | 4.03 |

TABLE B-5. CASTINGS 4, 5, AND 6, RATIOS OF MATURITY STRENGTHS TO FOUR STANDARDS, AT 1 DAY

| | MATURITY STRENGTHS LISTED BY BASIS OF MATURITY CURVE | | | | |
|---|---|---|---|---|---|
| | CYLINDERS | | CORES | BEAMS | |
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors |
| **CASTING 4** | | | | | |
| ratio to strength of wetroom cylinders | 1.33 | 1.12 | 1.82 | 0.63 | 0.33 |
| ratio to strength of outdoor cylinders | 1.49 | 1.26 | 2.04 | 0.71 | 0.37 |
| ratio to strength of wetroom beams | 4.76 | 4.01 | 6.51 | 2.25 | 1.17 |
| ratio to strength of outdoor beams | 6.44 | 5.42 | 8.81 | 3.05 | 1.58 |
| **CASTING 5** | | | | | |
| ratio to strength of wetroom cylinders | 1.30 | 1.29 | 1.62 | 0.29 | 0.22 |
| ratio to strength of outdoor cylinders | 1.21 | 1.21 | 1.52 | 0.27 | 0.21 |
| ratio to strength of wetroom beams | --- | --- | --- | --- | --- |
| ratio to strength of outdoor beams | 5.93 | 5.90 | 7.44 | 1.33 | 1.01 |
| **CASTING 6** | | | | | |
| ratio to strength of wetroom cylinders | 1.37 | 0.99 | 1.21 | 0.36 | 0.23 |
| ratio to strength of outdoor cylinders | 1.91 | 1.37 | 1.68 | 0.50 | 0.33 |
| ratio to strength of wetroom beams | 6.28 | 4.51 | 5.53 | 1.66 | 1.07 |
| ratio to strength of outdoor beams | 8.44 | 6.07 | 7.44 | 2.23 | 1.44 |

TABLE B-6. CASTINGS 4, 5, AND 6, RATIOS OF VARIOUS STRENGTHS TO FOUR STANDARDS, AT 3 DAYS

| | CYLINDERS | | CORES | BEAMS | | PULLOUT | PEN. RESIST. |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors | LOK-Test | Windsor Probe |
| CASTING 4 | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.74 | 0.87 | 0.19 | 0.17 | 0.64 | 0.61 |
| ratio to strength of outdoor cylinders | 1.35 | 1.00 | 1.18 | 0.25 | 0.22 | 0.87 | 0.83 |
| ratio to strength of wetroom beams | 5.31 | 3.93 | 4.63 | 1.00 | 0.88 | 3.41 | 3.26 |
| ratio to strength of outdoor beams | 6.06 | 4.49 | 5.28 | 1.14 | 1.00 | 3.89 | 3.72 |
| CASTING 5 | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.92 | 1.09 | 0.14 | 0.13 | 0.83 | 0.00 |
| ratio to strength of outdoor cylinders | 1.08 | 1.00 | 1.18 | 0.15 | 0.14 | 0.90 | 0.00 |
| ratio to strength of wetroom beams | 7.05 | 6.50 | 7.65 | 1.00 | 0.92 | 5.87 | 0.00 |
| ratio to strength of outdoor beams | 7.66 | 7.07 | 8.32 | 1.09 | 1.00 | 6.39 | 0.00 |
| CASTING 6 | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.73 | 0.91 | 0.13 | 0.12 | 0.73 | 0.65 |
| ratio to strength of outdoor cylinders | 1.37 | 1.00 | 1.25 | 0.18 | 0.16 | 1.00 | 0.89 |
| ratio to strength of wetroom beams | 7.75 | 5.66 | 7.07 | 1.00 | 0.89 | 5.64 | 5.06 |
| ratio to strength of outdoor beams | 8.69 | 6.35 | 7.93 | 1.12 | 1.00 | 6.32 | 5.68 |

TABLE B-7. CASTINGS 4, 5, AND 6, RATIOS OF MATURITY STRENGTHS TO FOUR STANDARDS, AT 3 DAYS

| | MATURITY STRENGTHS LISTED BY BASIS OF MATURITY CURVE | | | | |
|---|---|---|---|---|---|
| | CYLINDERS | | CORES | BEAMS | |
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors |
| **CASTING 4** | | | | | |
| ratio to strength of wetroom cylinders | 0.97 | 0.77 | 0.87 | 0.19 | 0.14 |
| ratio to strength of outdoor cylinders | 1.31 | 1.04 | 1.18 | 0.26 | 0.19 |
| ratio to strength of wetroom beams | 5.14 | 4.11 | 4.64 | 1.02 | 0.74 |
| ratio to strength of outdoor beams | 5.86 | 4.69 | 5.30 | 1.17 | 0.85 |
| **CASTING 5** | | | | | |
| ratio to strength of wetroom cylinders | 0.88 | 0.81 | 1.00 | 0.14 | 0.22 |
| ratio to strength of outdoor cylinders | 0.95 | 0.88 | 1.08 | 0.15 | 0.13 |
| ratio to strength of wetroom beams | 6.21 | 5.70 | 7.02 | 0.99 | 0.86 |
| ratio to strength of outdoor beams | 6.75 | 6.20 | 7.63 | 1.08 | 0.93 |
| **CASTING 6** | | | | | |
| ratio to strength of wetroom cylinders | 0.89 | 0.72 | 0.83 | 0.13 | 0.09 |
| ratio to strength of outdoor cylinders | 1.22 | 0.99 | 1.13 | 0.18 | 0.13 |
| ratio to strength of wetroom beams | 6.88 | 5.60 | 6.40 | 1.01 | 0.73 |
| ratio to strength of outdoor beams | 7.71 | 6.28 | 7.17 | 1.13 | 0.82 |

TABLE B-8. CASTINGS 4, 5, AND 6, RATIOS OF VARIOUS STRENGTHS TO FOUR STANDARDS AT 7 DAYS

| | CYLINDERS | | CORES | BEAMS | | PULLOUT | PEN. RESIST. |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors | LOK-Test | Windsor Probe |
| **CASTING 4** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.97 | 0.95 | 0.15 | 0.15 | 0.69 | 0.60 |
| ratio to strength of outdoor cylinders | 1.03 | 1.00 | 0.98 | 0.16 | 0.15 | 0.71 | 0.62 |
| ratio to strength of wetroom beams | 6.57 | 6.38 | 6.23 | 1.00 | 0.97 | 4.51 | 3.95 |
| ratio to strength of outdoor beams | 6.81 | 6.61 | 6.46 | 1.04 | 1.00 | 4.67 | 4.09 |
| **CASTING 5** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.84 | 1.00 | 0.12 | 0.10 | 0.89 | 0.00 |
| ratio to strength of outdoor cylinders | 1.19 | 1.00 | 1.19 | 0.14 | 0.12 | 1.07 | 0.00 |
| ratio to strength of wetroom beams | 8.46 | 7.09 | 8.43 | 1.00 | 0.83 | 7.57 | 0.00 |
| ratio to strength of outdoor beams | 10.19 | 8.54 | 10.15 | 1.20 | 1.00 | 9.12 | 0.00 |
| **CASTING 6** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.93 | 0.92 | 0.12 | 0.09 | 0.74 | 0.55 |
| ratio to strength of outdoor cylinders | 1.08 | 1.00 | 0.99 | 0.12 | 0.10 | 0.80 | 0.59 |
| ratio to strength of wetroom beams | 8.62 | 8.01 | 7.92 | 1.00 | 0.82 | 6.42 | 4.71 |
| ratio to strength of outdoor beams | 10.56 | 9.82 | 9.70 | 1.23 | 1.00 | 7.86 | 5.77 |

TABLE B-9. CASTINGS 4, 5, AND 6, RATIOS OF MATURITY STRENGTHS TO FOUR STANDARDS, AT 7 DAYS

| | MATURITY STRENGTHS LISTED BY BASIS OF MATURITY CURVE | | | | |
|---|---|---|---|---|---|
| | CYLINDERS | | CORES | BEAMS | |
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors |
| **CASTING 4** | | | | | |
| ratio to strength of wetroom cylinders | 0.92 | 0.83 | 0.85 | 0.15 | 0.12 |
| ratio to strength of outdoor cylinders | 0.95 | 0.85 | 0.88 | 0.15 | 0.13 |
| ratio to strength of wetroom beams | 6.07 | 5.44 | 5.59 | 0.97 | 0.82 |
| ratio to strength of outdoor beams | 6.29 | 5.63 | 5.79 | 1.00 | 0.85 |
| **CASTING 5** | | | | | |
| ratio to strength of wetroom cylinders | 0.89 | 0.78 | 0.95 | 0.12 | 0.22 |
| ratio to strength of outdoor cylinders | 1.07 | 0.93 | 1.14 | 0.14 | 0.13 |
| ratio to strength of wetroom beams | 7.55 | 6.63 | 8.05 | 1.02 | 0.93 |
| ratio to strength of outdoor beams | 9.10 | 7.98 | 9.69 | 1.23 | 1.12 |
| **CASTING 6** | | | | | |
| ratio to strength of wetroom cylinders | 0.92 | 0.84 | 0.85 | 0.11 | 0.09 |
| ratio to strength of outdoor cylinders | 0.99 | 0.90 | 0.92 | 0.12 | 0.09 |
| ratio to strength of wetroom beams | 7.95 | 7.24 | 7.36 | 0.95 | 0.76 |
| ratio to strength of outdoor beams | 9.73 | 8.87 | 9.02 | 1.16 | 0.93 |

TABLE B-10. CASTINGS 4, 5, AND 6, RATIOS OF VARIOUS STRENGTHS TO FOUR STANDARDS, AT 14 DAYS

| | CYLINDERS | | CORES | BEAMS | | PULLOUT | PEN. RESIST. |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors | LOK-Test | Windsor Probe |
| **CASTING 4** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.91 | 0.88 | 0.14 | 0.11 | --- | --- |
| ratio to strength of outdoor cylinders | 1.10 | 1.00 | 0.97 | 0.15 | 0.12 | --- | --- |
| ratio to strength of wetroom beams | 7.26 | 6.57 | 6.41 | 1.00 | 0.82 | --- | --- |
| ratio to strength of outdoor beams | 8.84 | 8.01 | 7.80 | 1.22 | 1.00 | --- | --- |
| **CASTING 5** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.85 | 1.10 | 0.13 | 0.13 | --- | --- |
| ratio to strength of outdoor cylinders | 1.17 | 1.00 | 1.29 | 0.15 | 0.15 | --- | --- |
| ratio to strength of wetroom beams | 7.71 | 6.58 | 8.48 | 1.00 | 0.97 | --- | --- |
| ratio to strength of outdoor beams | 7.95 | 6.79 | 8.76 | 1.03 | 1.00 | --- | --- |
| **CASTING 6** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 1.00 | 0.91 | 0.10 | 0.10 | --- | --- |
| ratio to strength of outdoor cylinders | 1.00 | 1.00 | 0.91 | 0.10 | 0.10 | --- | --- |
| ratio to strength of wetroom beams | 9.91 | 9.89 | 9.00 | 1.00 | 0.97 | --- | --- |
| ratio to strength of outdoor beams | 10.18 | 10.16 | 9.24 | 1.03 | 1.00 | --- | --- |

TABLE B-11. CASTINGS 4, 5, AND 6, RATIOS OF MATURITY STRENGTHS TO FOUR STANDARDS, AT 14 DAYS

| | MATURITY STRENGTHS LISTED BY BASIS OF MATURITY CURVE | | | | |
|---|---|---|---|---|---|
| | CYLINDERS | | CORES | BEAMS | |
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors |
| **CASTING 4** | | | | | |
| ratio to strength of wetroom cylinders | 0.99 | 0.98 | 0.89 | 0.14 | 0.13 |
| ratio to strength of outdoor cylinders | 1.09 | 1.08 | 0.98 | 0.15 | 0.14 |
| ratio to strength of wetroom beams | 7.17 | 7.08 | 6.43 | 1.01 | 0.93 |
| ratio to strength of outdoor beams | 8.73 | 8.62 | 7.83 | 1.22 | 1.13 |
| **CASTING 5** | | | | | |
| ratio to strength of wetroom cylinders | 1.04 | 0.89 | 1.08 | 0.13 | 0.22 |
| ratio to strength of outdoor cylinders | 1.22 | 1.05 | 1.26 | 0.15 | 0.14 |
| ratio to strength of wetroom beams | 8.05 | 6.89 | 8.32 | 0.98 | 0.93 |
| ratio to strength of outdoor beams | 8.31 | 7.11 | 8.59 | 1.01 | 0.96 |
| **CASTING 6** | | | | | |
| ratio to strength of wetroom cylinders | 1.01 | 0.93 | 0.93 | 0.11 | 0.09 |
| ratio to strength of outdoor cylinders | 1.01 | 0.94 | 0.93 | 0.11 | 0.09 |
| ratio to strength of wetroom beams | 9.96 | 9.26 | 9.21 | 1.07 | 0.89 |
| ratio to strength of outdoor beams | 10.23 | 9.51 | 9.46 | 1.10 | 0.91 |

TABLE B-12. CASTINGS 4, 5, AND 6, RATIOS OF VARIOUS STRENGTHS TO FOUR STANDARDS, AT 28 DAYS

| | CYLINDERS | | CORES | BEAMS | | PULLOUT | PEN. RESIST. |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors | LOK-Test | Windsor Probe |
| **CASTING 4** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.97 | 0.91 | 0.14 | 0.13 | 0.67 | 0.69 |
| ratio to strength of outdoor cylinders | 1.03 | 1.00 | 0.94 | 0.14 | 0.14 | 0.69 | 0.71 |
| ratio to strength of wetroom beams | 7.40 | 7.19 | 6.75 | 1.00 | 0.97 | 4.96 | 5.10 |
| ratio to strength of outdoor beams | 7.61 | 7.39 | 6.94 | 1.03 | 1.00 | 5.10 | 5.25 |
| **CASTING 5** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.87 | 1.00 | 0.12 | 0.12 | 0.93 | 0.73 |
| ratio to strength of outdoor cylinders | 1.15 | 1.00 | 1.15 | 0.13 | 0.13 | 1.06 | 0.84 |
| ratio to strength of wetroom beams | 8.60 | 7.49 | 8.62 | 1.00 | 1.00 | 7.98 | 6.26 |
| ratio to strength of outdoor beams | 8.60 | 7.49 | 8.62 | 1.00 | 1.00 | 7.98 | 6.26 |
| **CASTING 6** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.97 | 0.97 | 0.11 | 0.08 | 0.85 | 0.50 |
| ratio to strength of outdoor cylinders | 1.03 | 1.00 | 1.00 | 0.11 | 0.08 | 0.87 | 0.51 |
| ratio to strength of wetroom beams | 9.24 | 8.98 | 8.99 | 1.00 | 0.74 | 7.84 | 4.58 |
| ratio to strength of outdoor beams | 12.40 | 12.07 | 12.08 | 1.34 | 1.00 | 10.54 | 6.16 |

TABLE B-13. CASTINGS 4, 5, AND 6, RATIOS OF MATURITY STRENGTHS TO FOUR STANDARDS, AT 28 DAYS

| | MATURITY STRENGTHS LISTED BY BASIS OF MATURITY CURVE | | | | |
|---|---|---|---|---|---|
| | CYLINDERS | | CORES | BEAMS | |
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors |
| **CASTING 4** | | | | | |
| ratio to strength of wetroom cylinders | 1.05 | 0.97 | 0.95 | 0.14 | 0.13 |
| ratio to strength of outdoor cylinders | 1.08 | 1.00 | 0.98 | 0.14 | 0.14 |
| ratio to strength of wetroom beams | 7.74 | 7.19 | 7.02 | 1.01 | 0.99 |
| ratio to strength of outdoor beams | 7.97 | 7.39 | 7.23 | 1.04 | 1.02 |
| **CASTING 5** | | | | | |
| ratio to strength of wetroom cylinders | 1.05 | 0.90 | 1.06 | 0.12 | 0.22 |
| ratio to strength of outdoor cylinders | 1.20 | 1.03 | 1.22 | 0.13 | 0.13 |
| ratio to strength of wetroom beams | 9.01 | 7.70 | 9.11 | 1.01 | 1.00 |
| ratio to strength of outdoor beams | 9.01 | 7.70 | 9.11 | 1.01 | 1.00 |
| **CASTING 6** | | | | | |
| ratio to strength of wetroom cylinders | 1.06 | 1.06 | 1.01 | 0.11 | 0.09 |
| ratio to strength of outdoor cylinders | 1.09 | 1.09 | 1.04 | 0.11 | 0.10 |
| ratio to strength of wetroom beams | 9.80 | 9.79 | 9.32 | 0.97 | 0.86 |
| ratio to strength of outdoor beams | 13.17 | 13.14 | 12.52 | 1.31 | 1.16 |

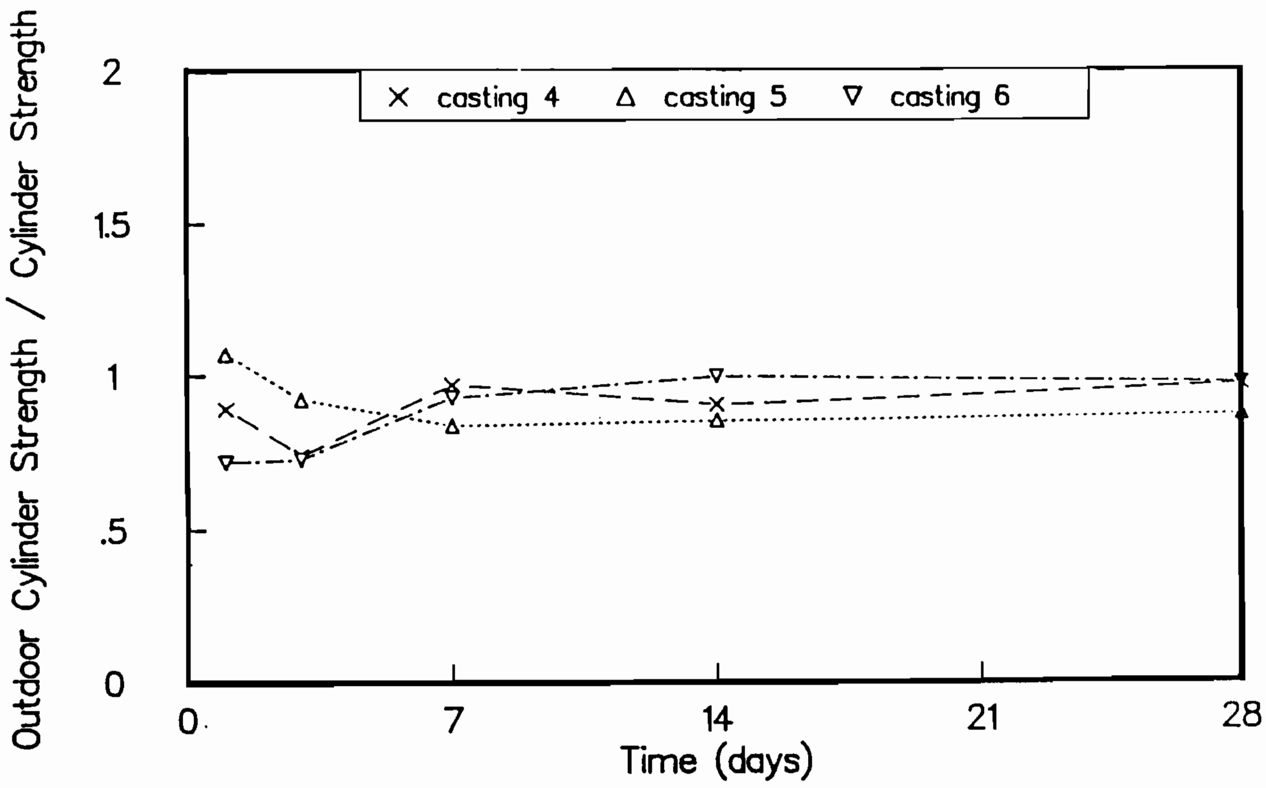


**Fig B-1. Ratio of strengths of outdoor-cured cylinders to strengths of wetroom-cured cylinders for castings 4, 5, and 6.**

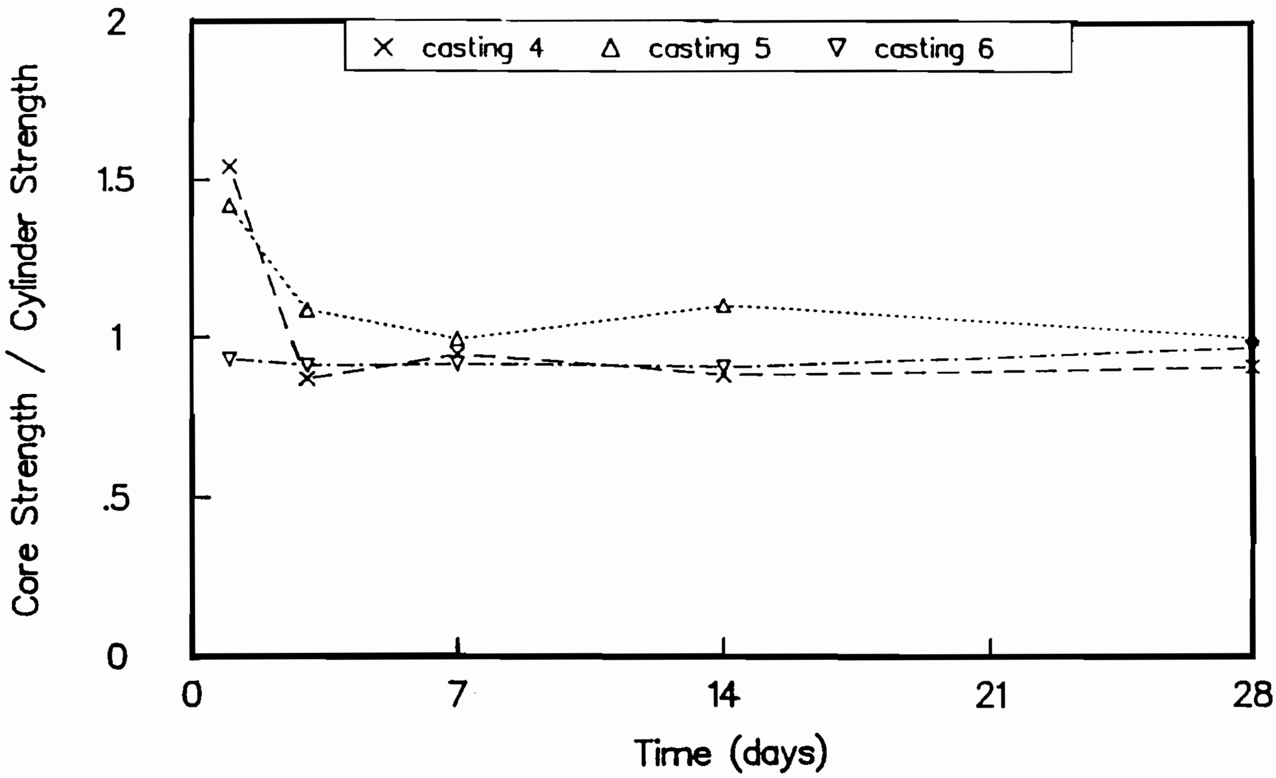


**Fig B-2. Ratio of strengths of cores to strengths of wetroom-cured cylinders for castings 4, 5, and 6.**

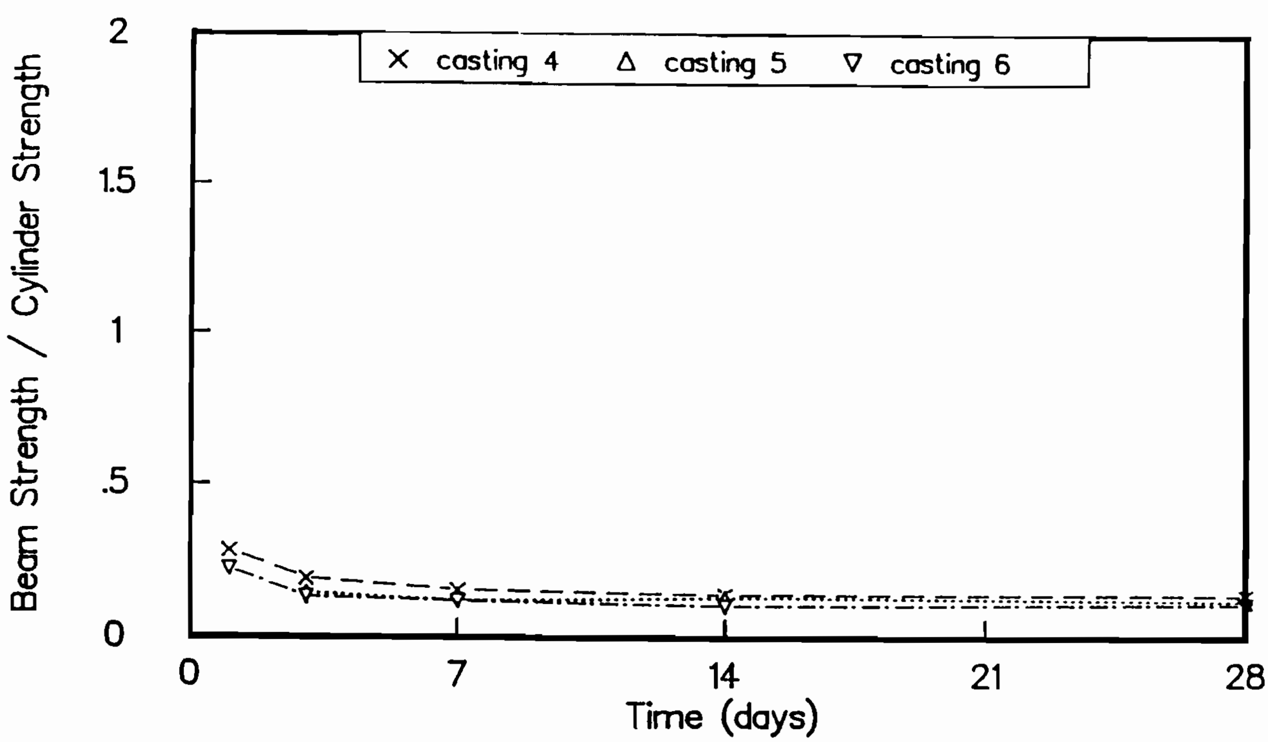


**Fig B-3. Ratio of flexural strengths of wetroom-cured beams to strengths of wetroom-cured cylinders for castings 4, 5, and 6.**

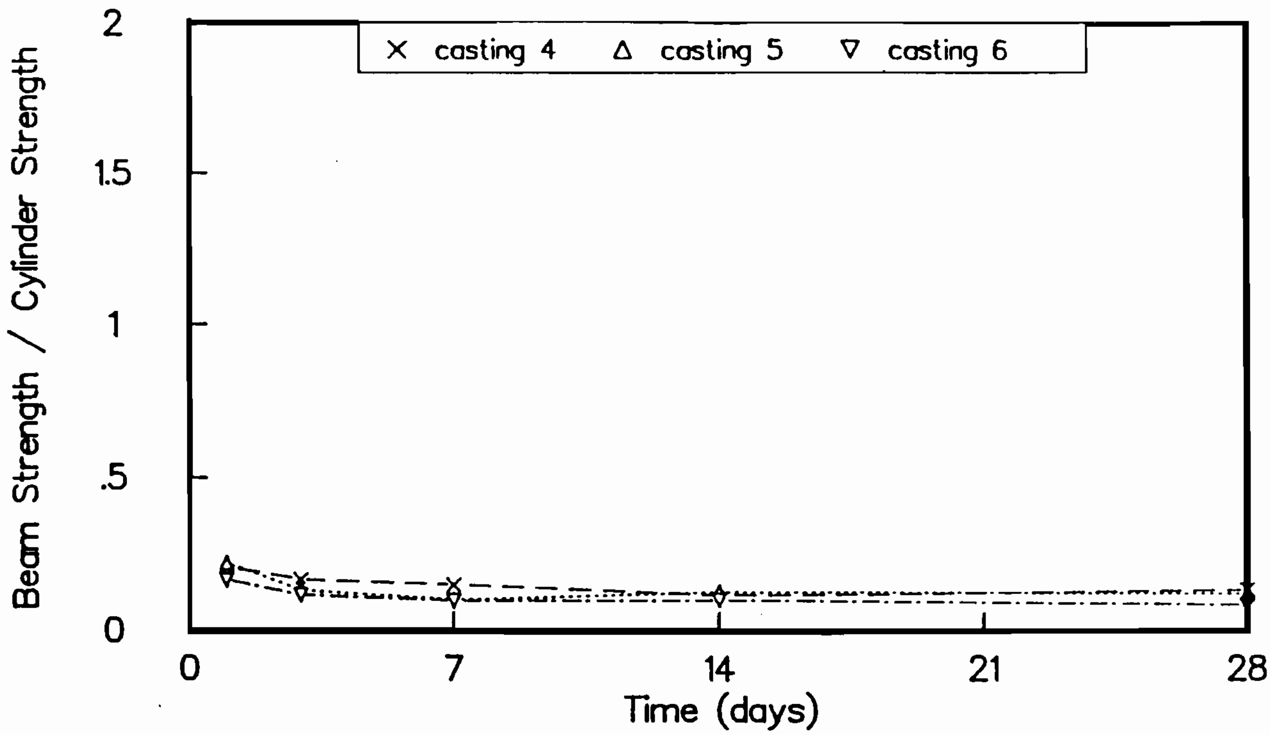


**Fig B-4. Ratio of flexural strengths of outdoor-cured beams to strengths of wetroom-cured cylinders for castings 4, 5, and 6.**

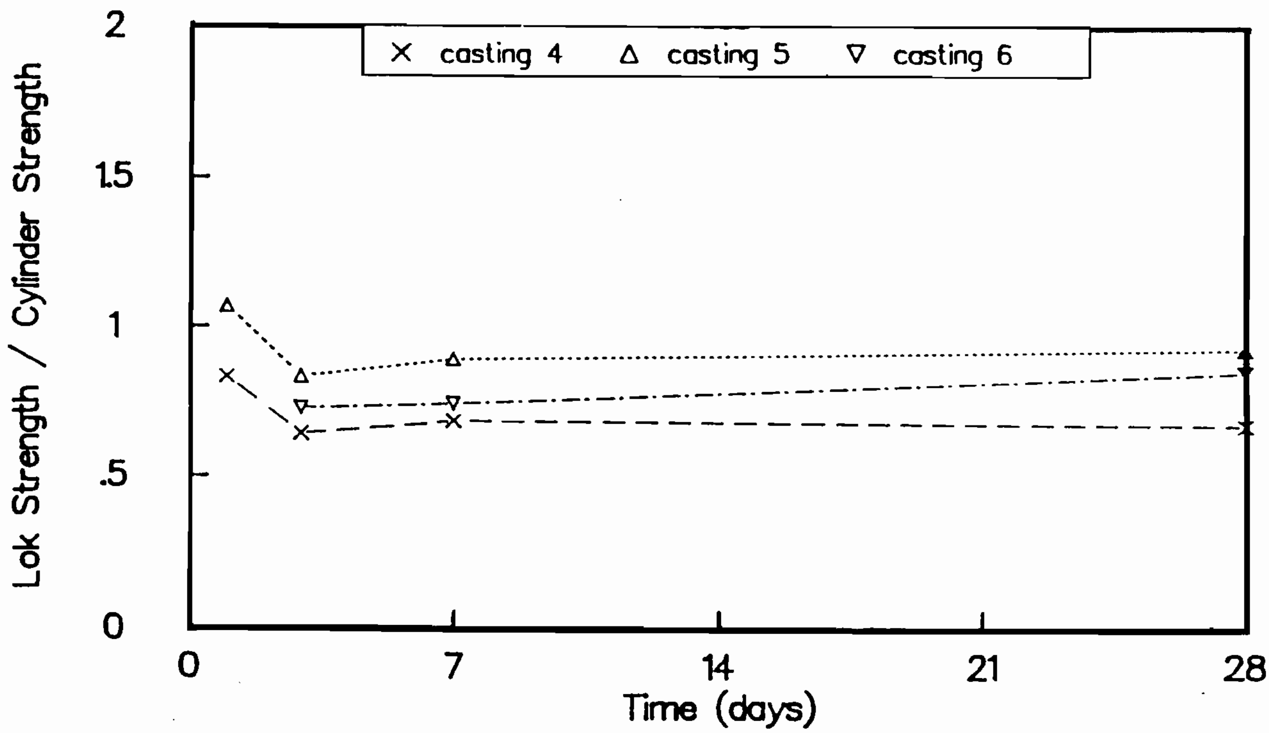


**Fig B-5. Ratio of "Lok-Test" pullout strengths to strengths of wetroom-cured cylinders for castings 4, 5, and 6.**

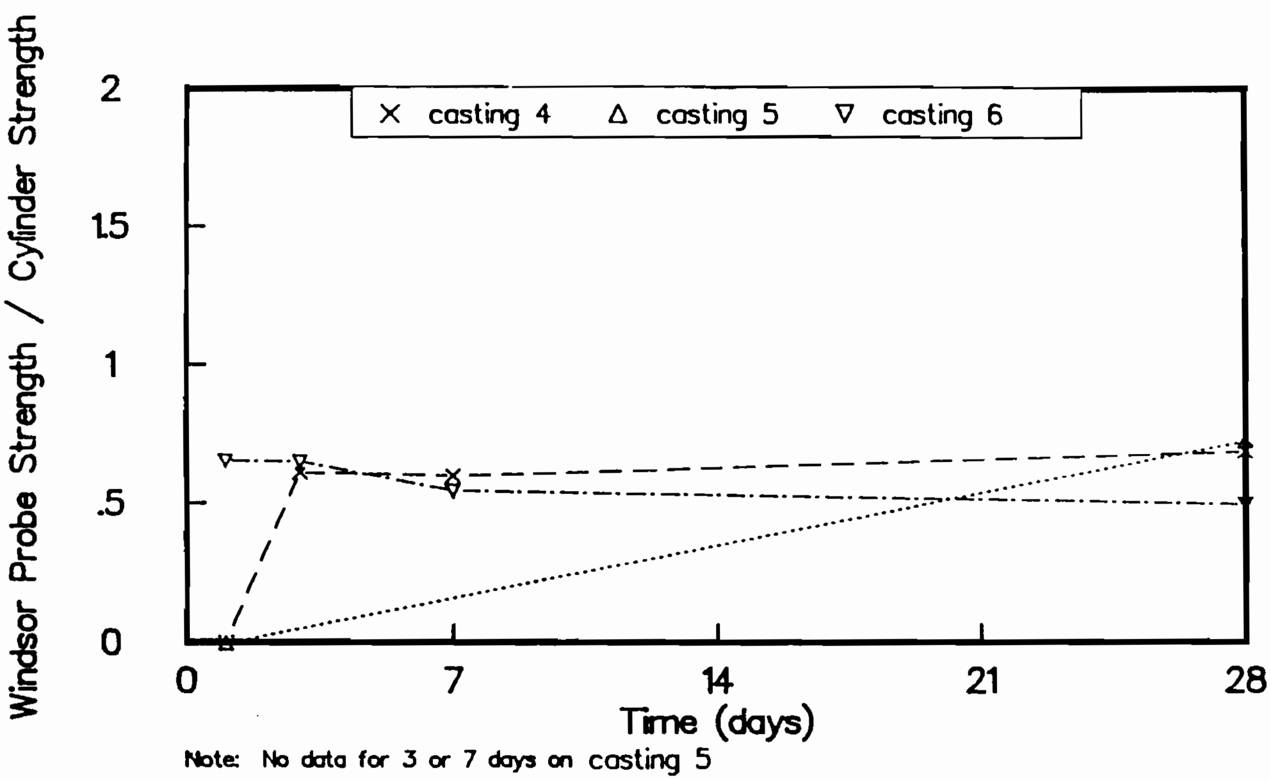


**Fig B-6. Ratio of Windsor probe penetration resistance strengths to strengths of wetroom-cured cylinders for castings 4, 5, and 6.**

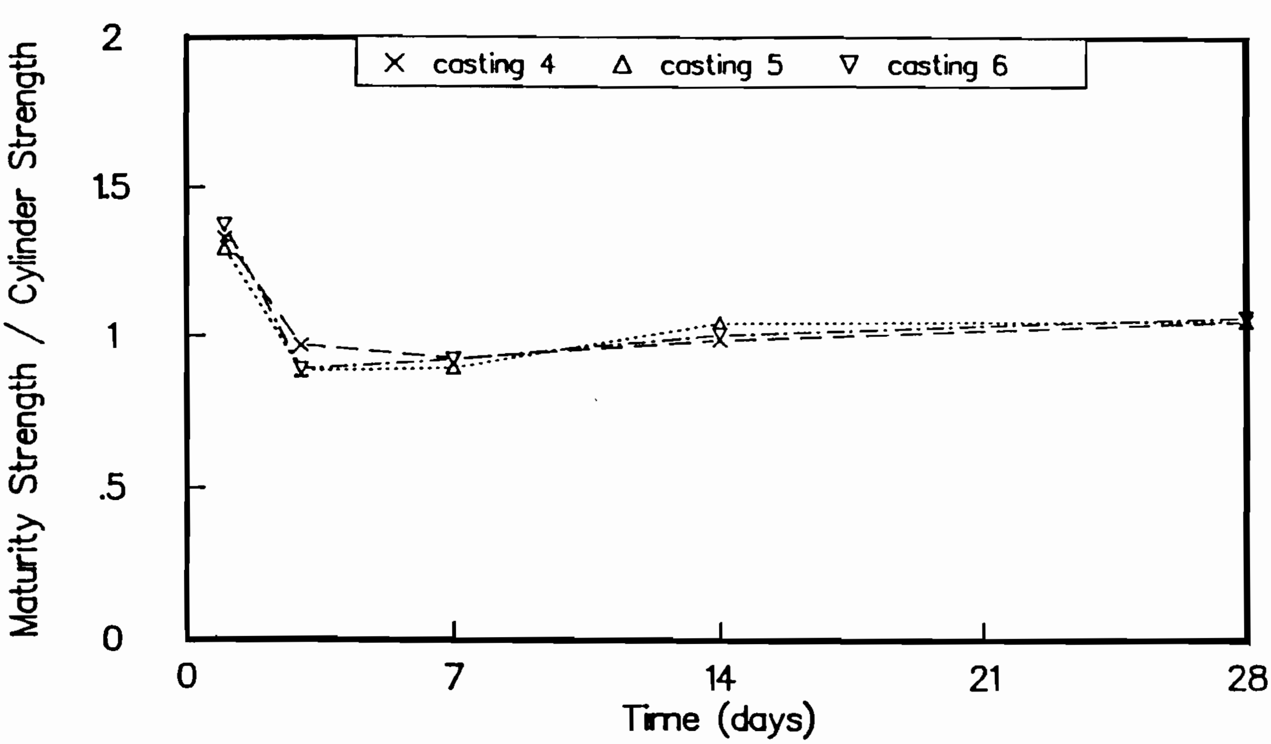


**Fig B-7. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 4, 5, and 6. Maturity curve based on wetroom-cured cylinders.**

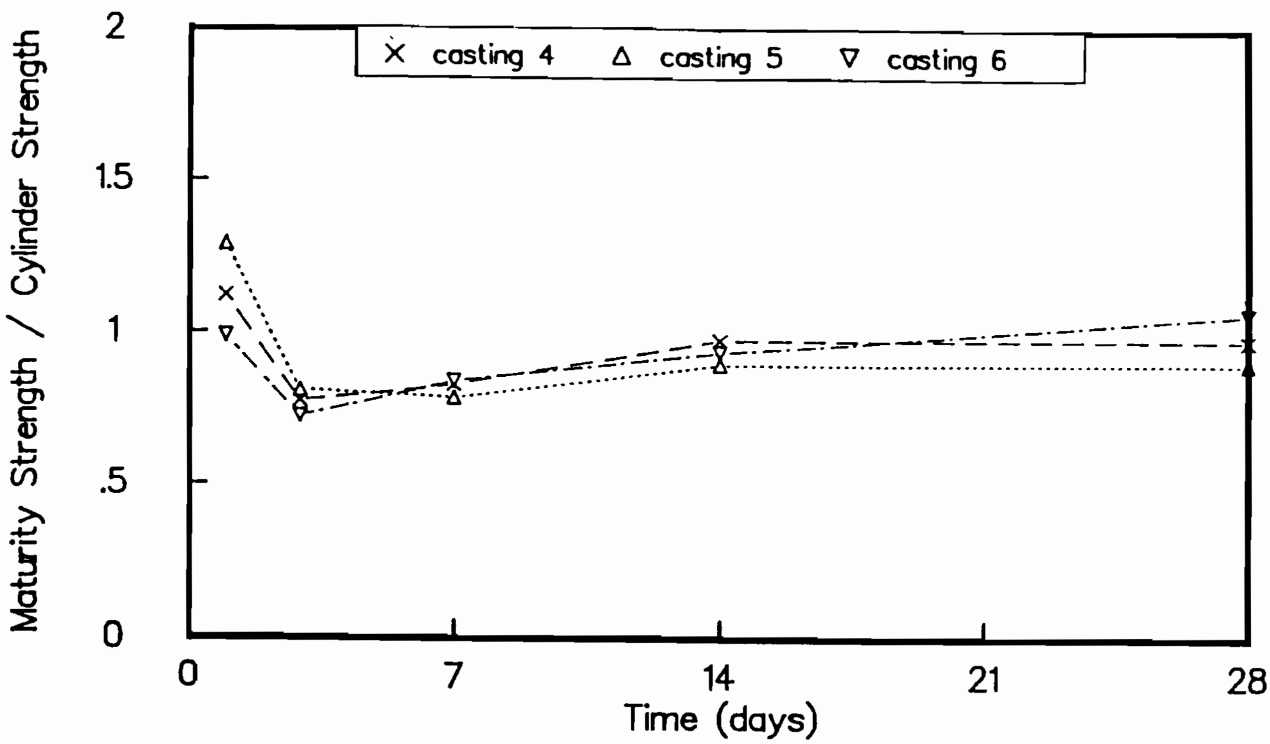


**Fig B-8. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 4, 5, and 6. Maturity curve based on outdoor-cured cylinders.**

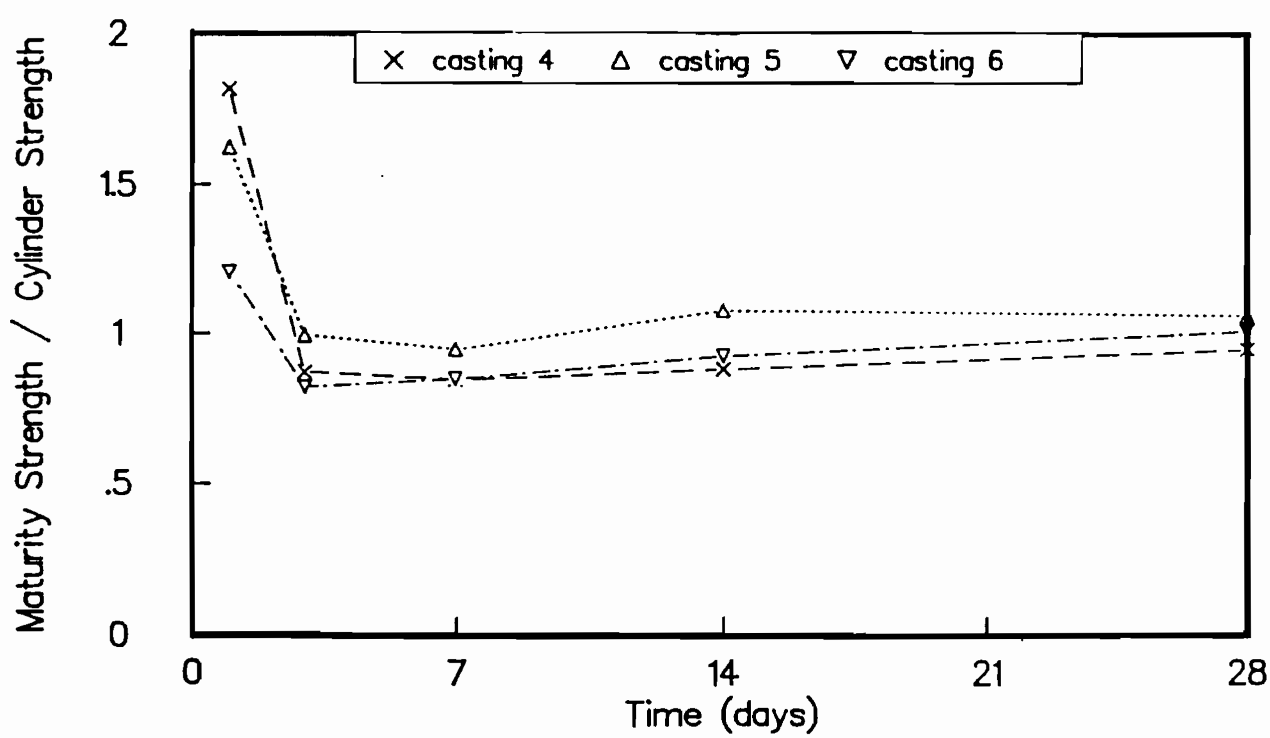


**Fig B-9. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 4, 5, and 6. Maturity curve based on cores drilled from slabs.**

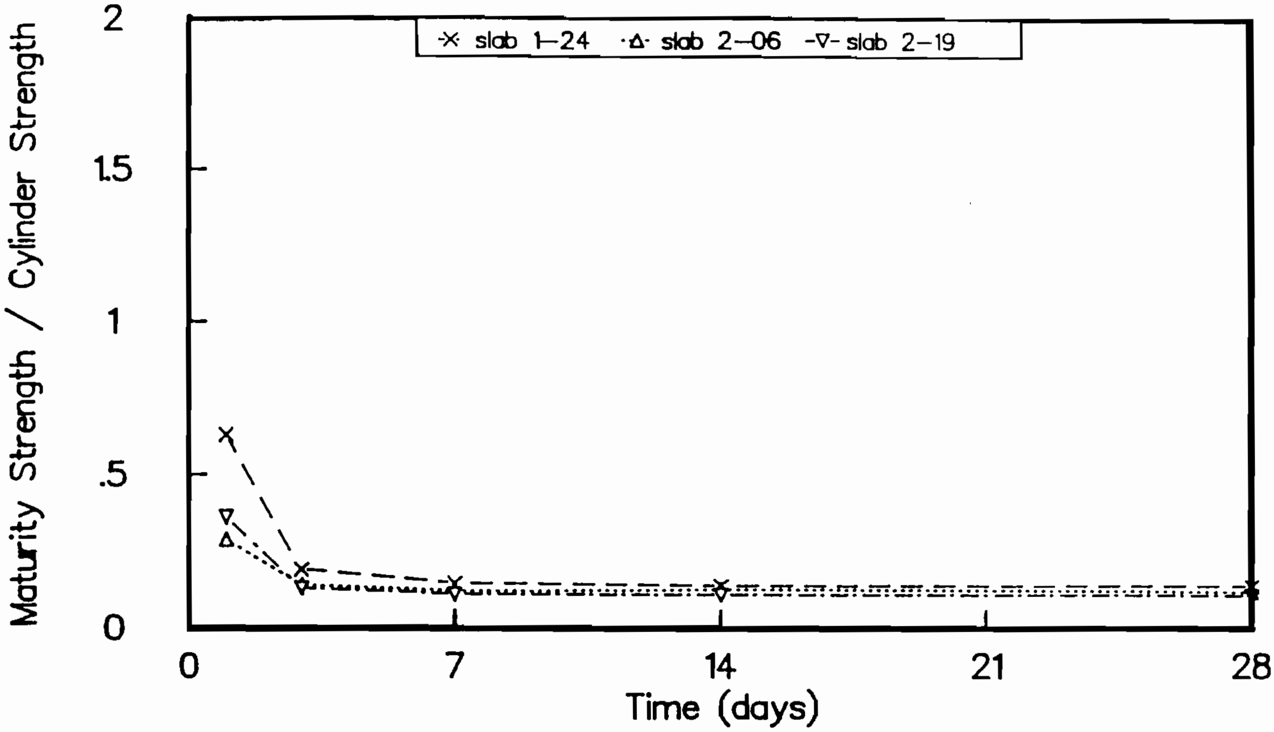


**Fig B-10. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 4, 5, and 6. Maturity curve based on wetroom-cured beams.**

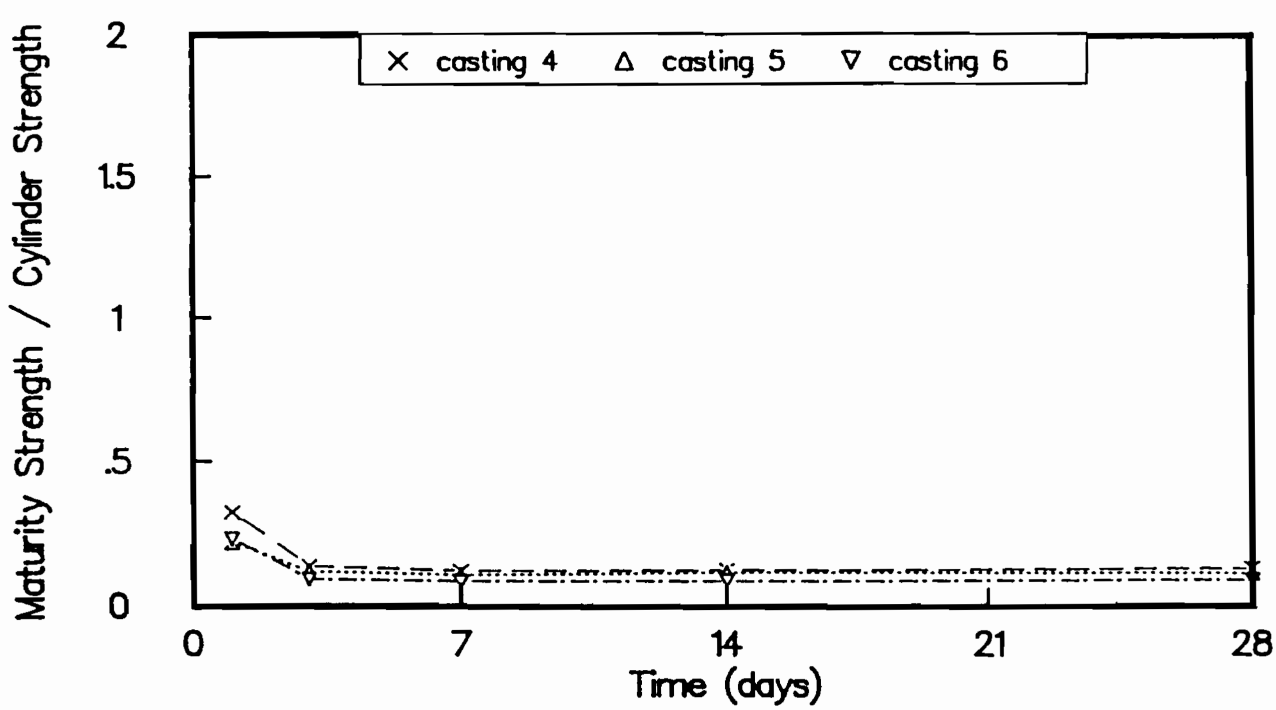


**Fig B-11. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 4, 5, and 6. Maturity curve based on outdoor-cured beams.**

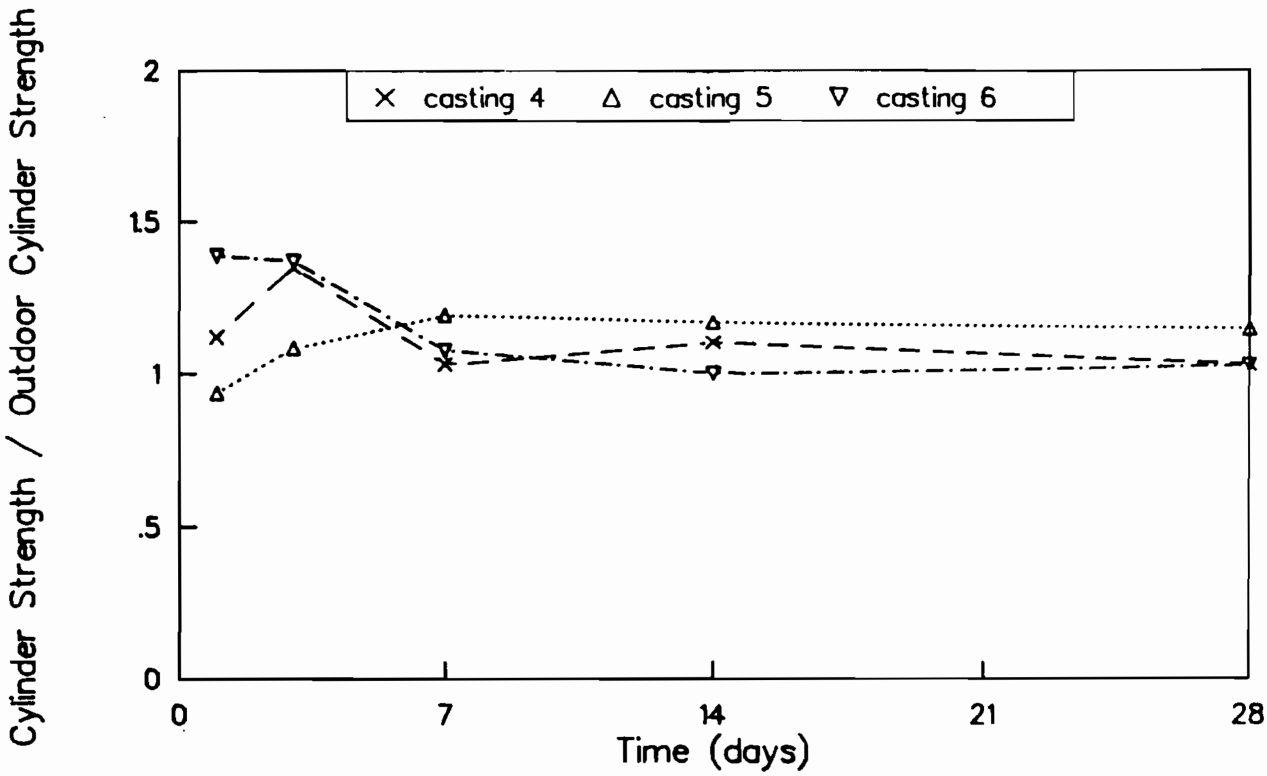


**Fig B-12. Ratio of strengths of wetroom-cured cylinders to strengths of outdoor-cured cylinders for castings 4, 5, and 6.**

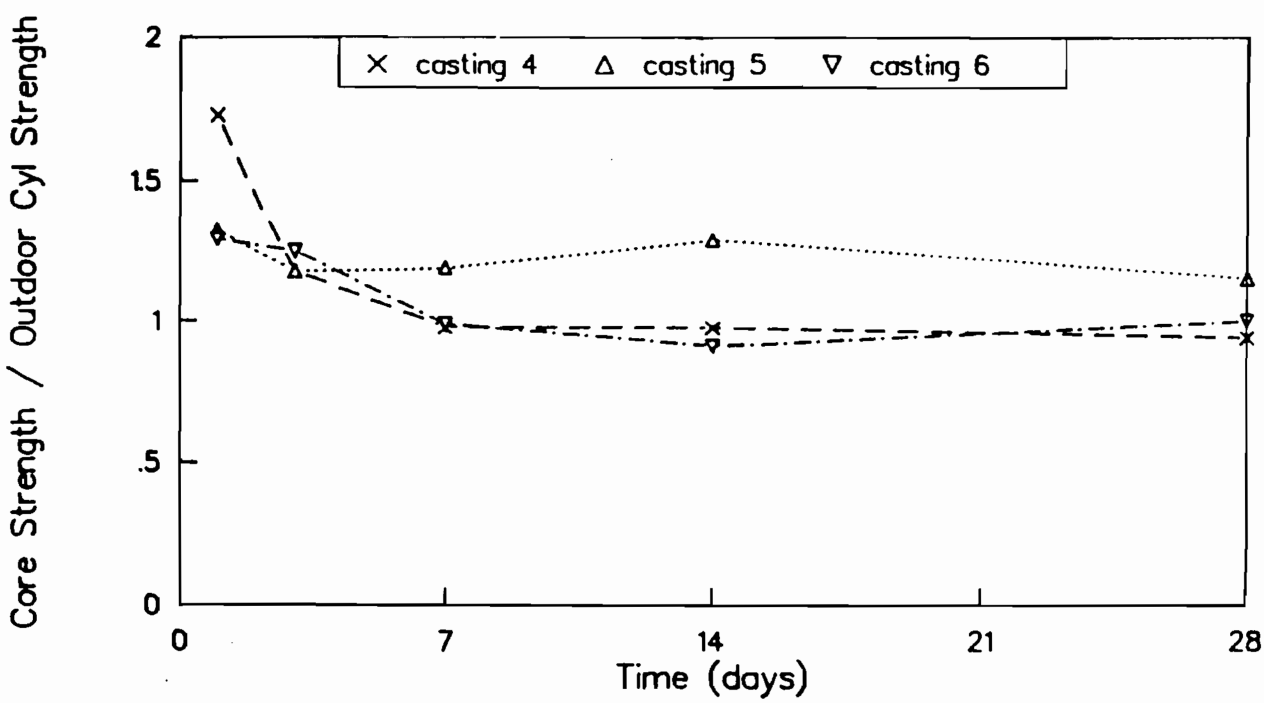


**Fig B-13. Ratio of strengths of cores to strengths of outdoor-cured cylinders for castings 4, 5, and 6.**

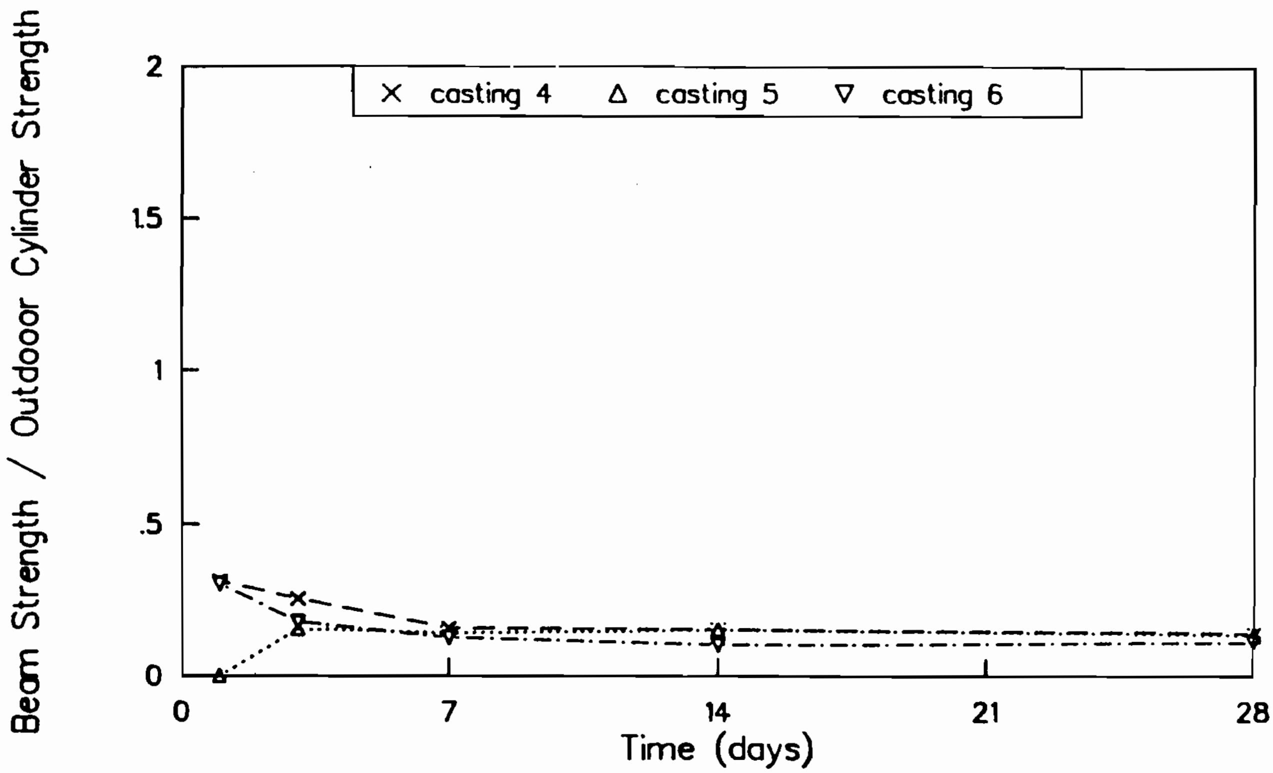


**Fig B-14. Ratio of flexural strengths of wetroom-cured beams to strengths of outdoor-cured cylinders for castings 4, 5, and 6.**

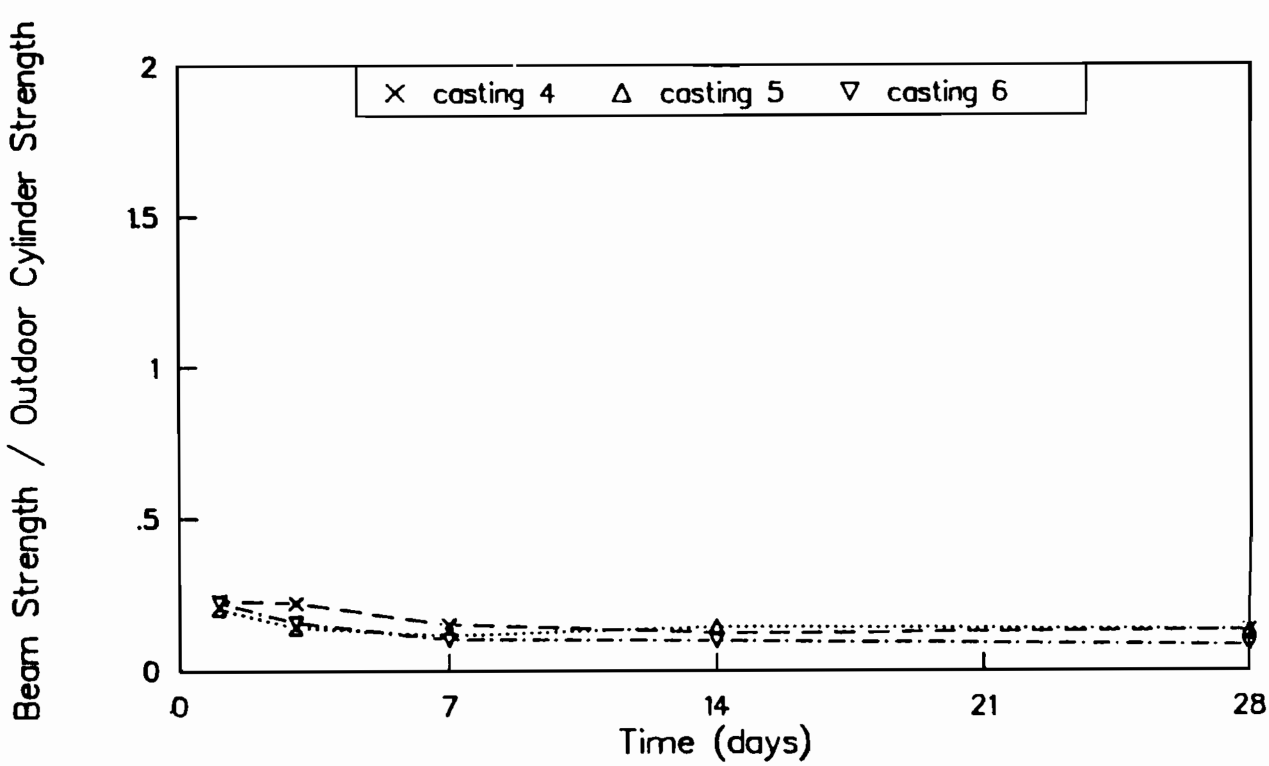


**Fig B-15. Ratio of flexural strengths of outdoor-cured beams to strengths of outdoor-cured cylinders for castings 4, 5, and 6.**

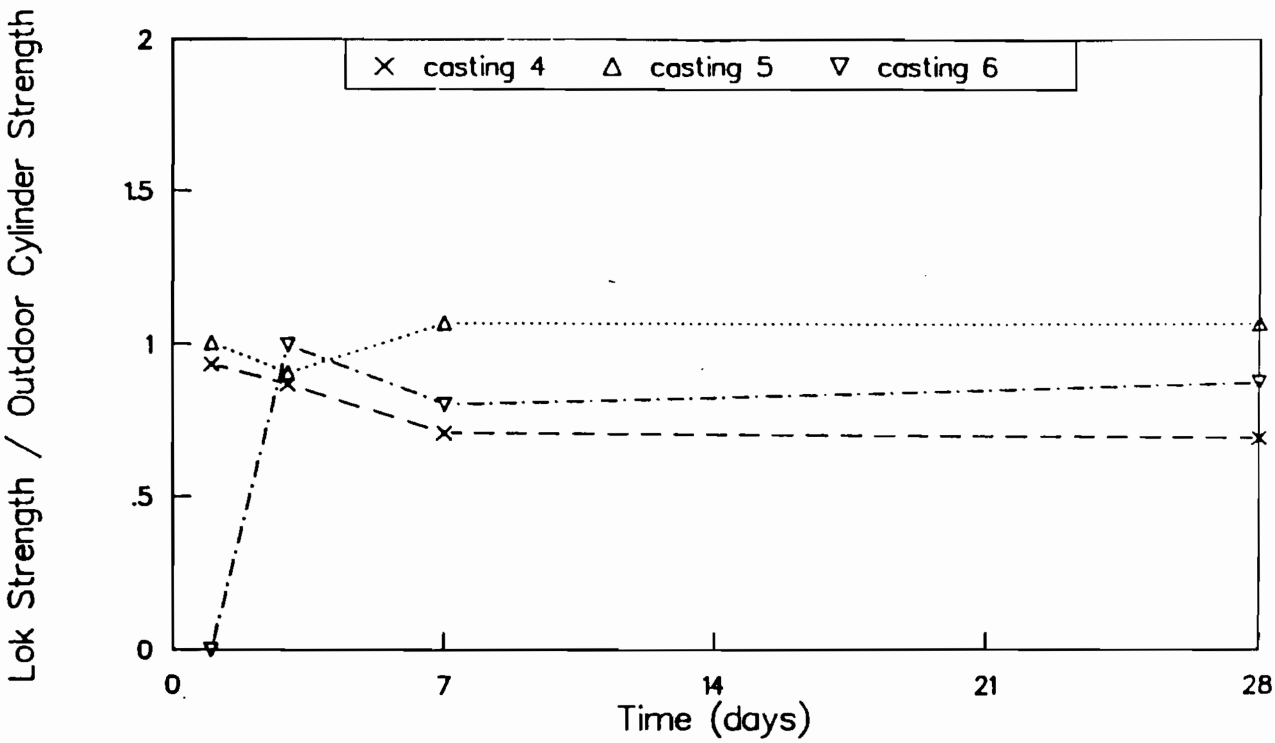


Fig B-16. Ratio of "Lok-Test" pullout strengths to strengths of outdoor-cured cylinders for castings 4, 5, and 6.

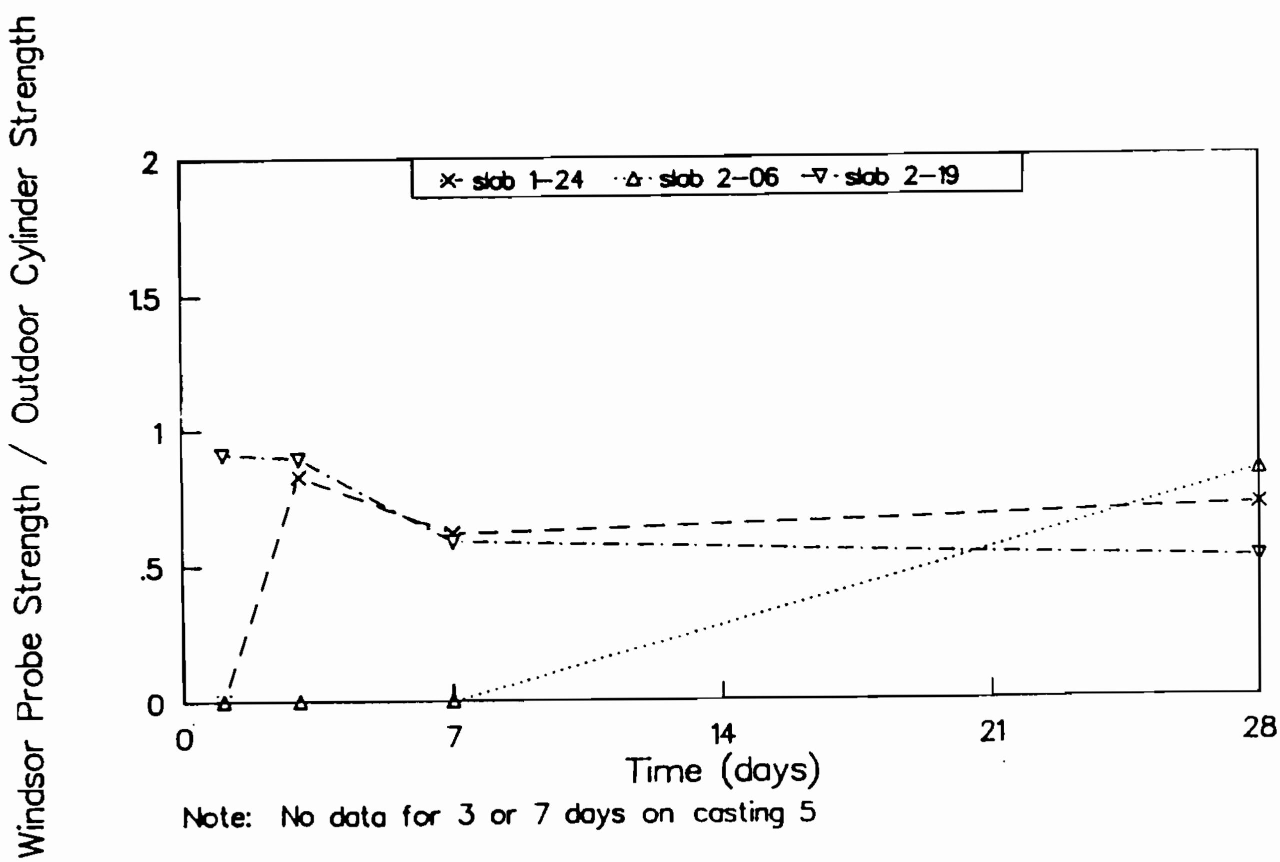


Fig B-17. Ratio of Windsor probe penetration resistance strengths to strengths of outdoor-cured cylinders for castings 4, 5, and 6.

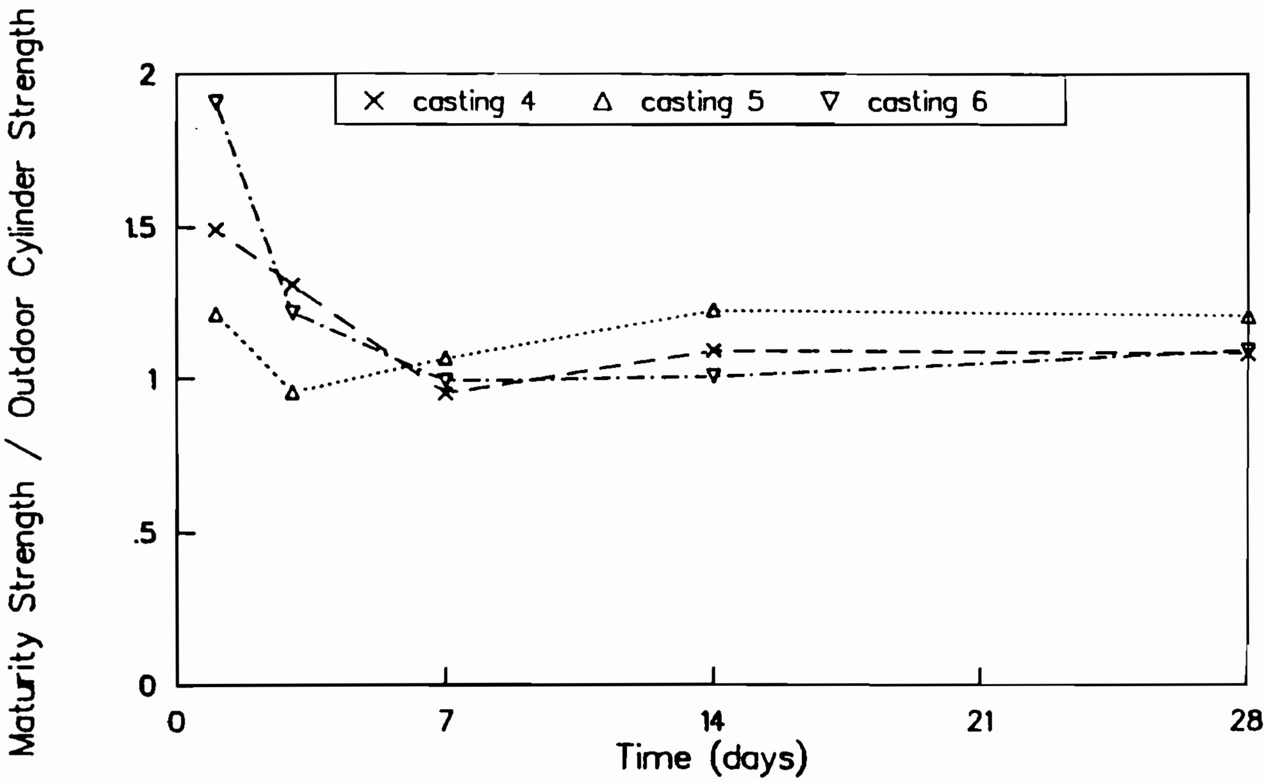


**Fig B-18. Ratio of maturity method strengths to strengths of outdoor-cured cylinders for castings 4, 5, and 6. Maturity curve based on wetroom-cured cylinders.**

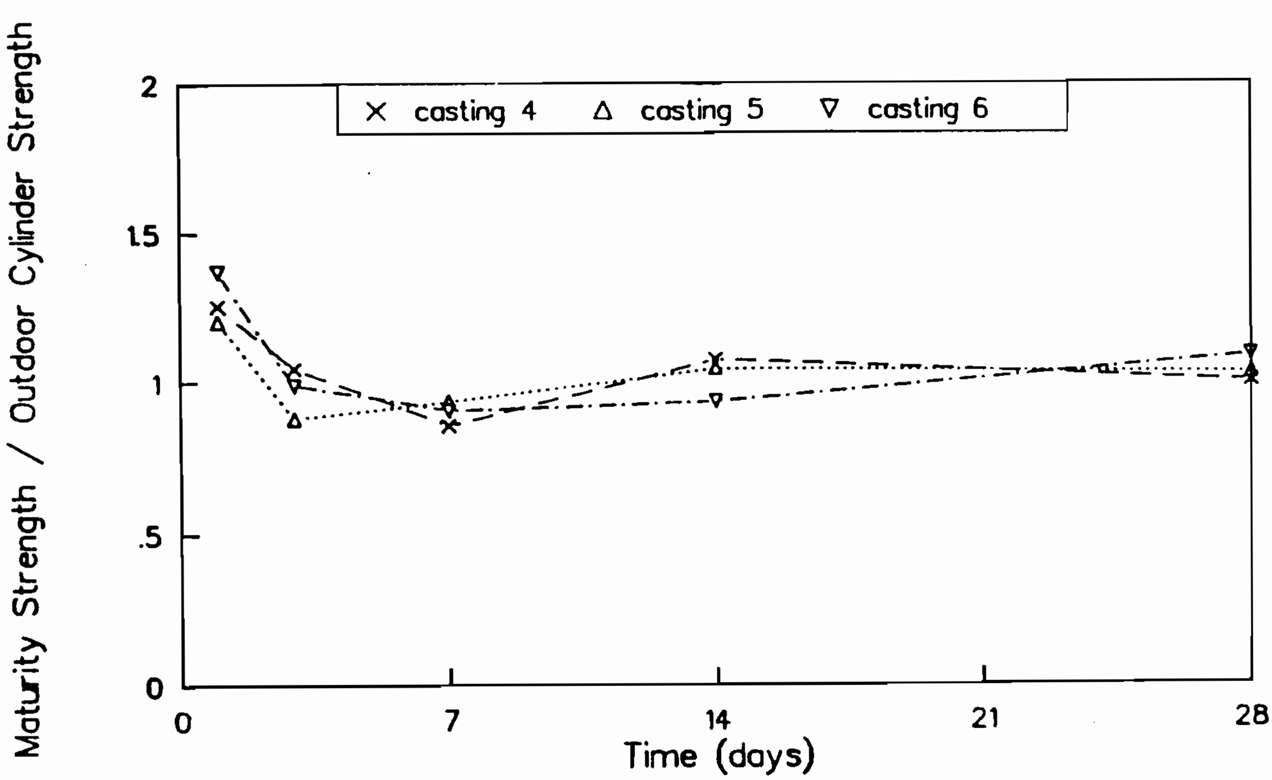


**Fig B-19. Ratio of maturity method strengths to strengths of outdoor-cured cylinders for castings 4, 5, and 6. Maturity curve based on outdoor-cured cylinders.**

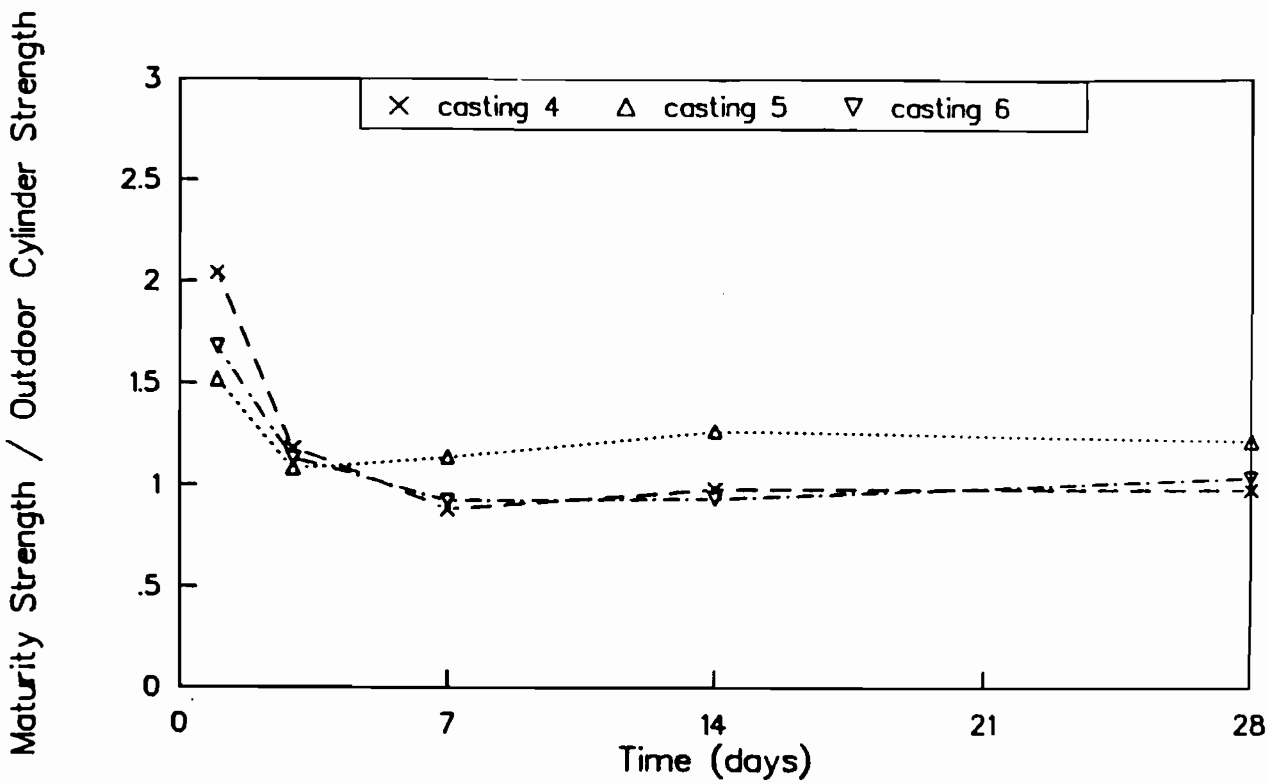


**Fig B-20. Ratio of maturity method strengths to strengths of outdoor-cured cylinders for castings 4, 5, and 6. Maturity curve based on cores drilled from slabs.**

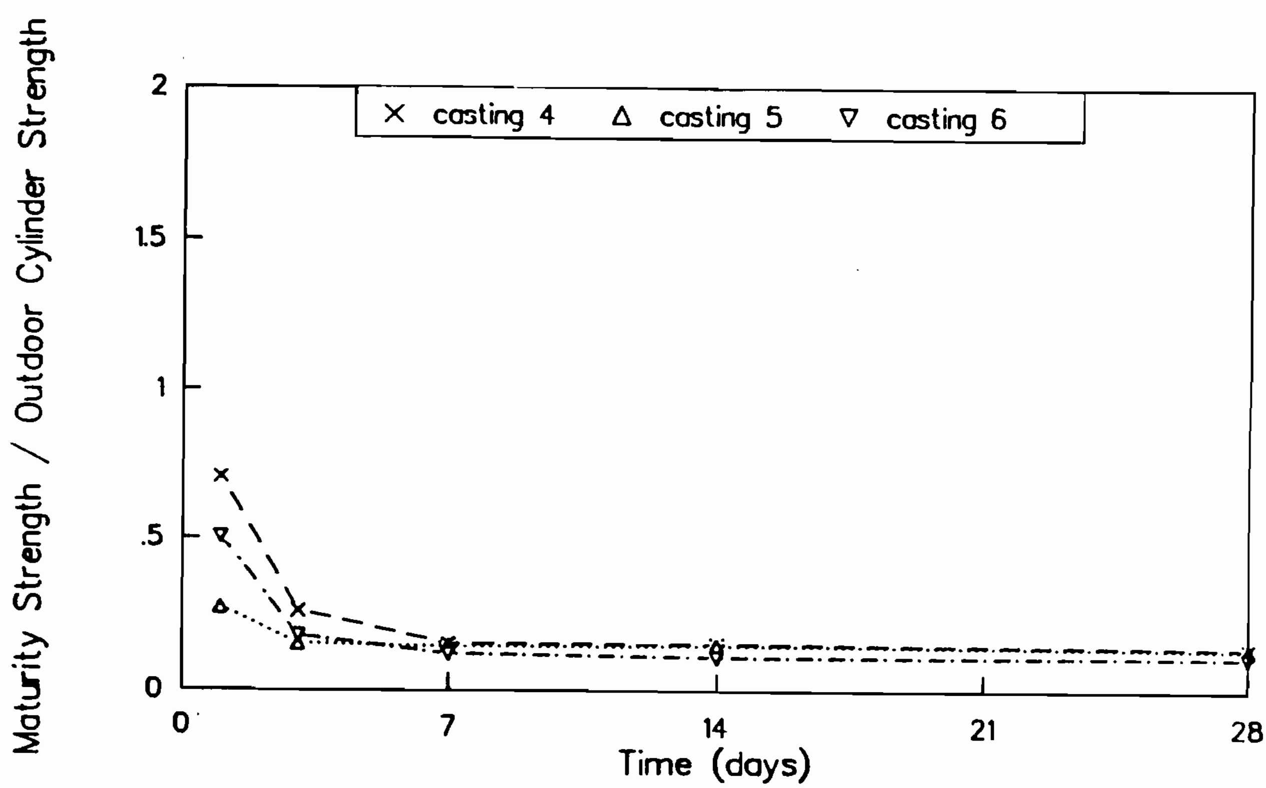


**Fig B-21. Ratio of maturity method strengths to strengths of outdoor-cured cylinders for castings 4, 5, and 6. Maturity curve based on wetroom-cured beams.**

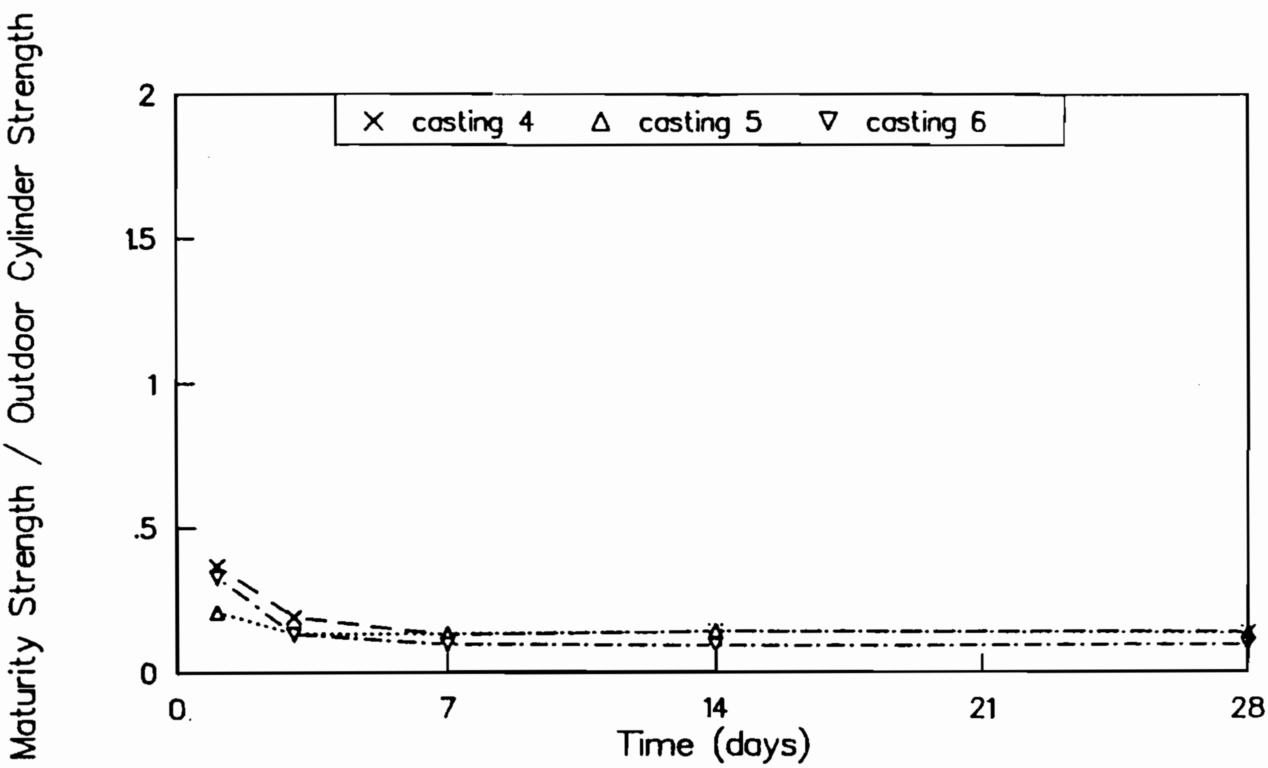


**Fig B-22. Ratio of maturity method strengths to strengths of outdoor-cured cylinders for castings 4, 5, and 6. Maturity curve based on outdoor-cured beams.**

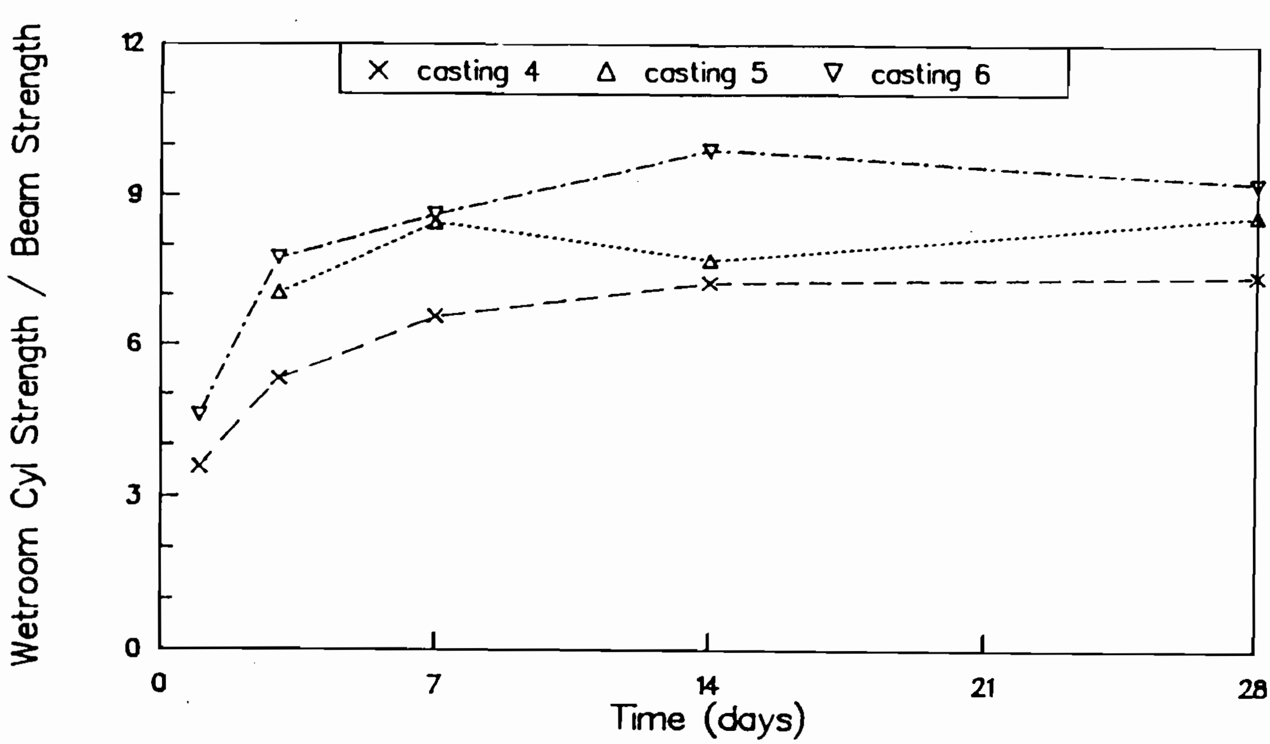


**Fig B-23. Ratio of strengths of wetroom-cured cylinders to flexural strengths of wetroom-cured beams for castings 4, 5, and 6.**

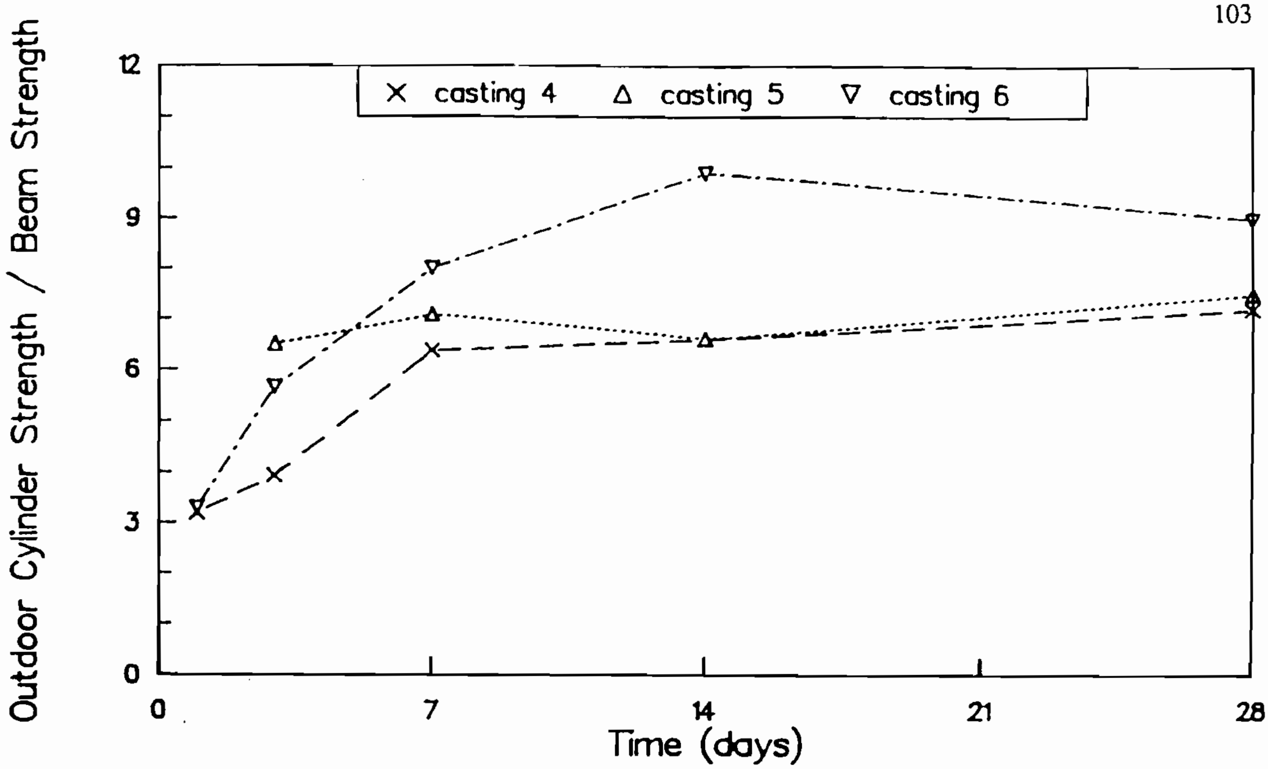


Fig B-24. Ratio of strengths of outdoor-cured cylinders to flexural strengths of wetroom-cured beams for castings 4, 5, and 6.

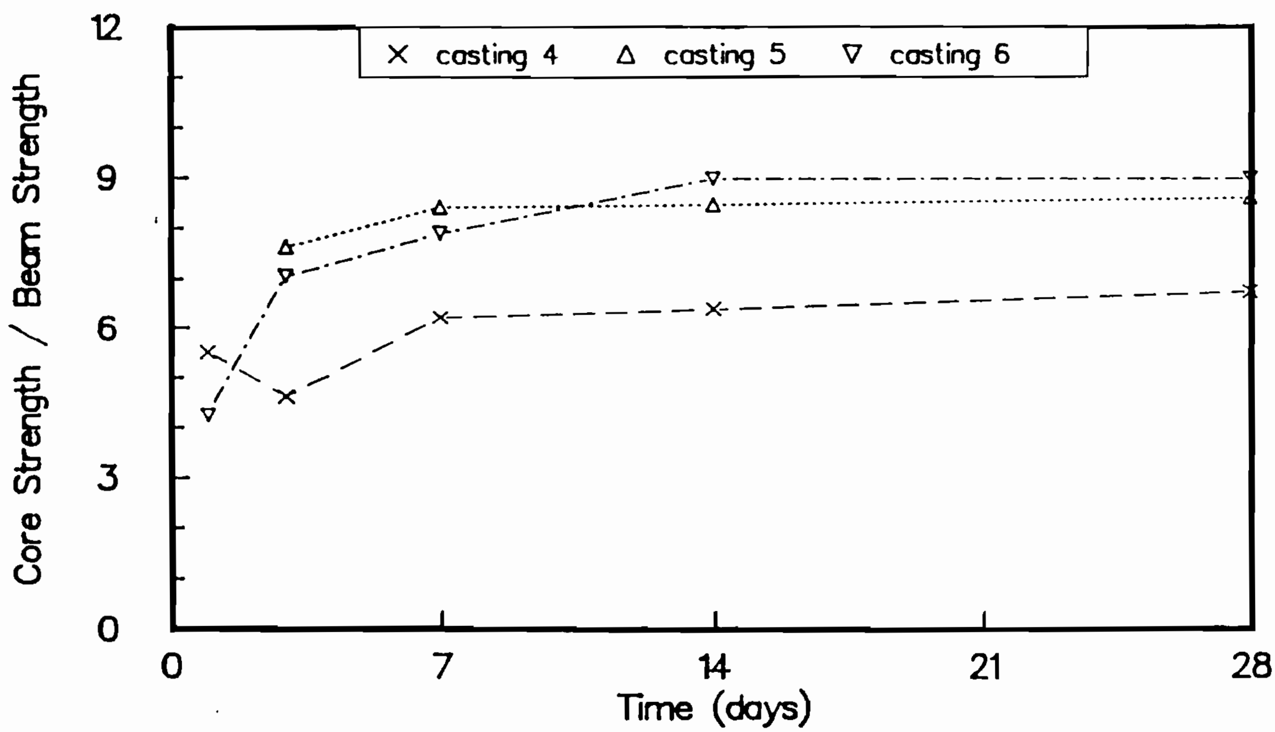


Fig B-25. Ratio of strengths of cores to flexural strengths of wetroom-cured beams for castings 4, 5, and 6.

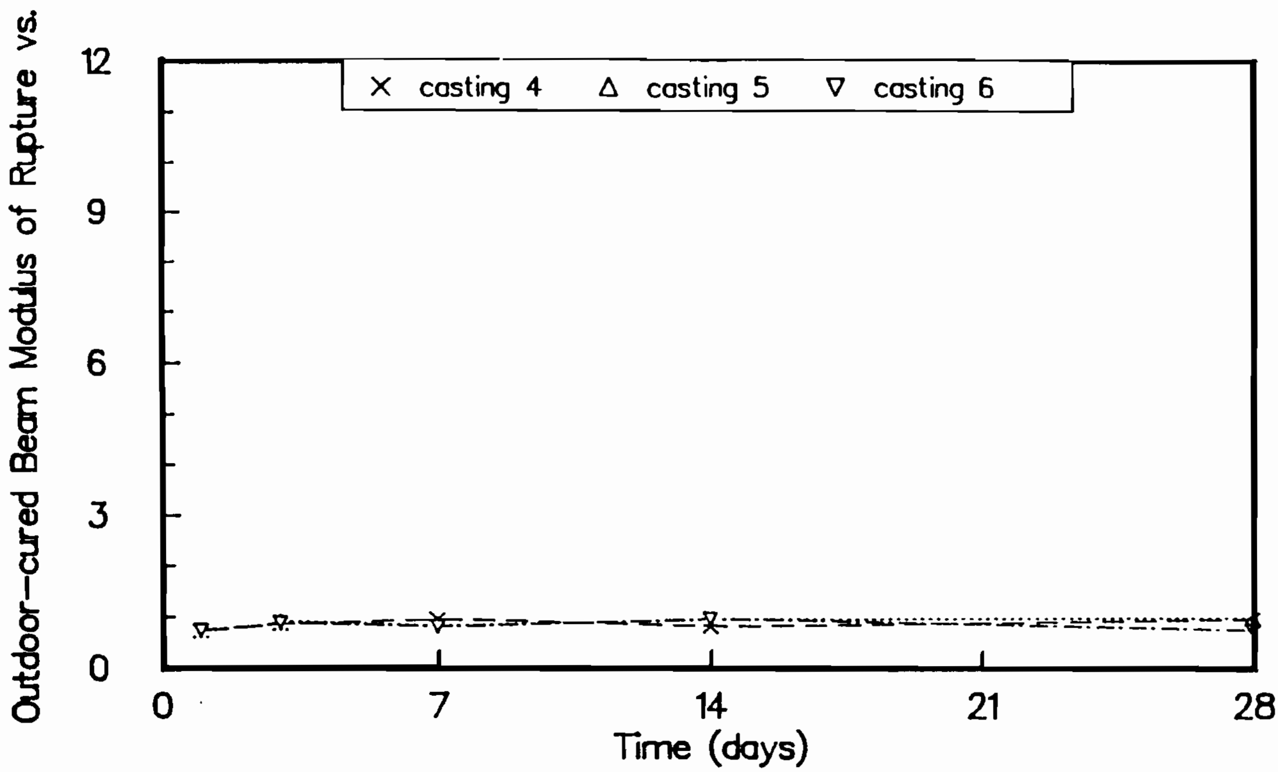


**Fig B-26. Ratio of flexural strengths of outdoor-cured beams to flexural strengths of wetroom-cured beams for castings 4, 5, and 6.**

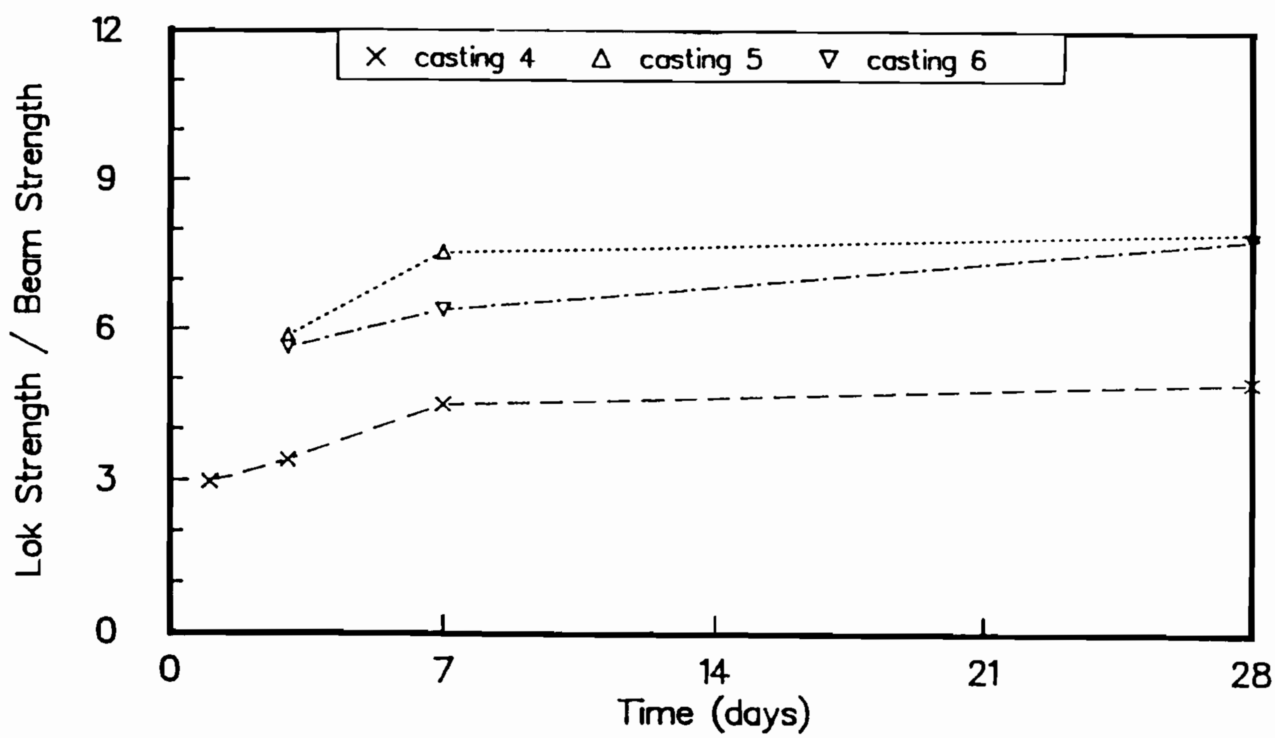


**Fig B-27. Ratio of "Lok-Test" pullout strengths to flexural strengths of wetroom-cured beams for castings 4, 5, and 6.**

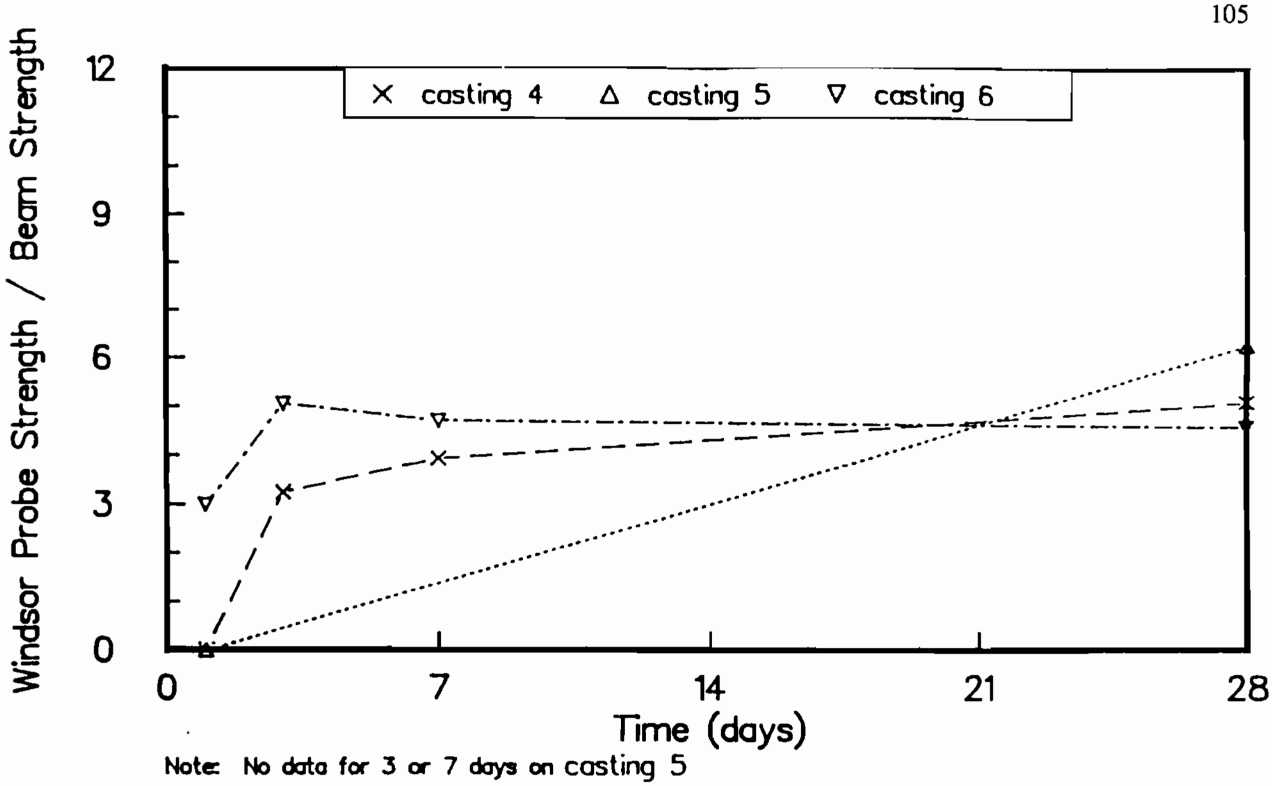


Fig B-28. Ratio of Windsor probe penetration resistance strengths to flexural strengths of wetroom-cured beams for castings 4, 5, and 6.

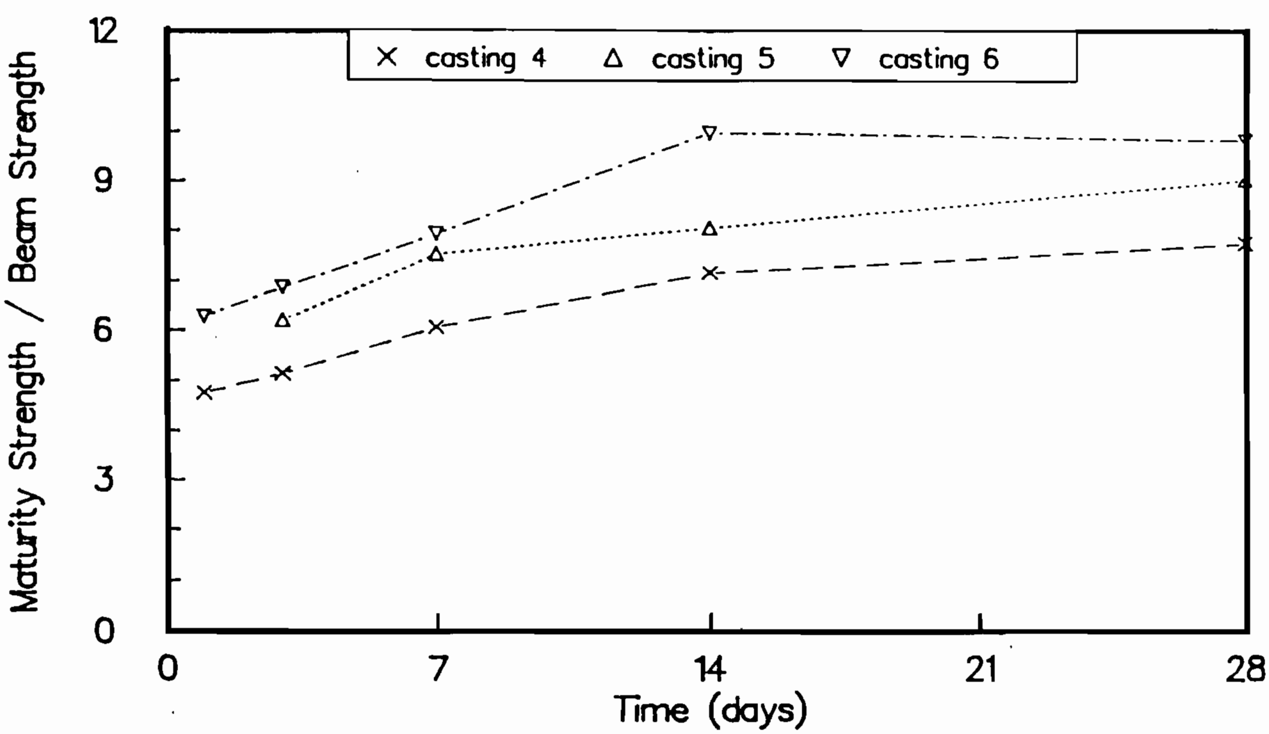


Fig B-29. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 4, 5, and 6. Maturity curve based on wetroom-cured cylinders.

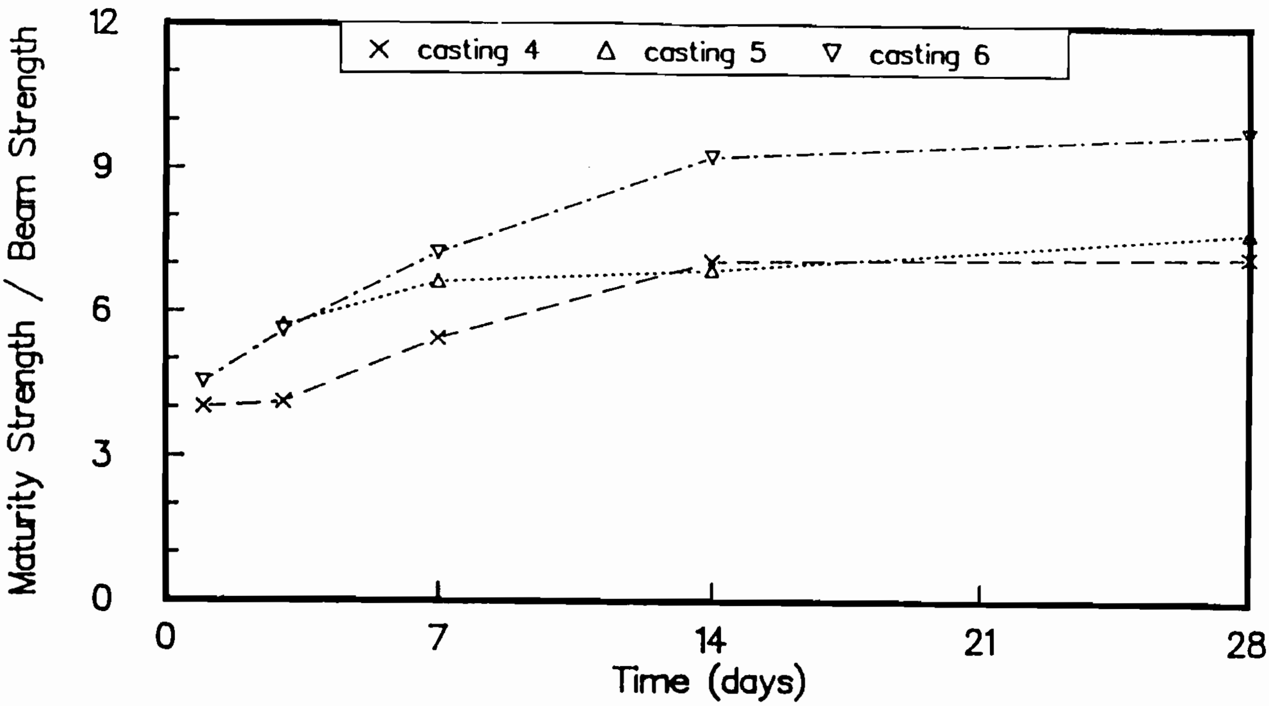


**Fig B-30. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 4, 5, and 6. Maturity curve based on outdoor-cured cylinders.**

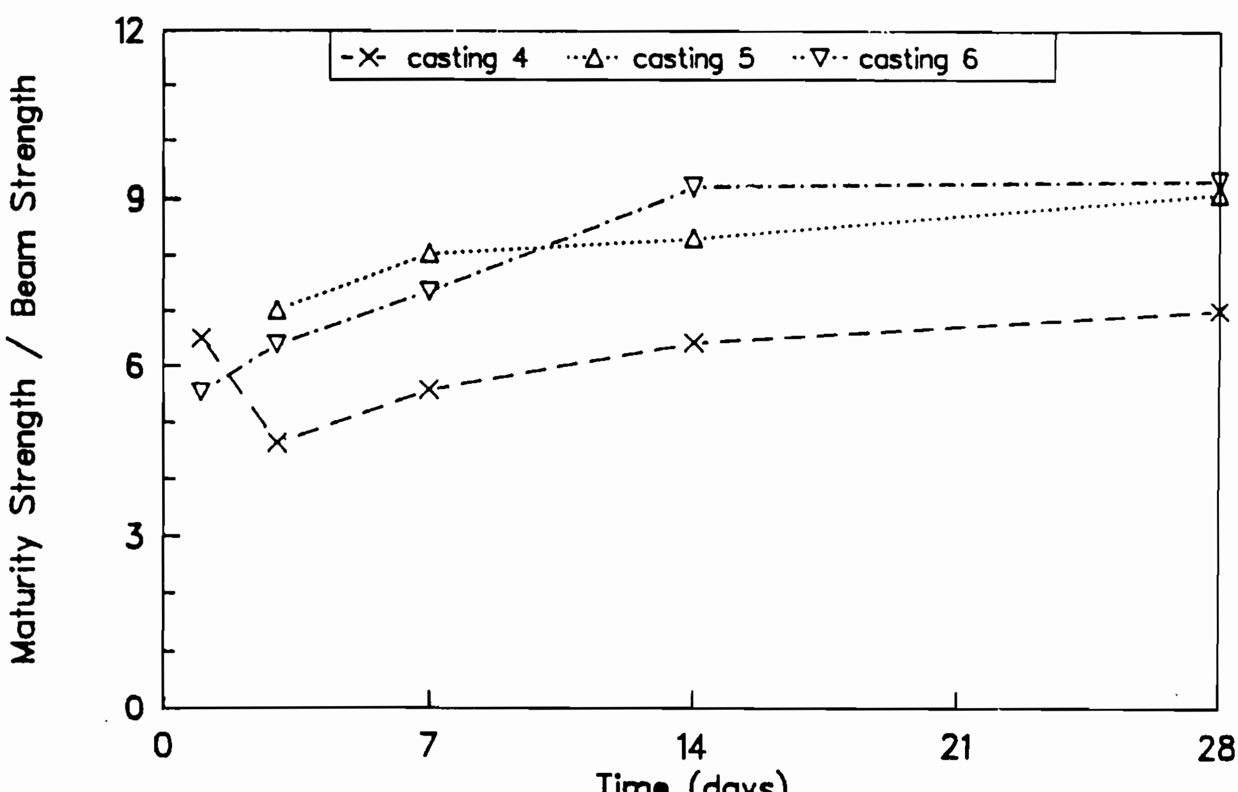


**Fig B-31. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 4, 5, and 6. Maturity curve based on cores drilled from slabs.**

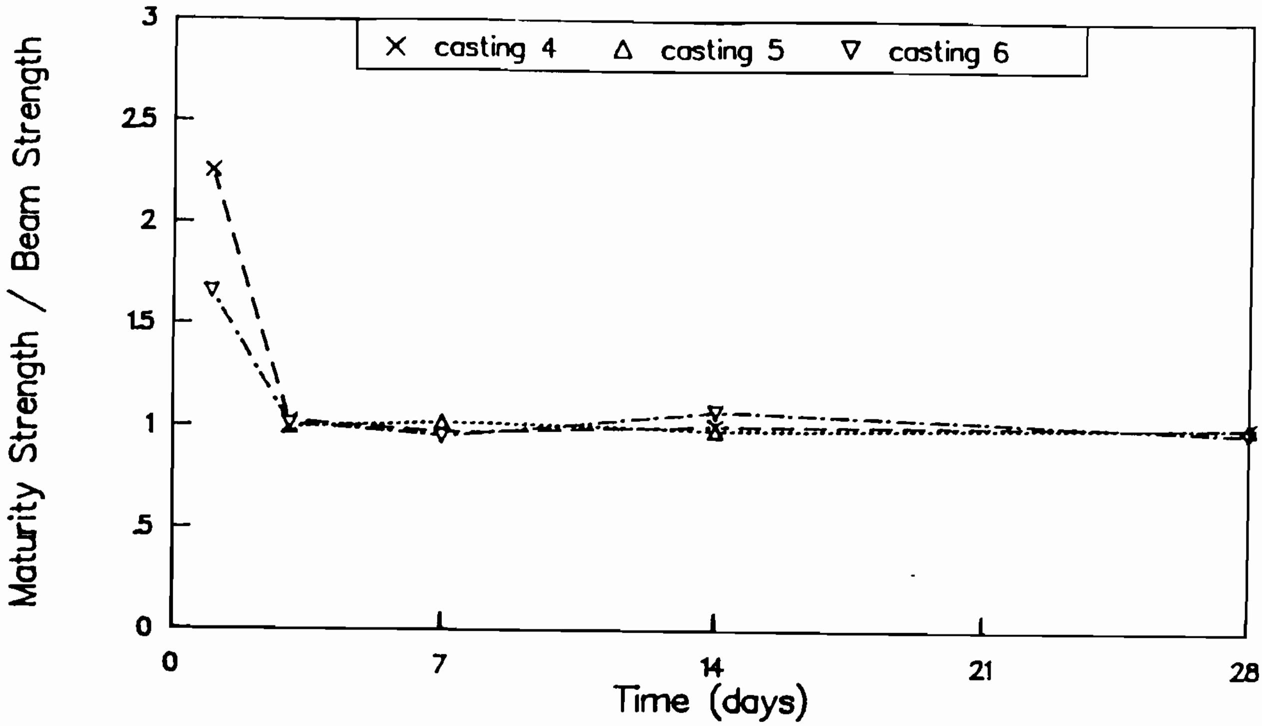


Fig B-32. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 4, 5, and 6. Maturity curve based on wetroom-cured beams.

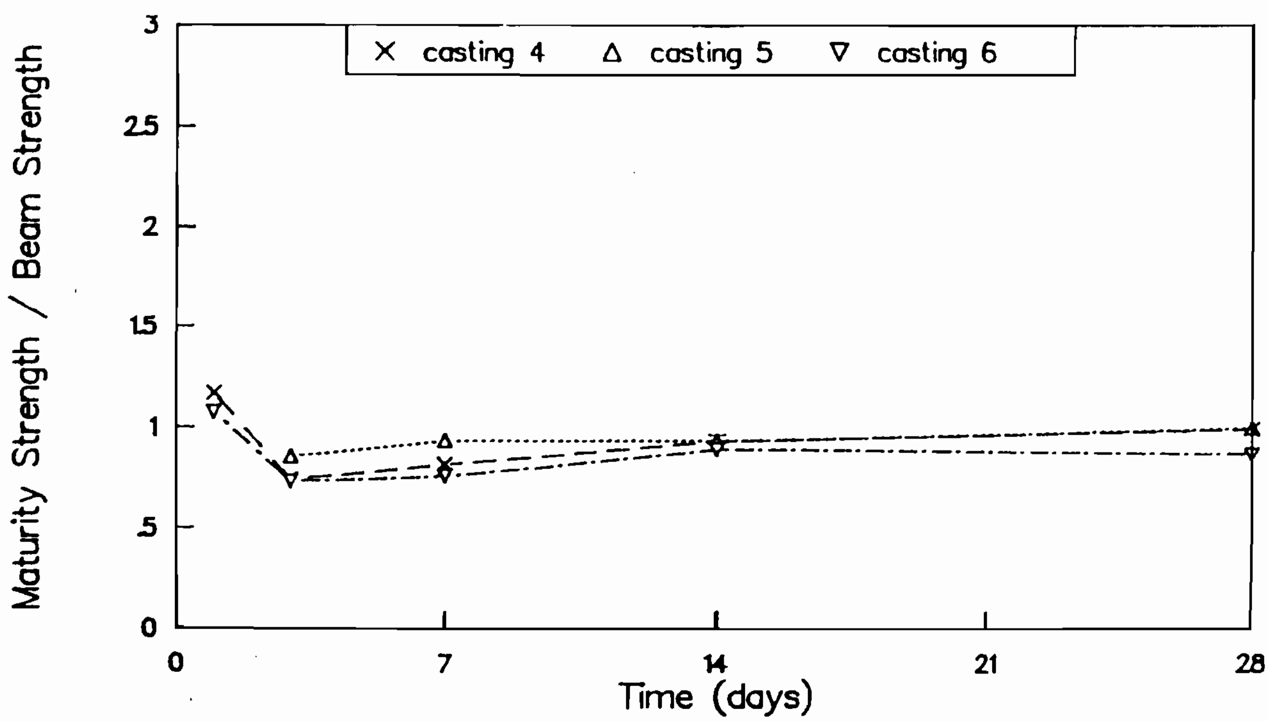


Fig B-33. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 4, 5, and 6. Maturity curve based on outdoor-cured beams.

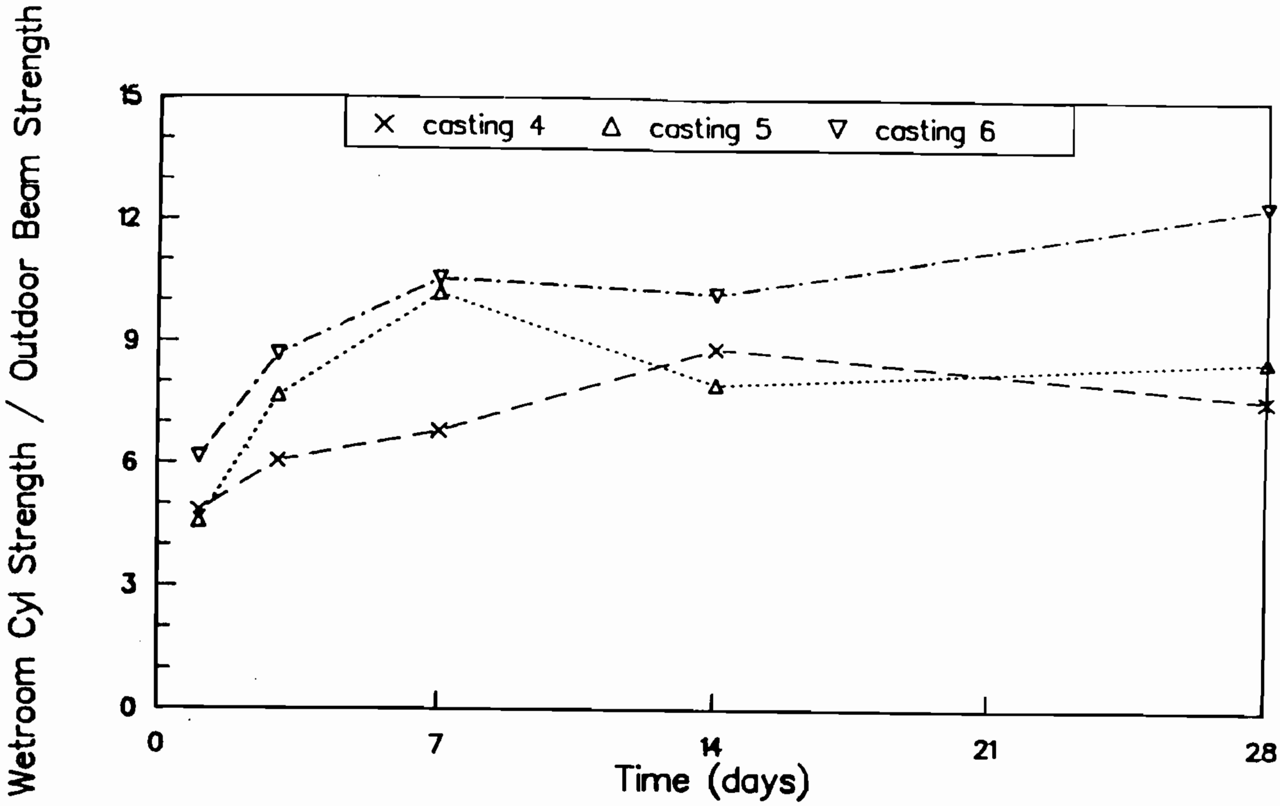


**Fig B-34. Ratio of strengths of wetroom-cured cylinders to flexural strengths of outdoor-cured beams for castings 4, 5, and 6.**

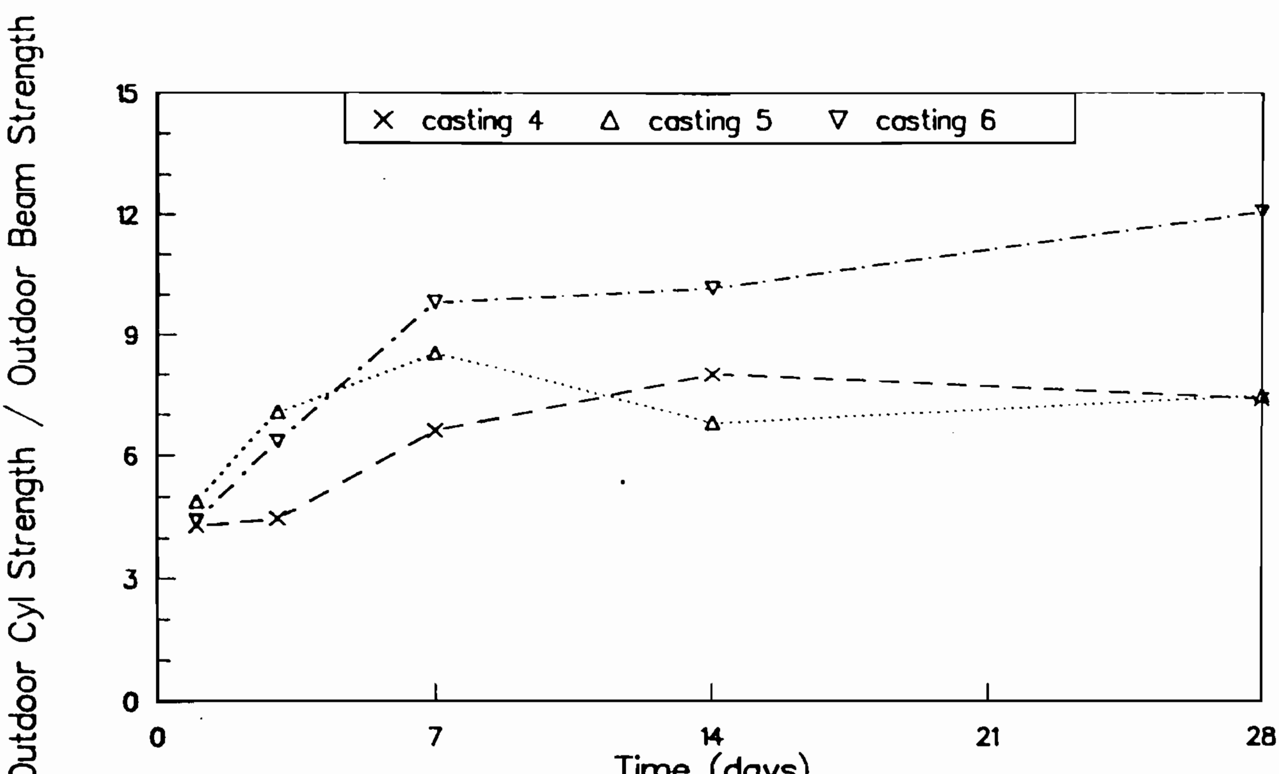


**Fig B-35. Ratio of strengths of outdoor-cured cylinders to flexural strengths of outdoor-cured beams for castings 4, 5, and 6.**

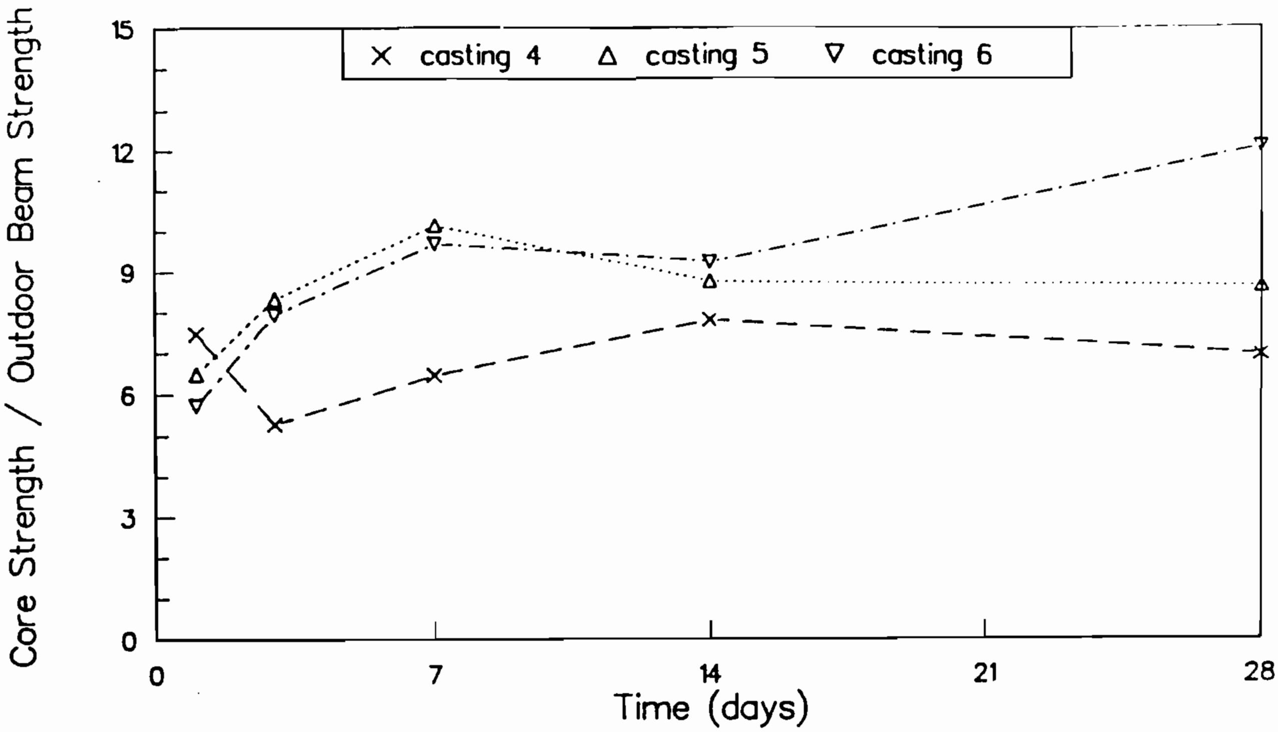


Fig B-36. Ratio of strengths of cores to flexural strengths of outdoor-cured beams for castings 4, 5, and 6.

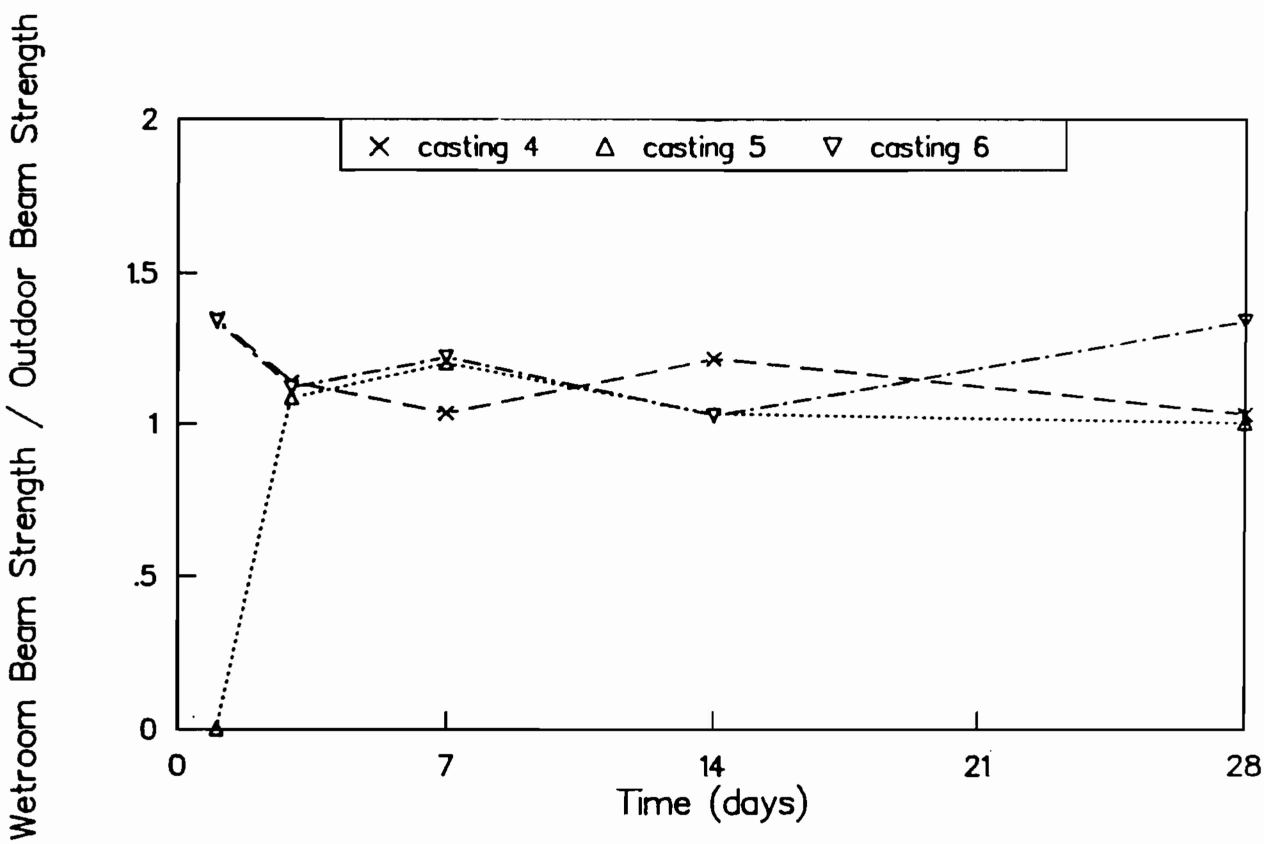


Fig B-37. Ratio of flexural strengths of wetroom-cured beams to flexural strengths of outdoor-cured beams for castings 4, 5, and 6.

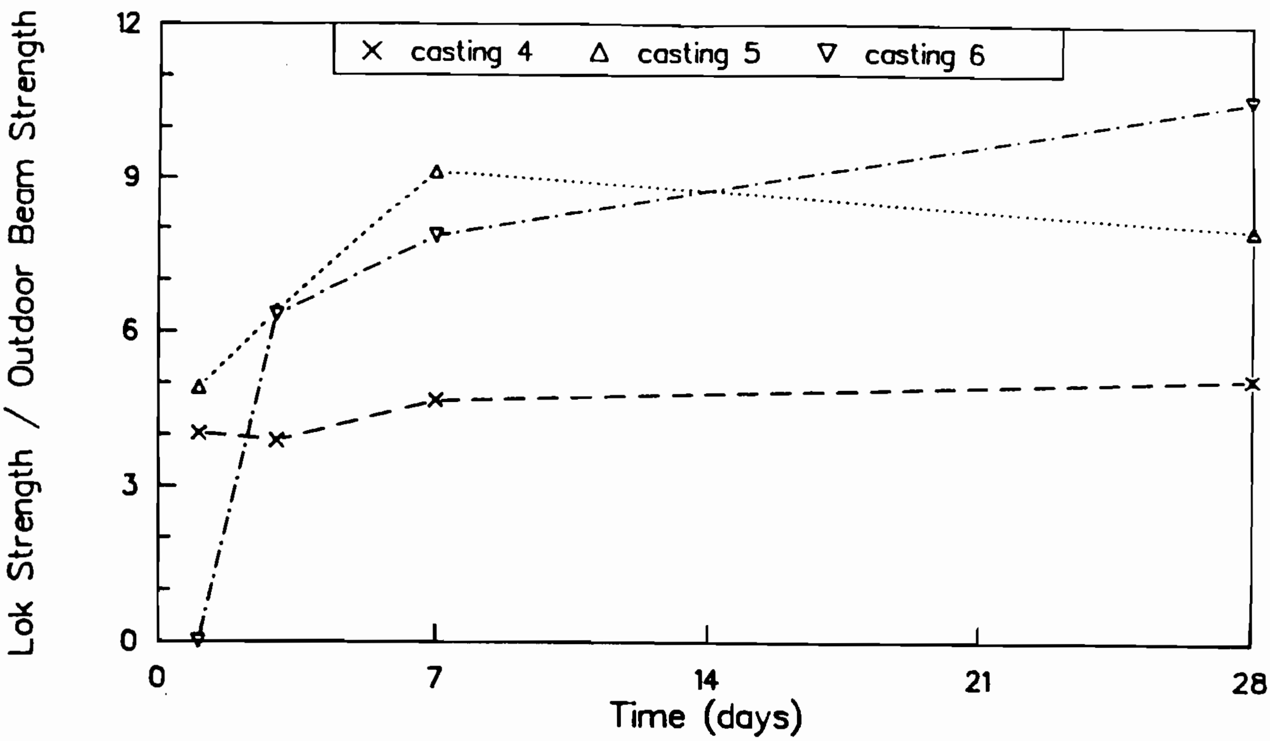


**Fig B-38. Ratio of "Lok-Test" pullout strengths to flexural strengths of outdoor-cured beams for castings 4, 5, and 6.**

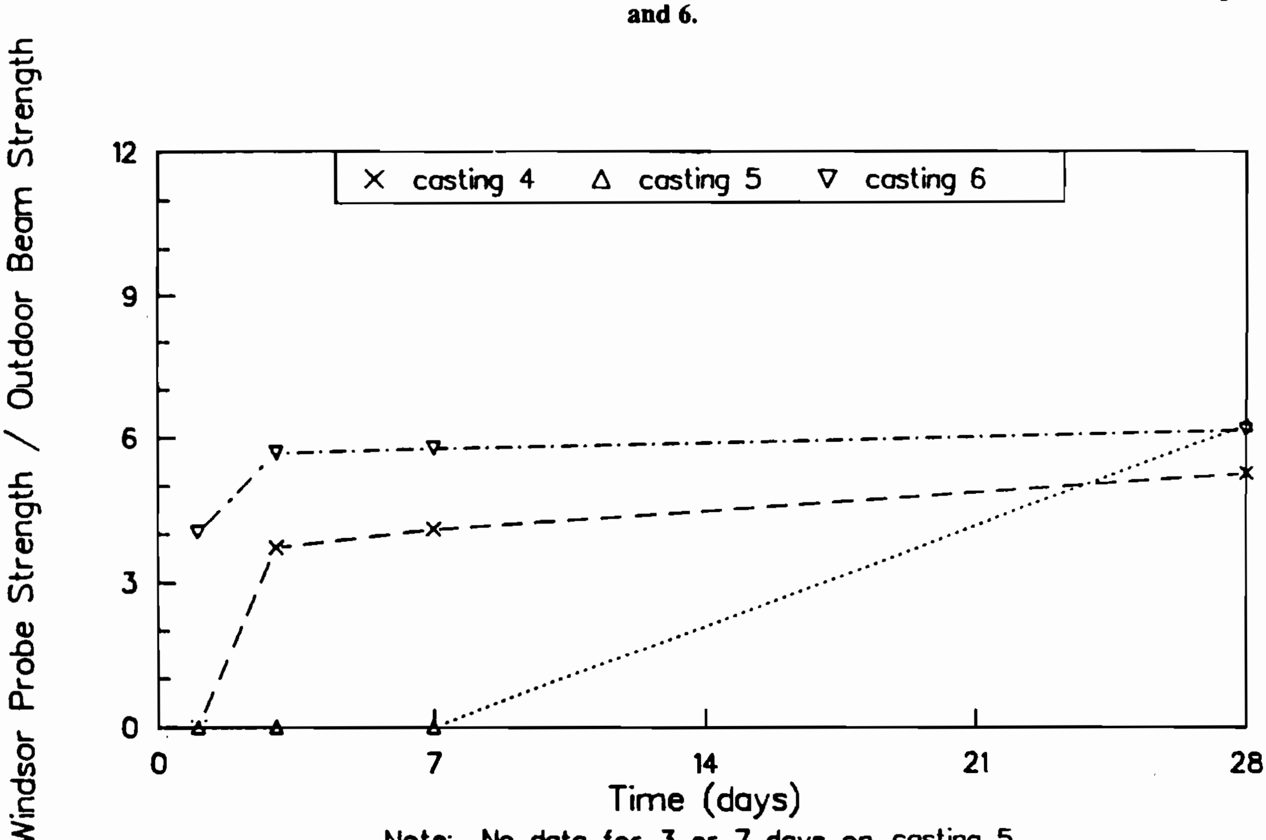


**Fig B-39. Ratio of Windsor probe penetration resistance strengths to flexural strengths of outdoor-cured beams for castings 4, 5, and 6.**

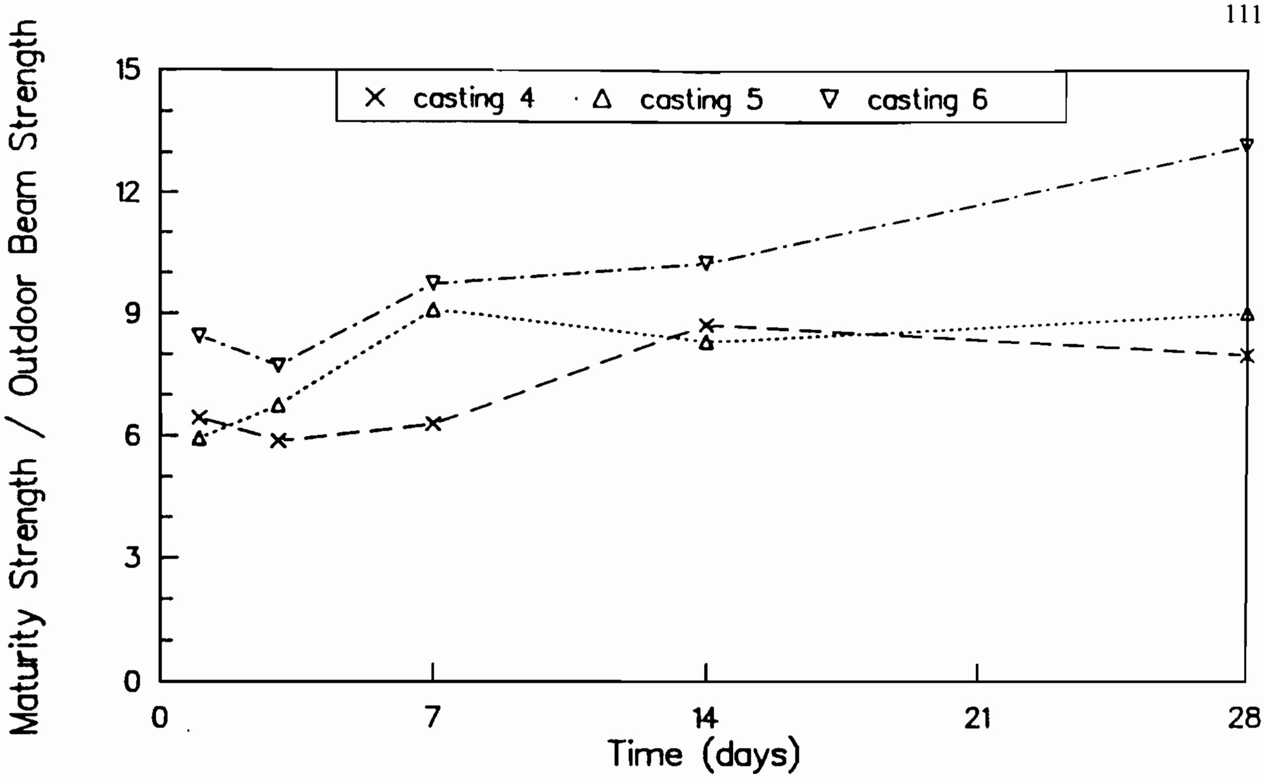


Fig B-40. Ratio of maturity method strengths to flexural strengths of outdoor-cured beams for castings 4, 5, and 6. Maturity curve based on wetroom-cured cylinders.

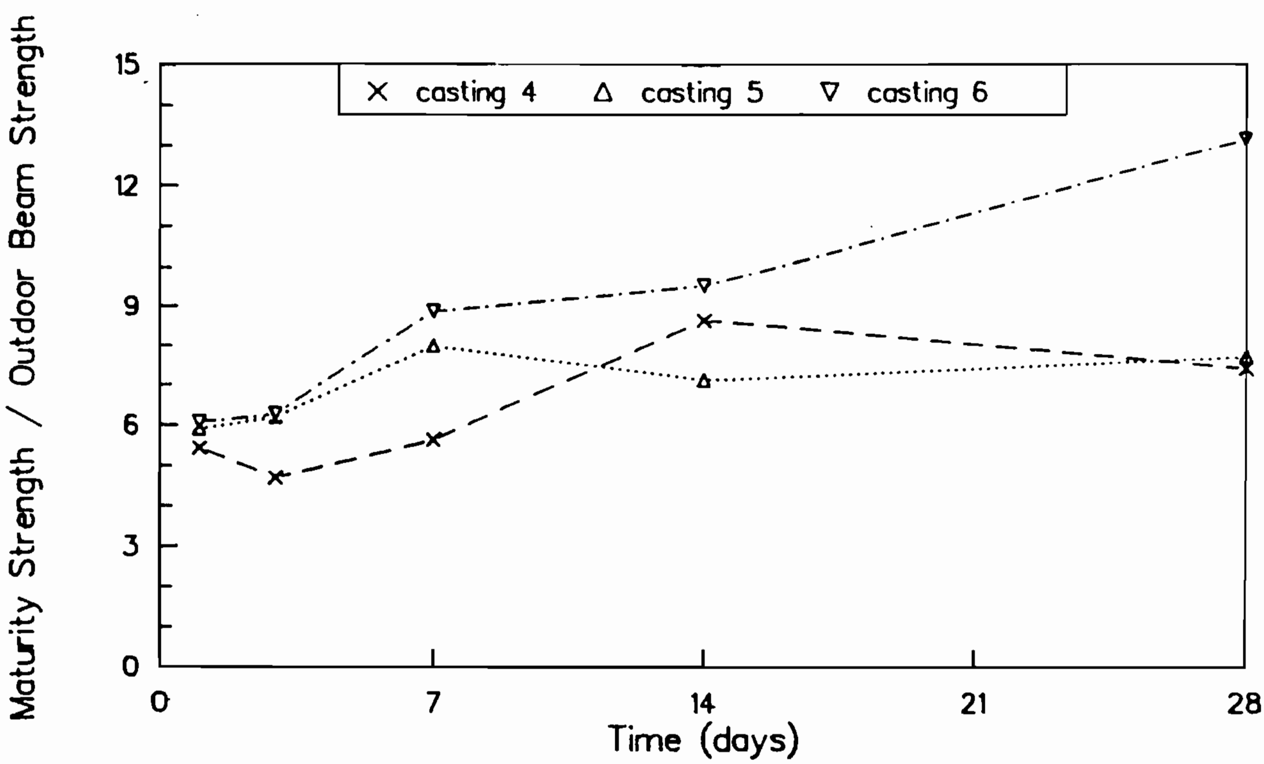


Fig B-41. Ratio of maturity method strengths to flexural strengths of outdoor-cured beams for castings 4, 5, and 6. Maturity curve based on outdoor-cured cylinders.

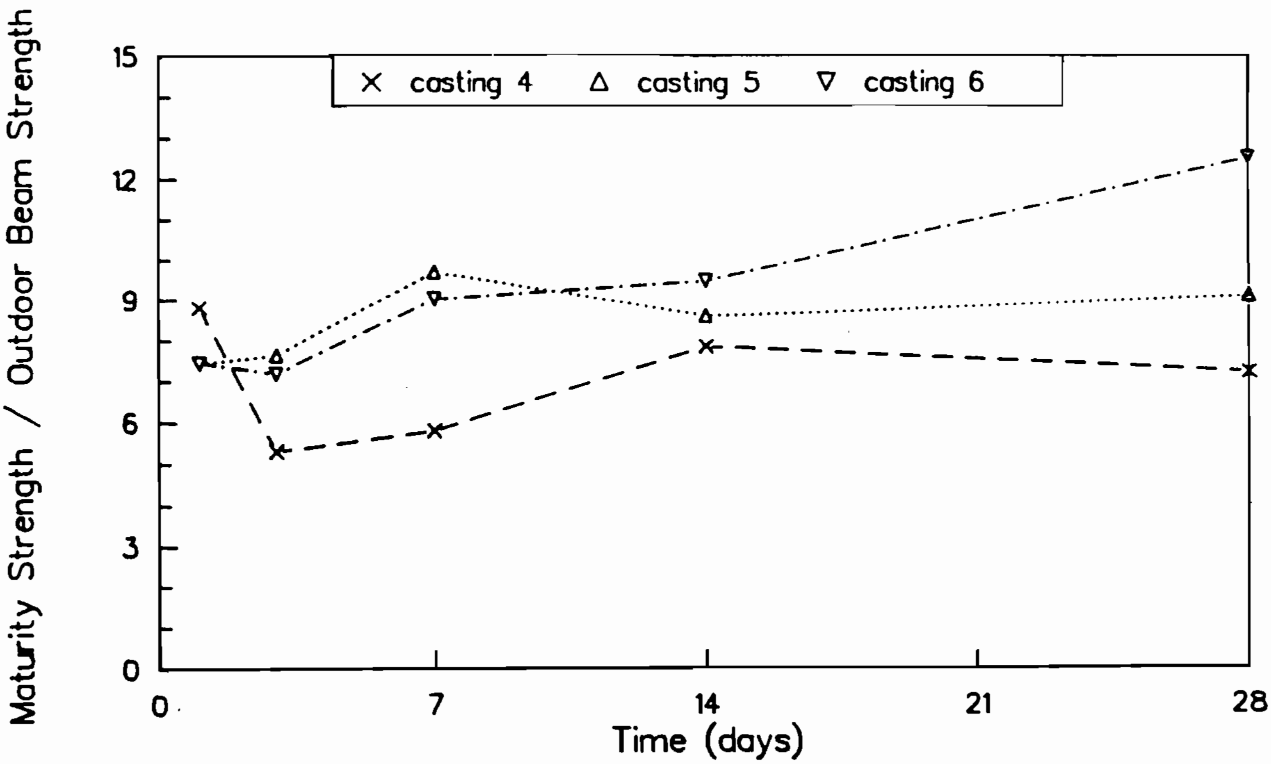


**Fig B-42. Ratio of maturity method strengths to flexural strengths of outdoor-cured beams for castings 4, 5, and 6. Maturity curve based on cores drilled from slabs.**

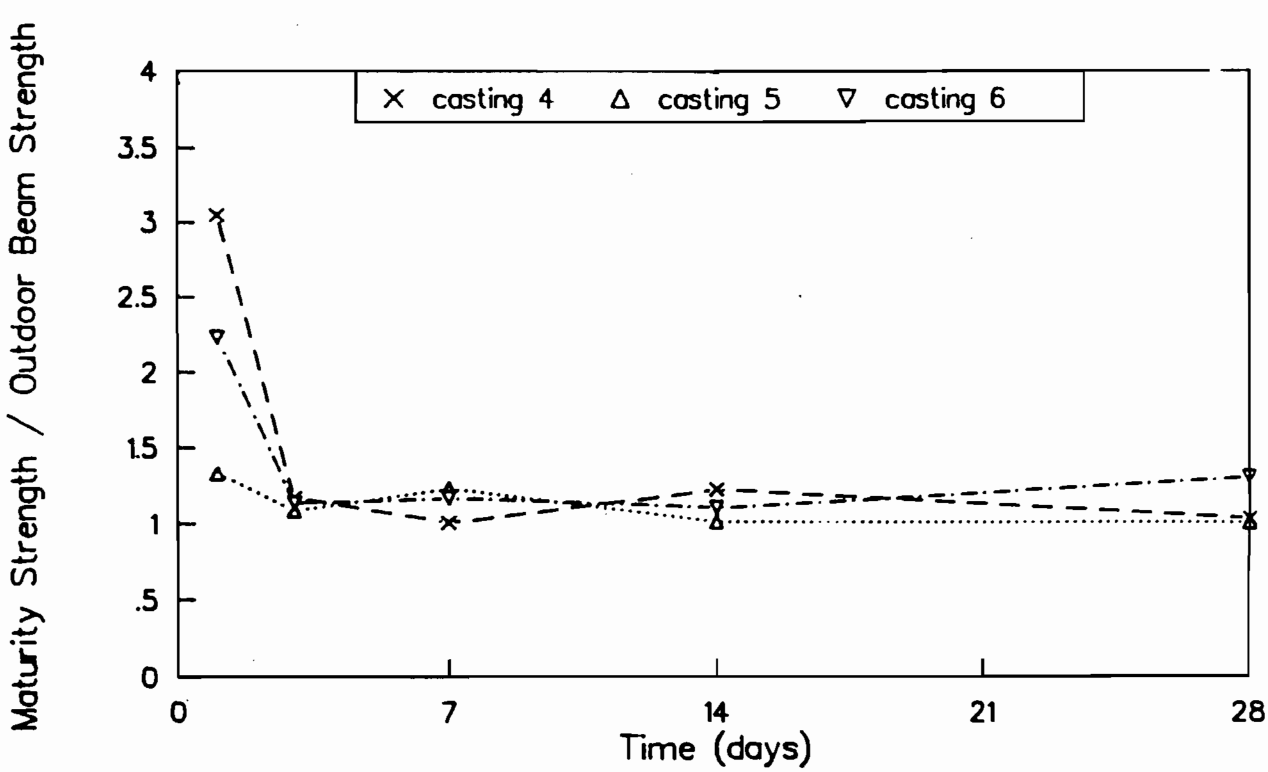


**Fig B-43. Ratio of maturity method strengths to flexural strengths of outdoor-cured beams for castings 4, 5, and 6. Maturity curve based on wetroom-cured beams.**

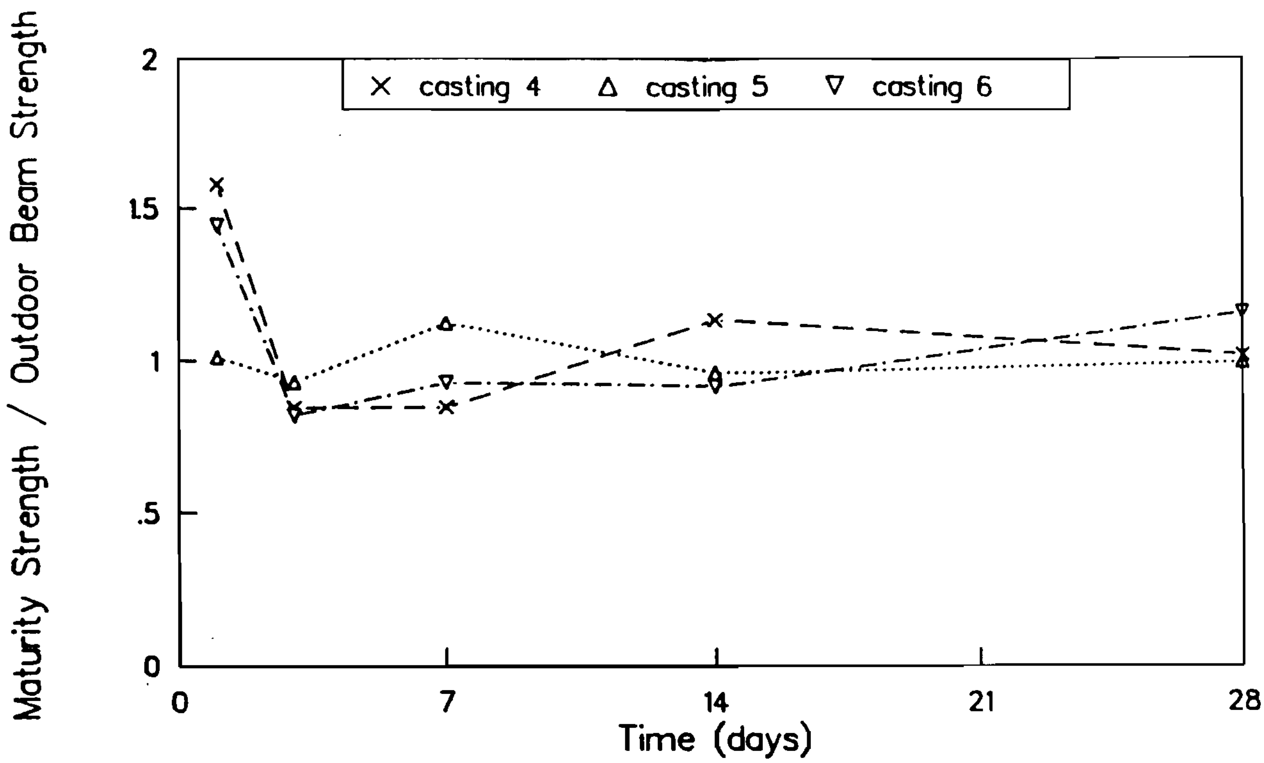


**Fig B-44. Ratio of maturity method strengths to flexural strengths of outdoor-cured beams for castings 4, 5, and 6. Maturity curve based on outdoor-cured beams.**

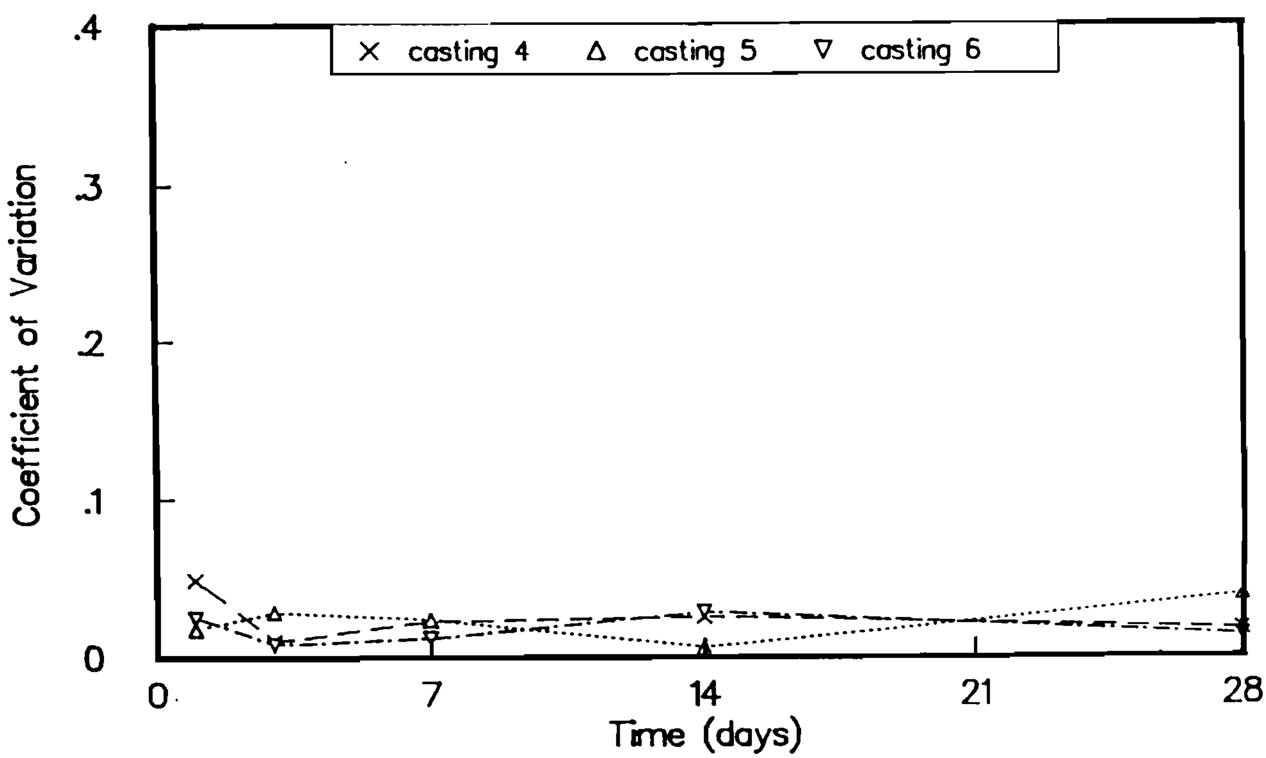


**Fig B-45. Coefficient of variation of wetroom-cured cylinder strengths for castings 4, 5, and 6.**

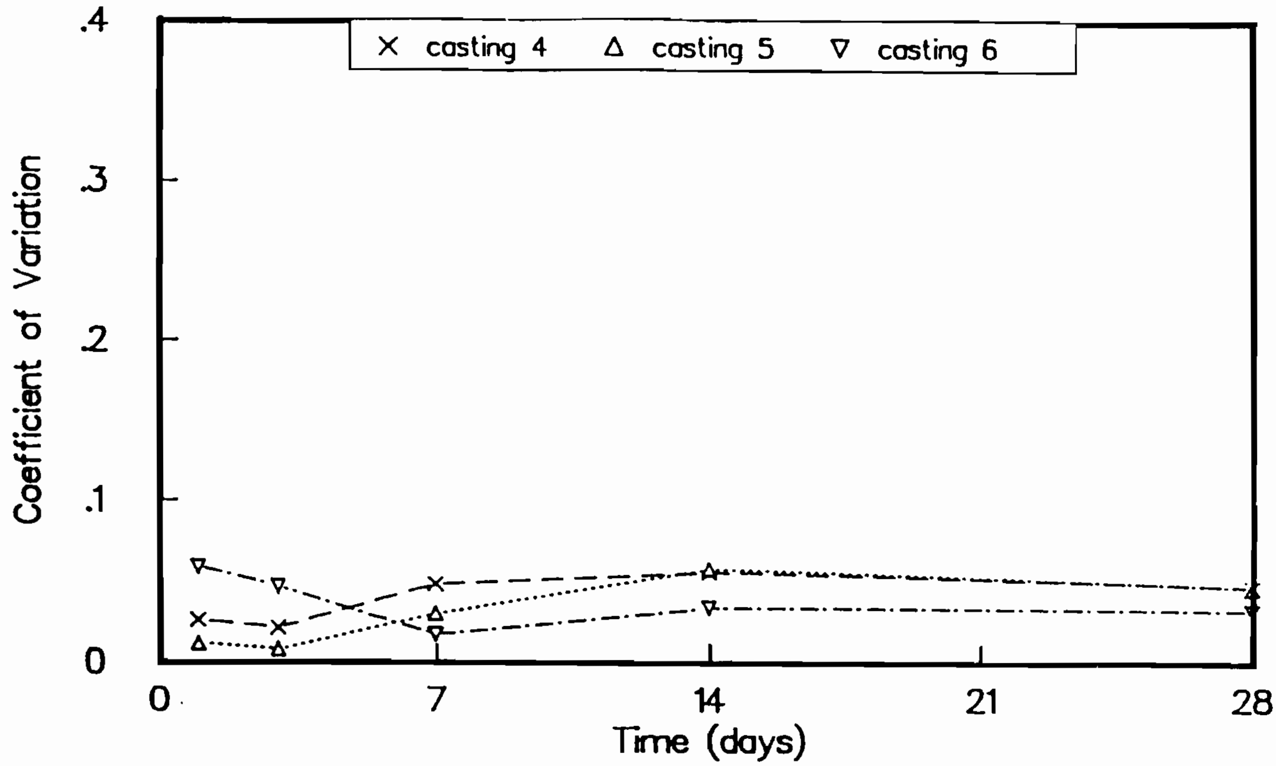


Fig B-46. Coefficient of variation of outdoor-cured cylinder strengths for castings 4, 5, and 6.

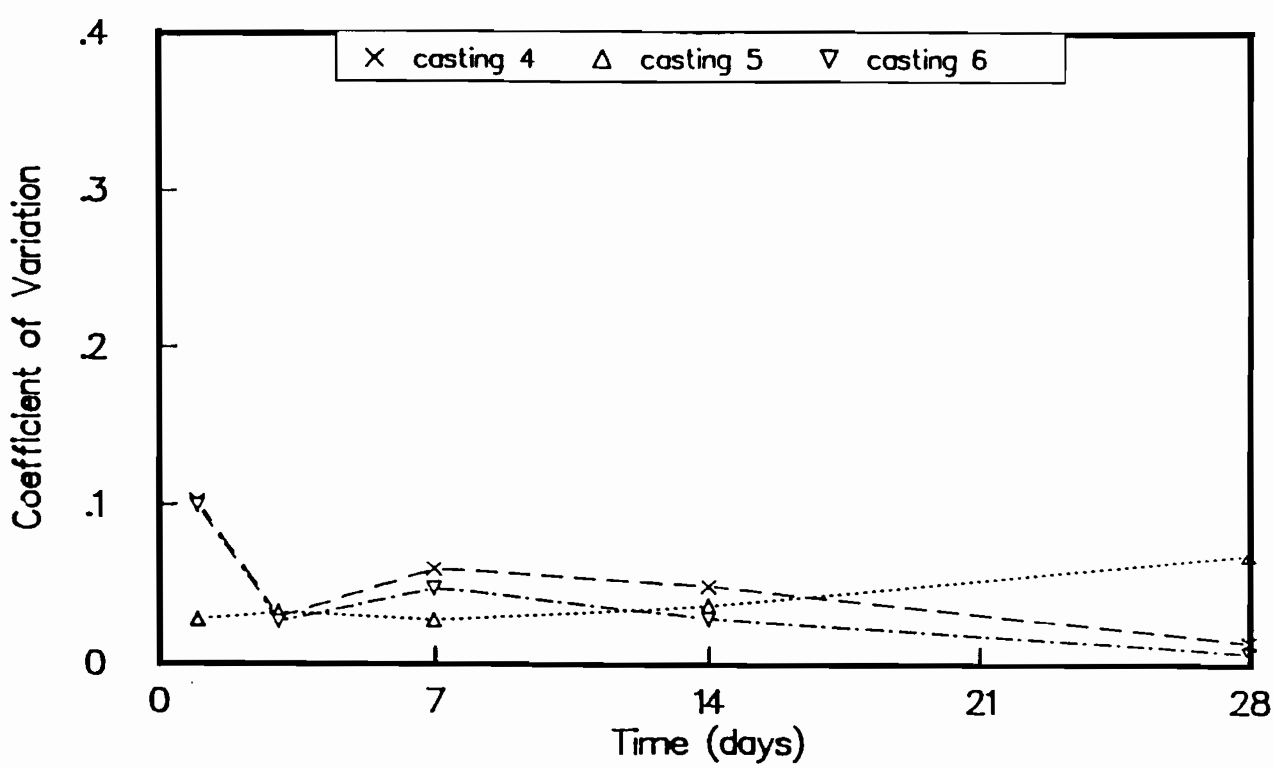


Fig B-47. Coefficient of variation of cores drilled from slabs for castings 4, 5, and 6.

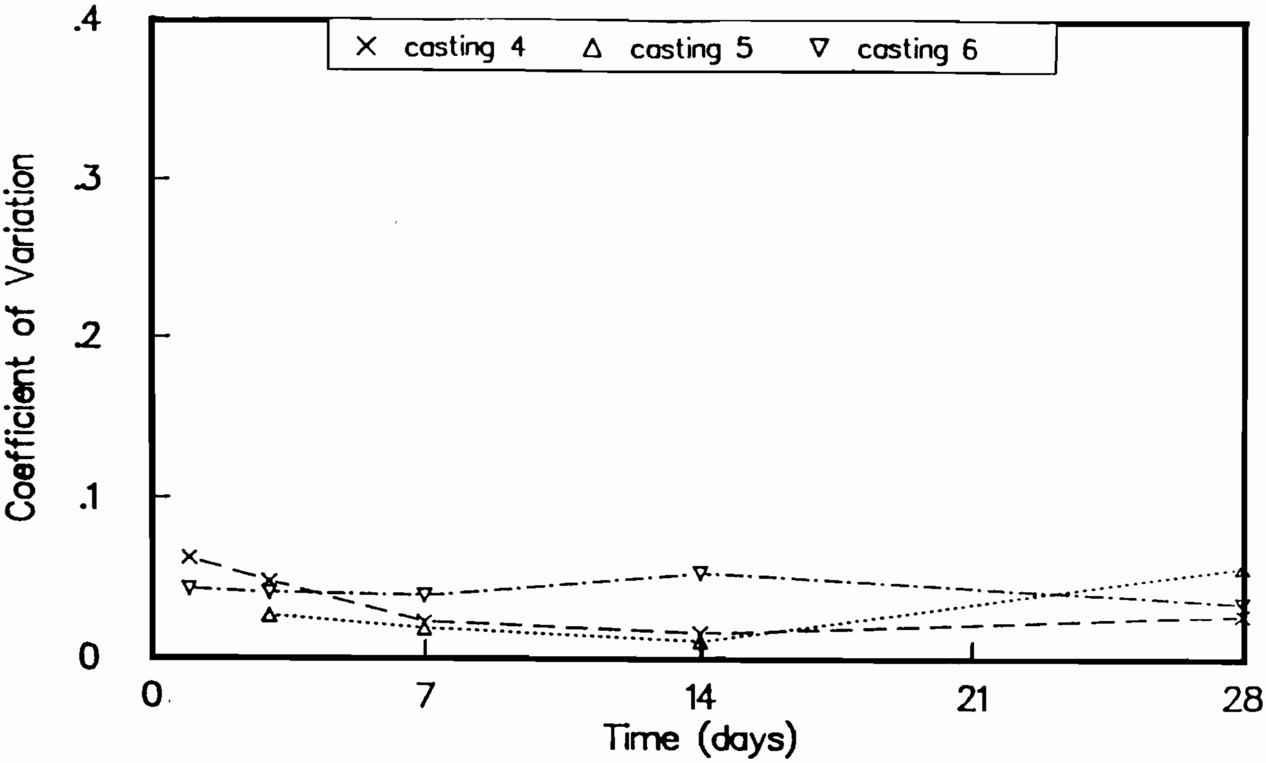


**Fig B-48. Coefficient of variation of flexural strength of wetroom-cured beams for castings 4, 5, and 6.**

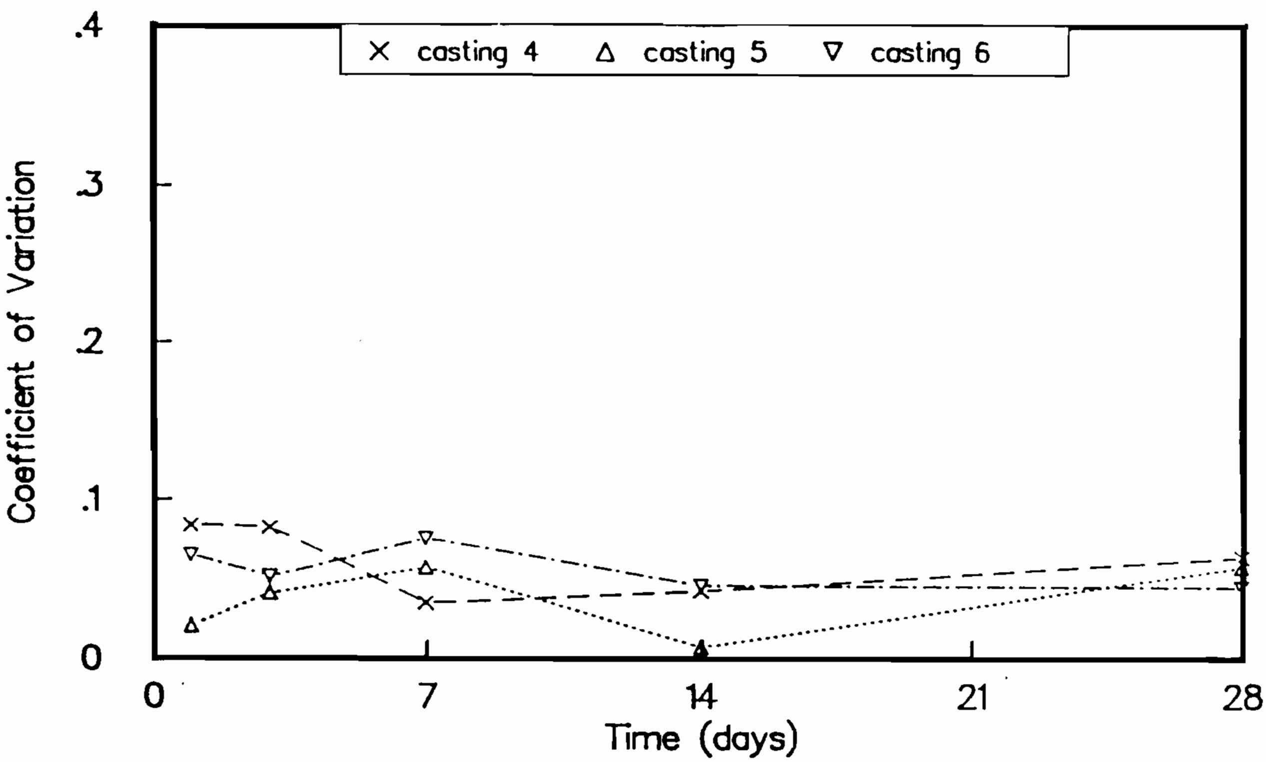


**Fig B-49. Coefficient of variation of flexural strength of outdoor-cured beams for castings 4, 5, and 6.**

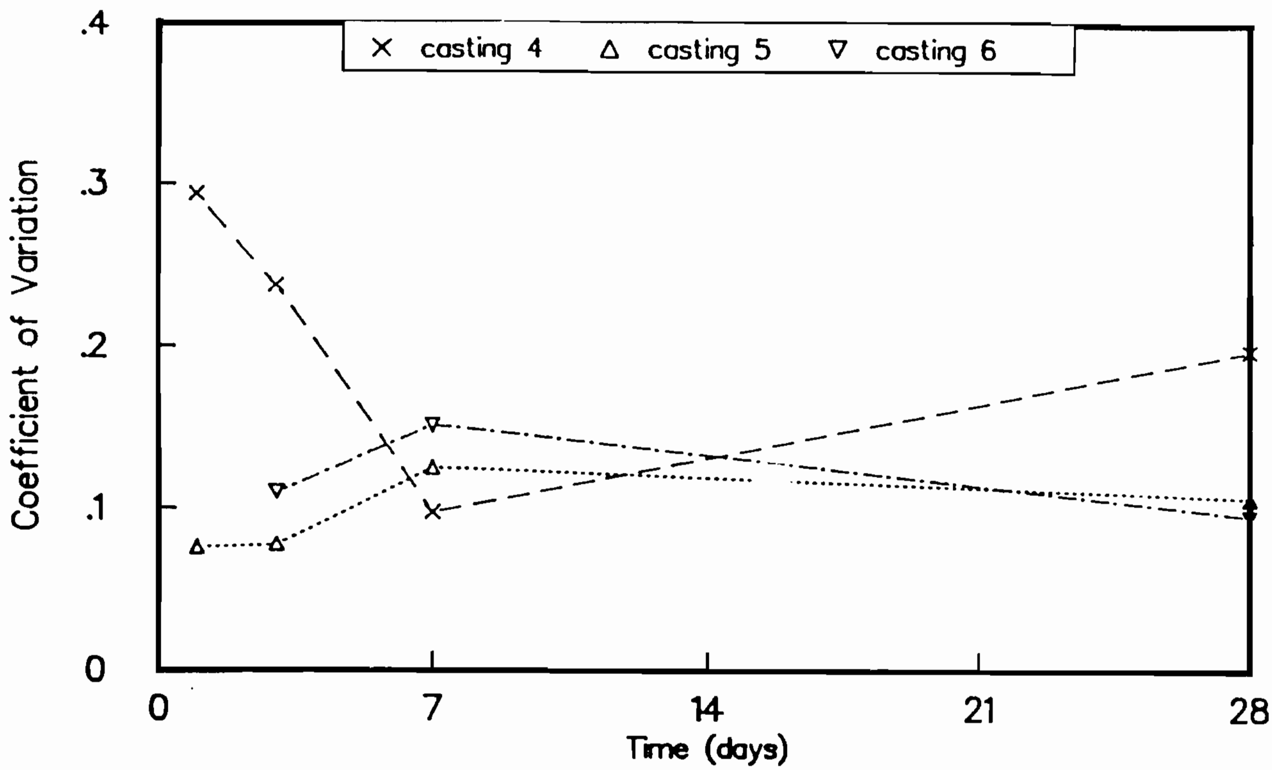


Fig B-50. Coefficient of variation of "Lok-Test" pullout strengths for castings 4, 5, and 6.

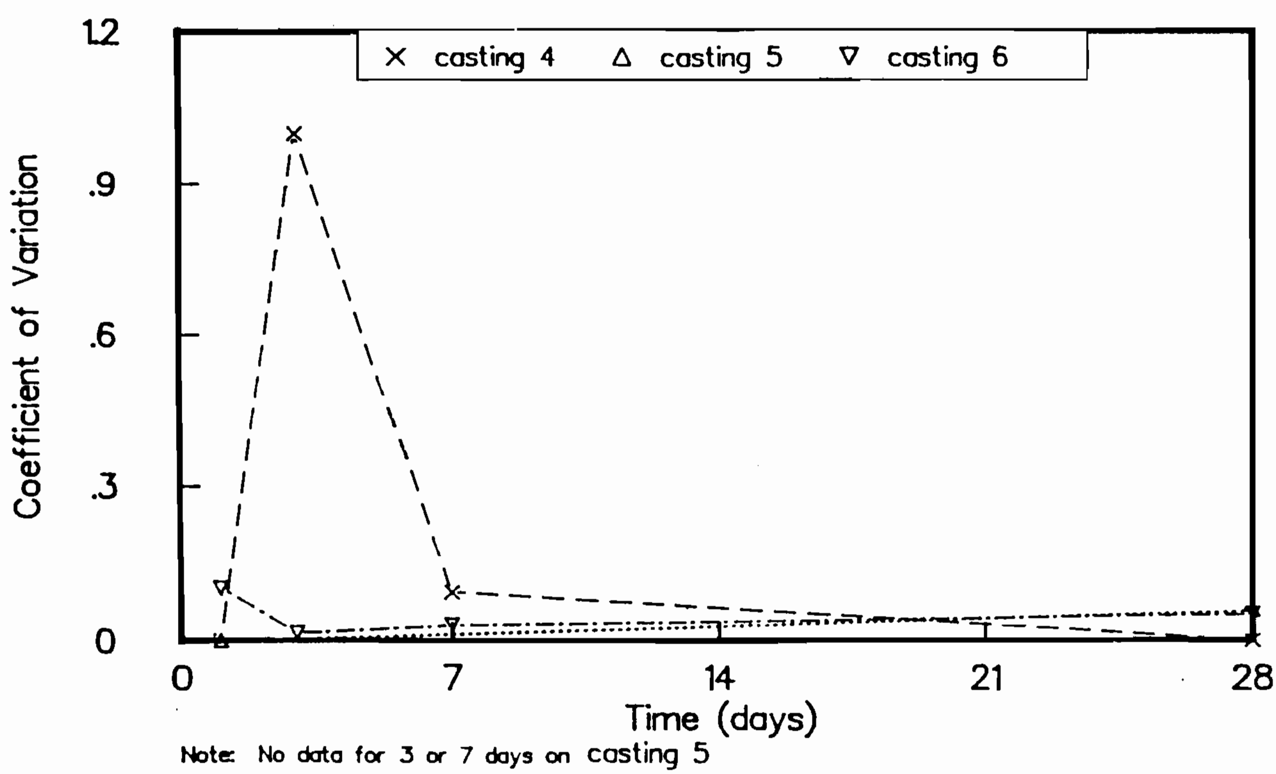


Fig B-51. Coefficient of variation of Windsor probe penetration resistance strengths for castings 4, 5, and 6.

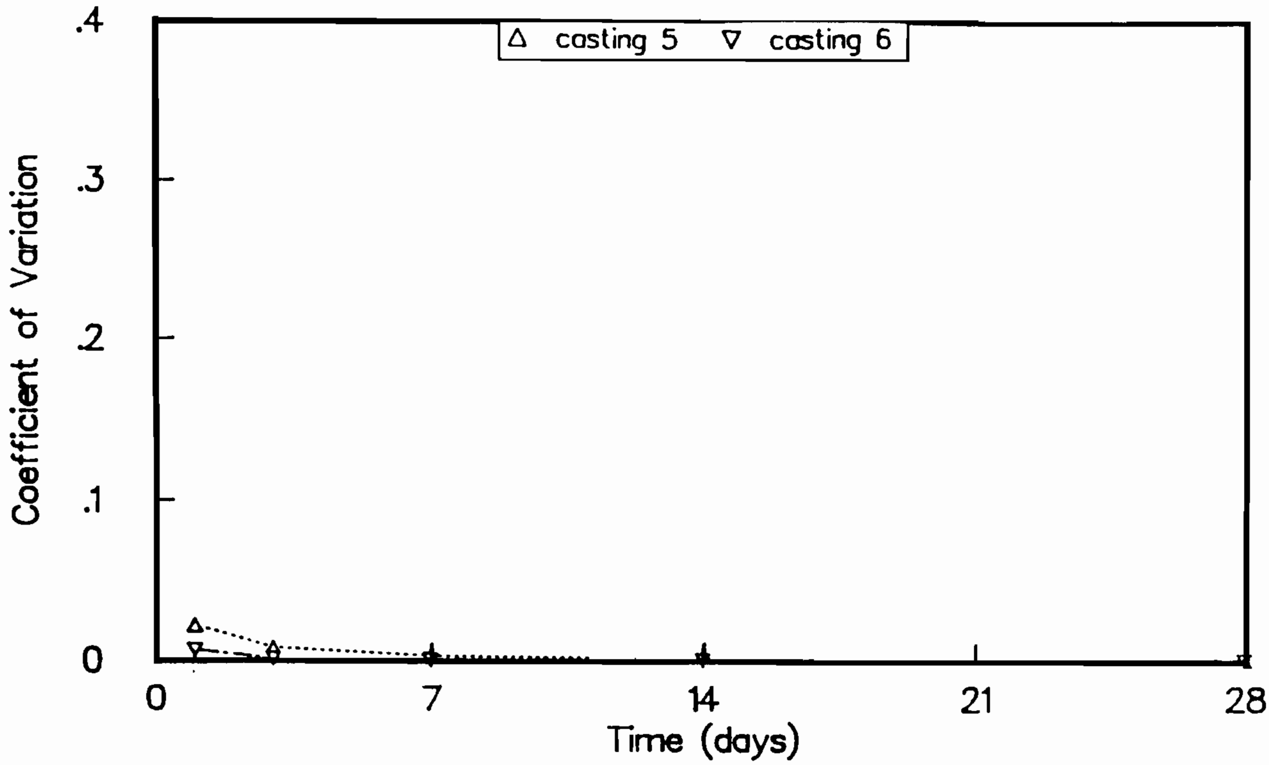


**Fig B-52. Coefficient of variation of maturity method strengths for castings 5 and 6. Maturity curve based on wetroom-cured cylinders.**

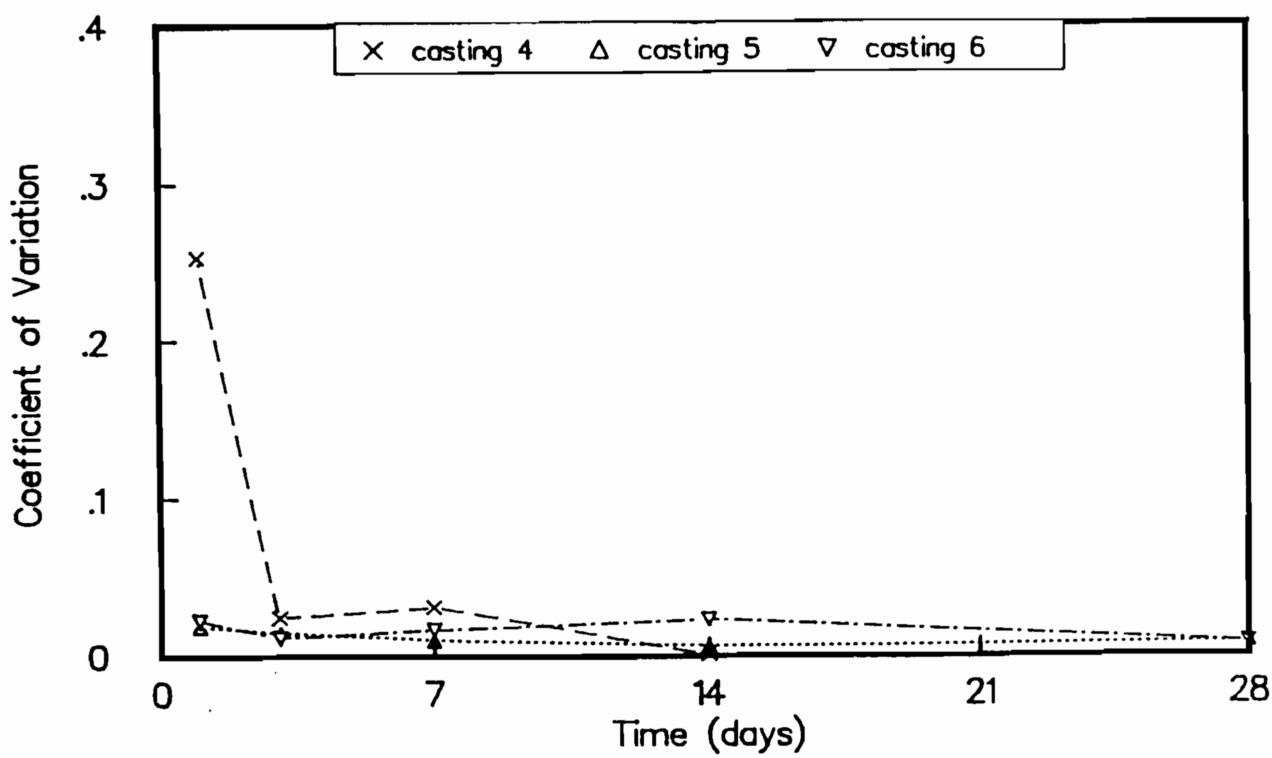


**Fig B-53. Coefficient of variation of maturity method strengths for castings 4, 5, and 6. Maturity curve based on outdoor-cured cylinders.**

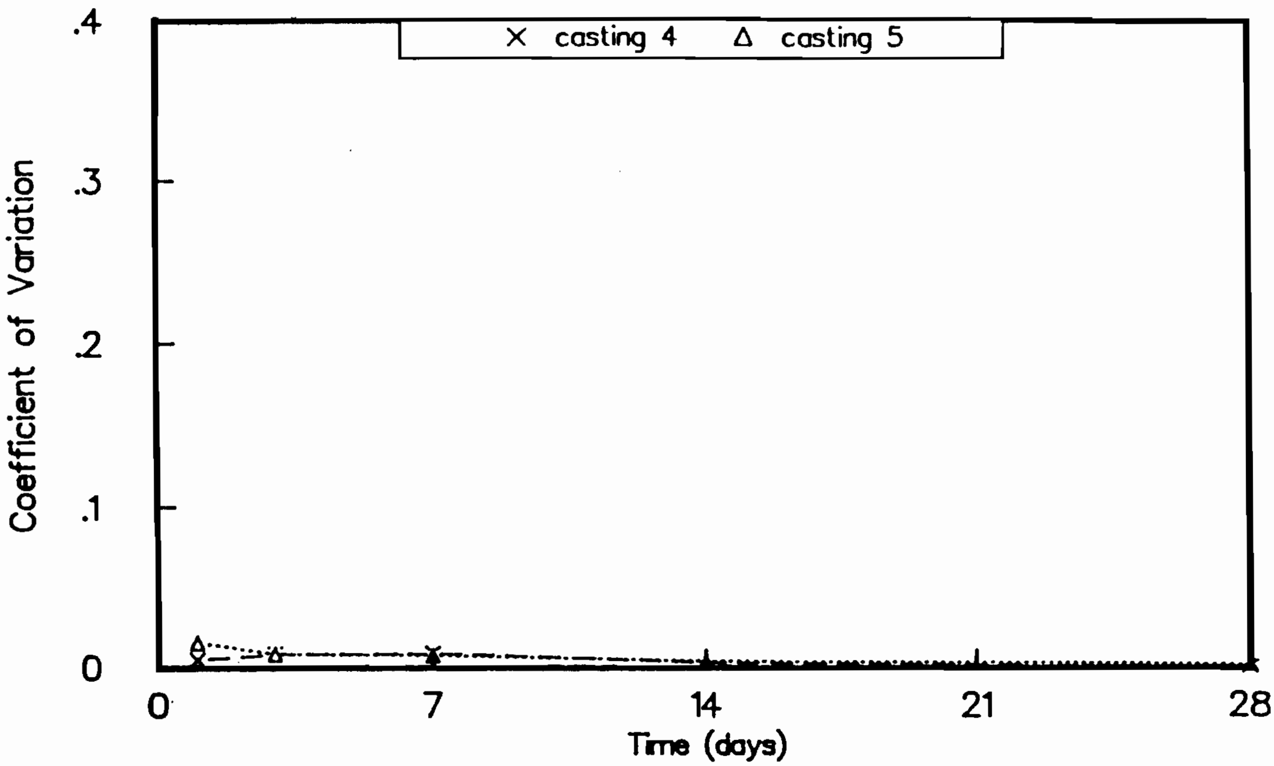


**Fig B-54. Coefficient of variation of maturity method strengths for castings 4 and 5. Maturity curve based on cores drilled from slabs.**

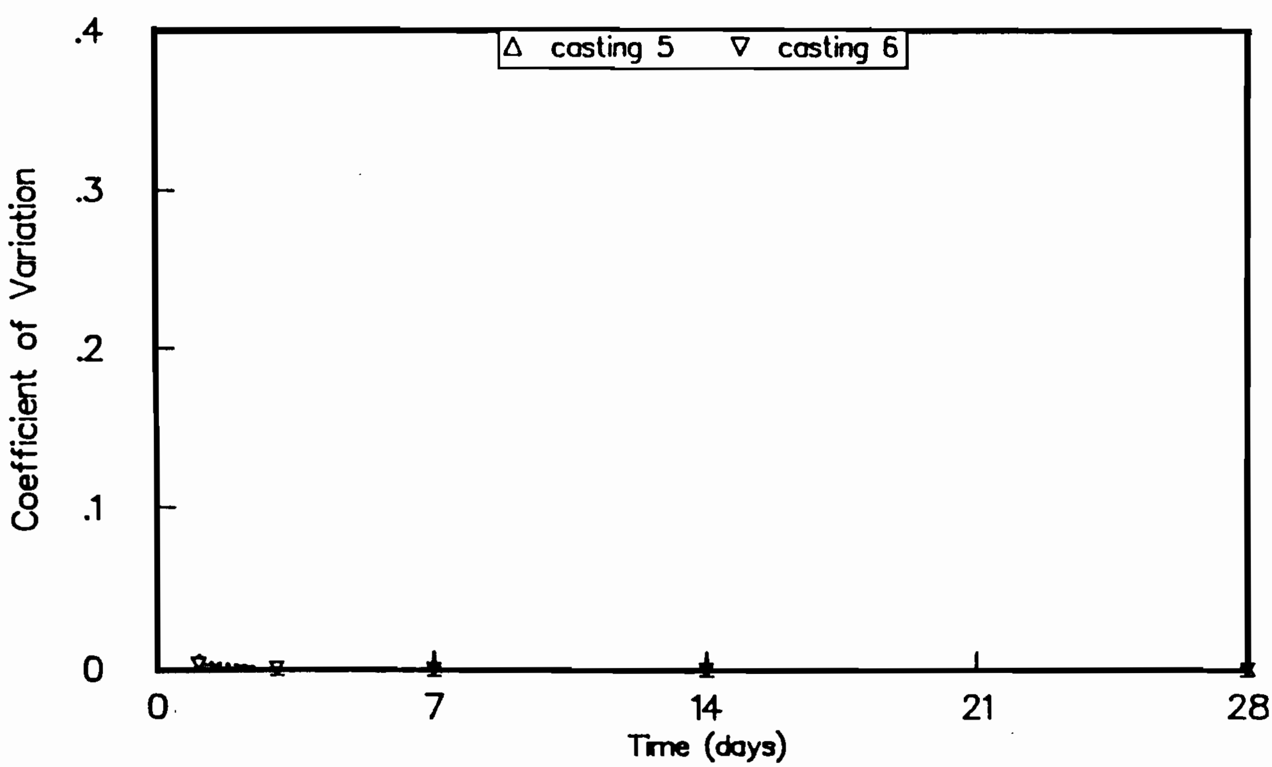


**Fig B-55. Coefficient of variation of maturity method strengths for castings 5 and 6. Maturity curve based on wetroom-cured beams.**

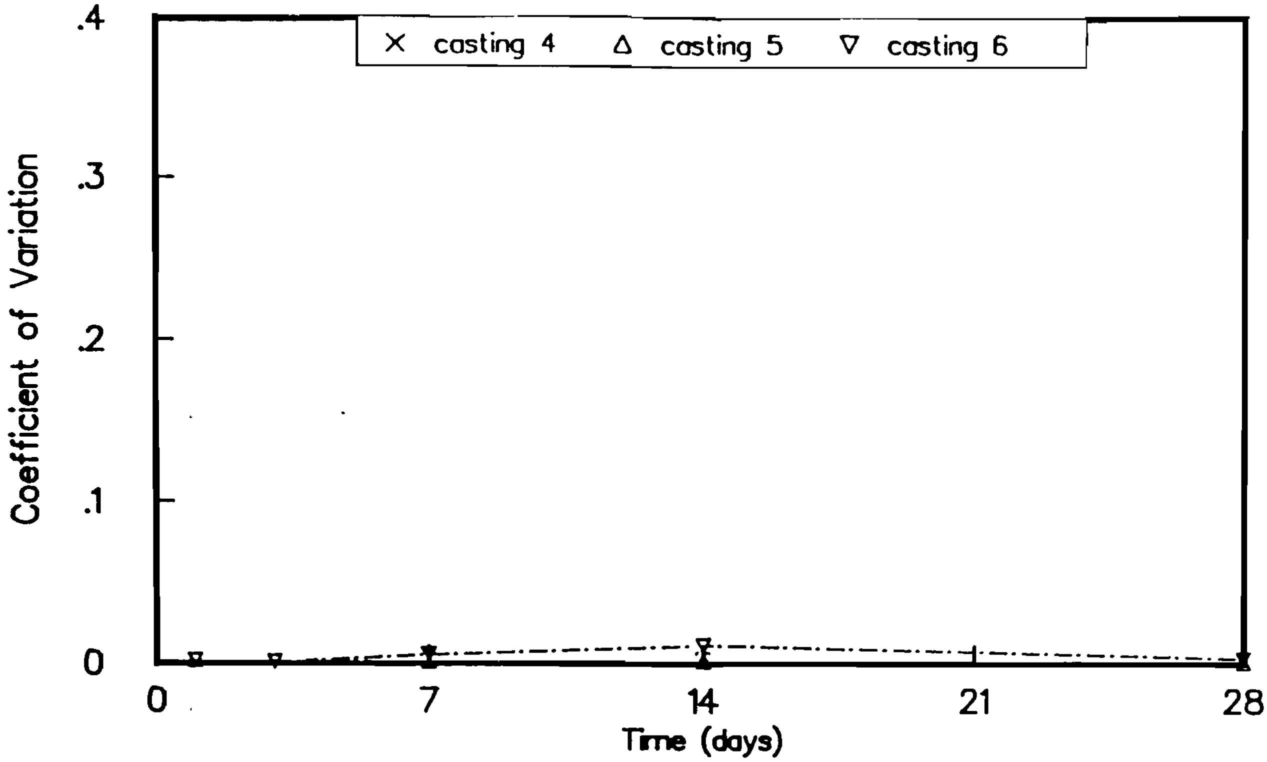


**Fig B-56. Coefficient of variation of maturity method strengths for castings 4, 5, and 6. Maturity curve based on outdoor-cured beams.**

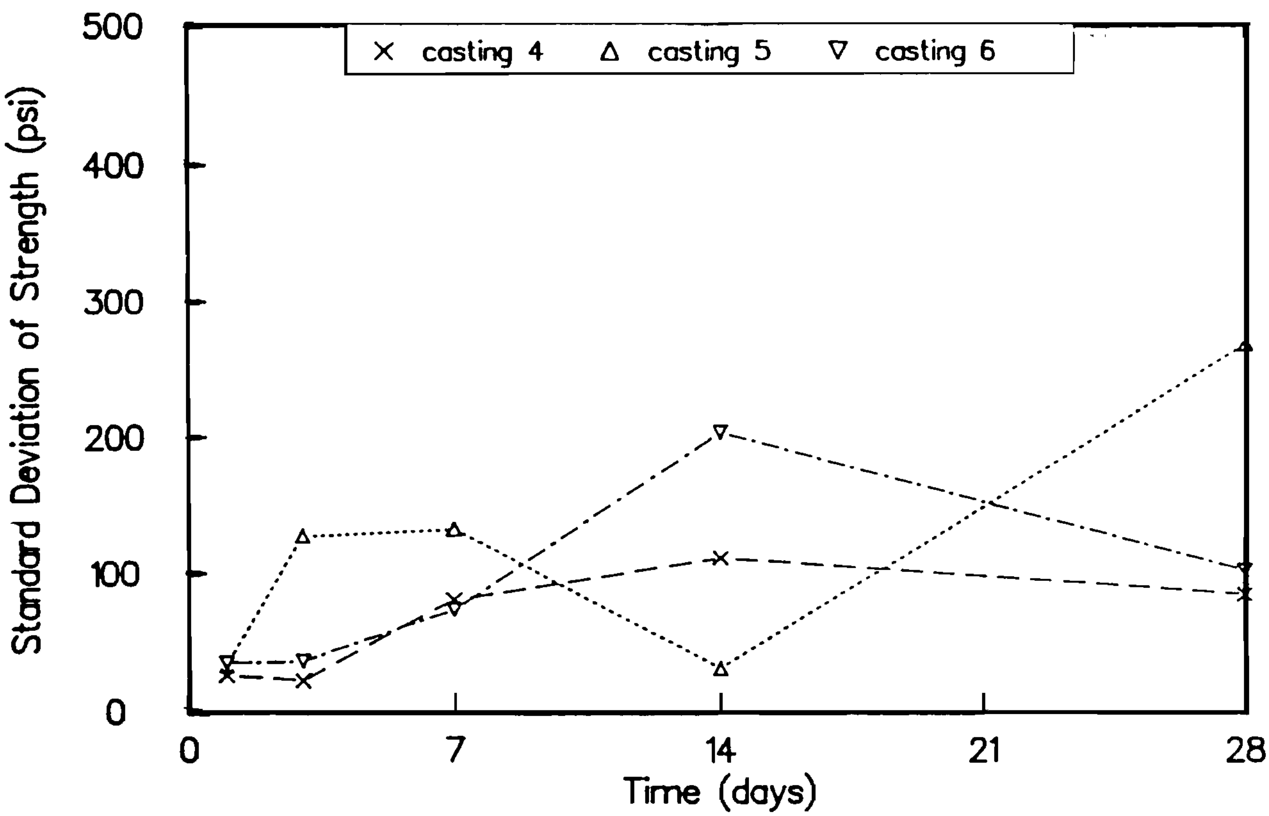


**Fig B-57. Standard deviation of wetroom-cured cylinder strengths for castings 4, 5, and 6.**

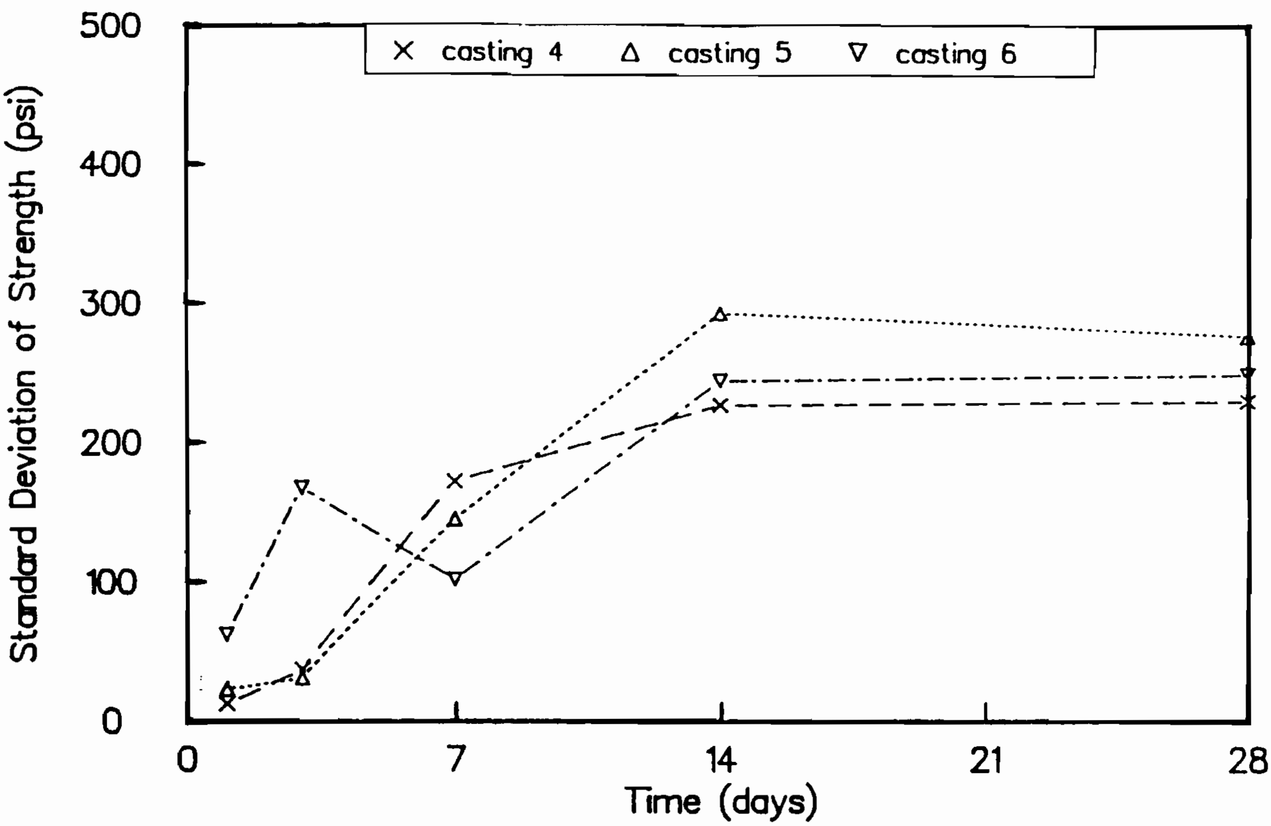


**Fig B-58. Standard deviation of outdoor-cured cylinder strengths for castings 4, 5, and 6.**

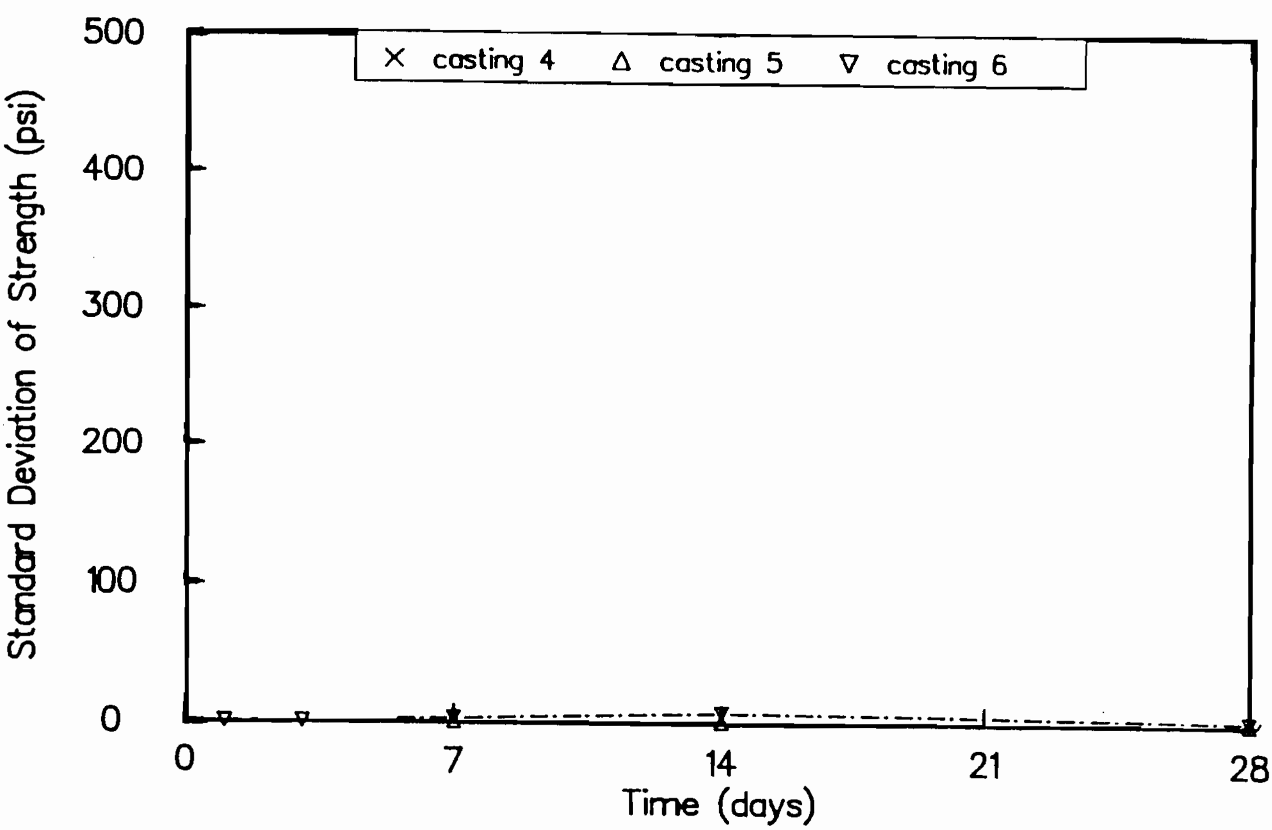


**Fig B-59. Standard deviation of cores drilled from slabs for castings 4, 5, and 6.**

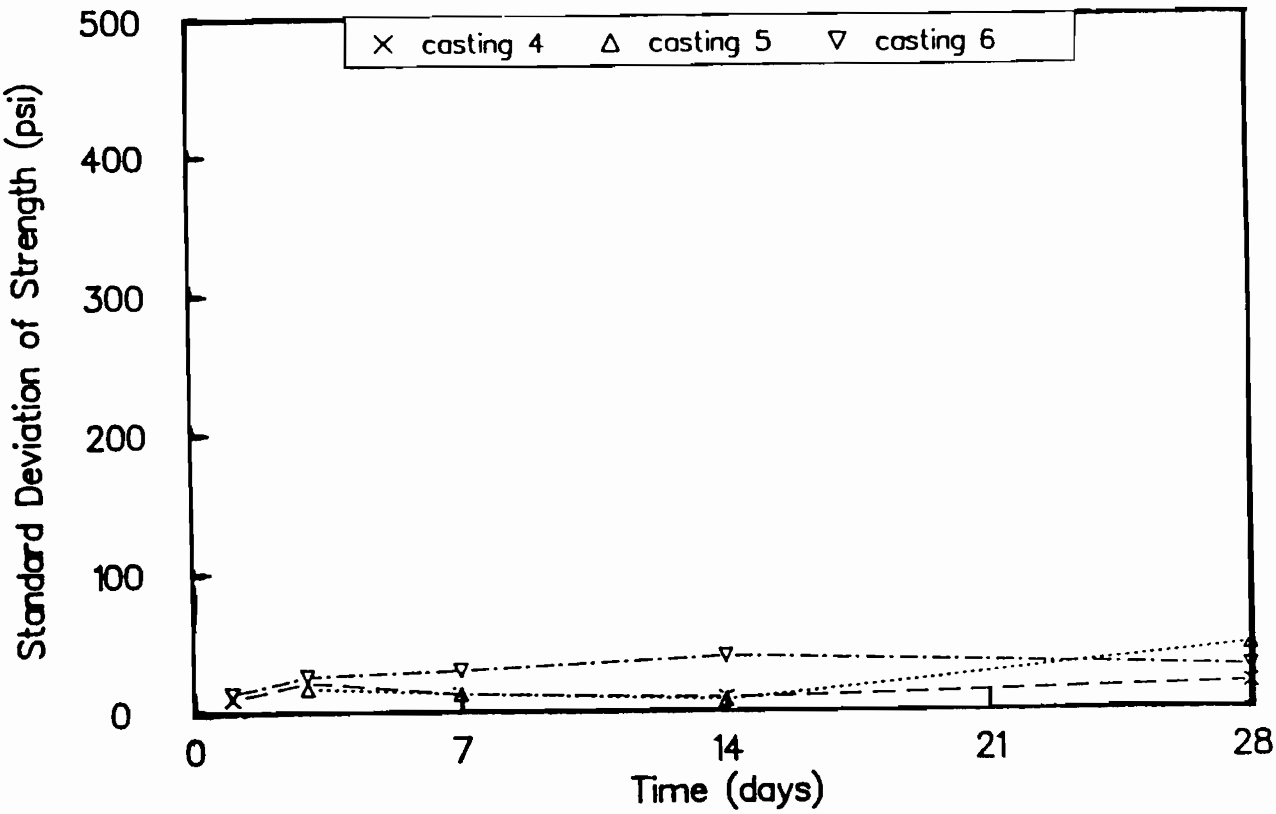


**Fig B-60. Standard deviation of flexural strength of wetroom-cured beams for castings 4, 5, and 6.**

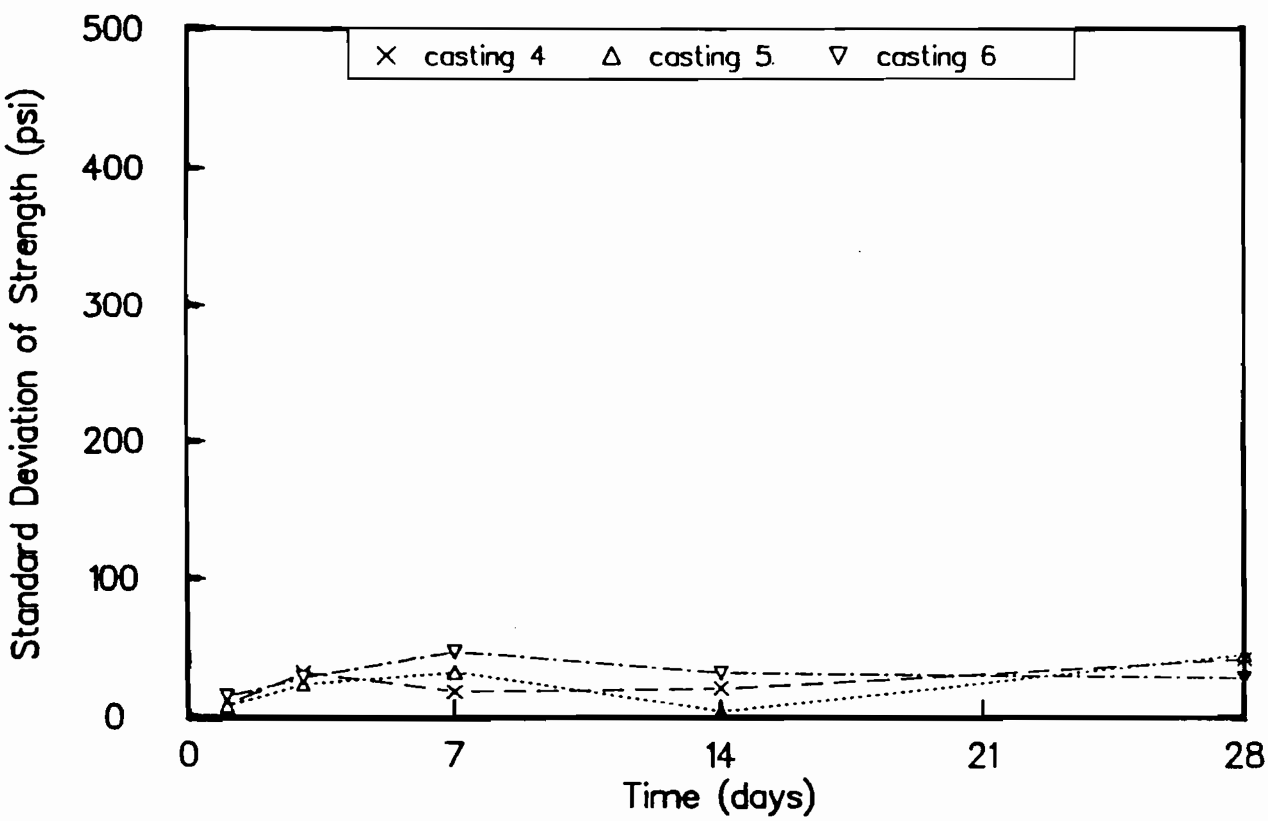


**Fig B-61. Standard deviation of flexural strength of outdoor-cured beams for castings 4, 5, and 6.**

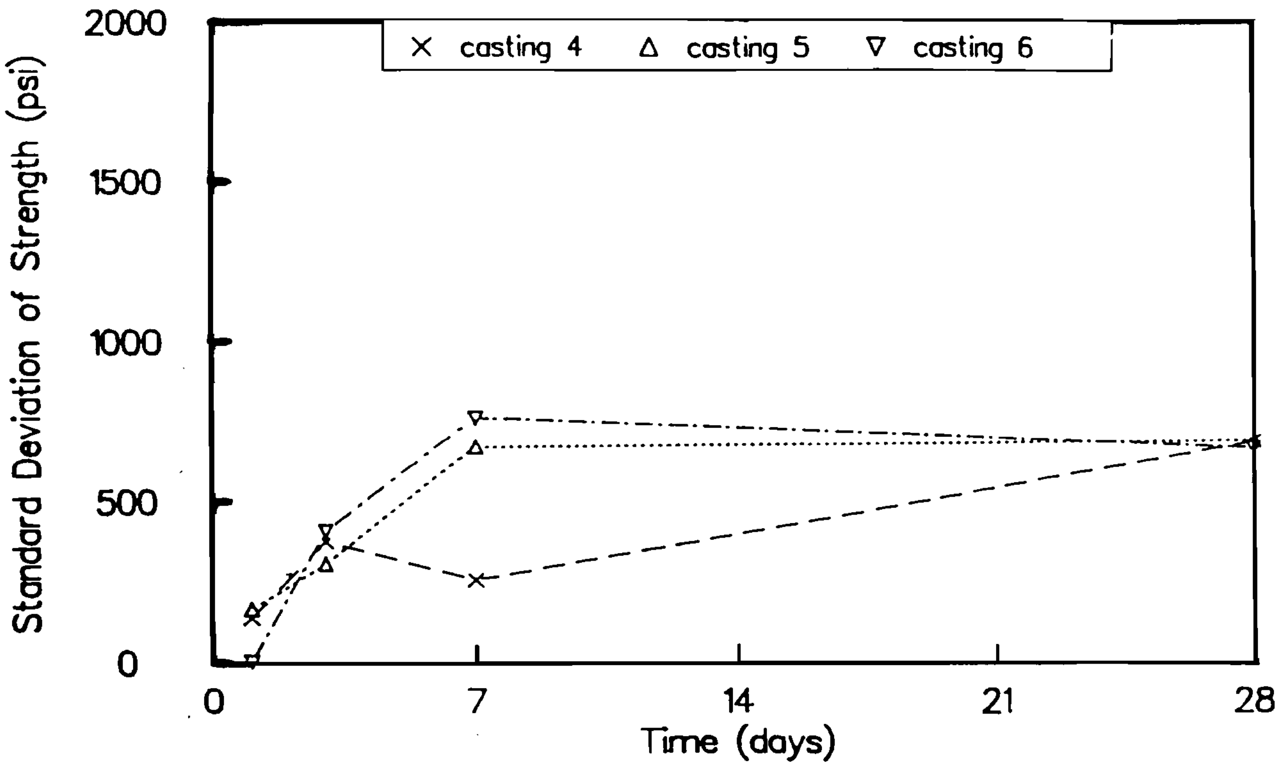


**Fig B-62. Standard deviation of "Lok-Test" pullout strengths for castings 4, 5, and 6.**

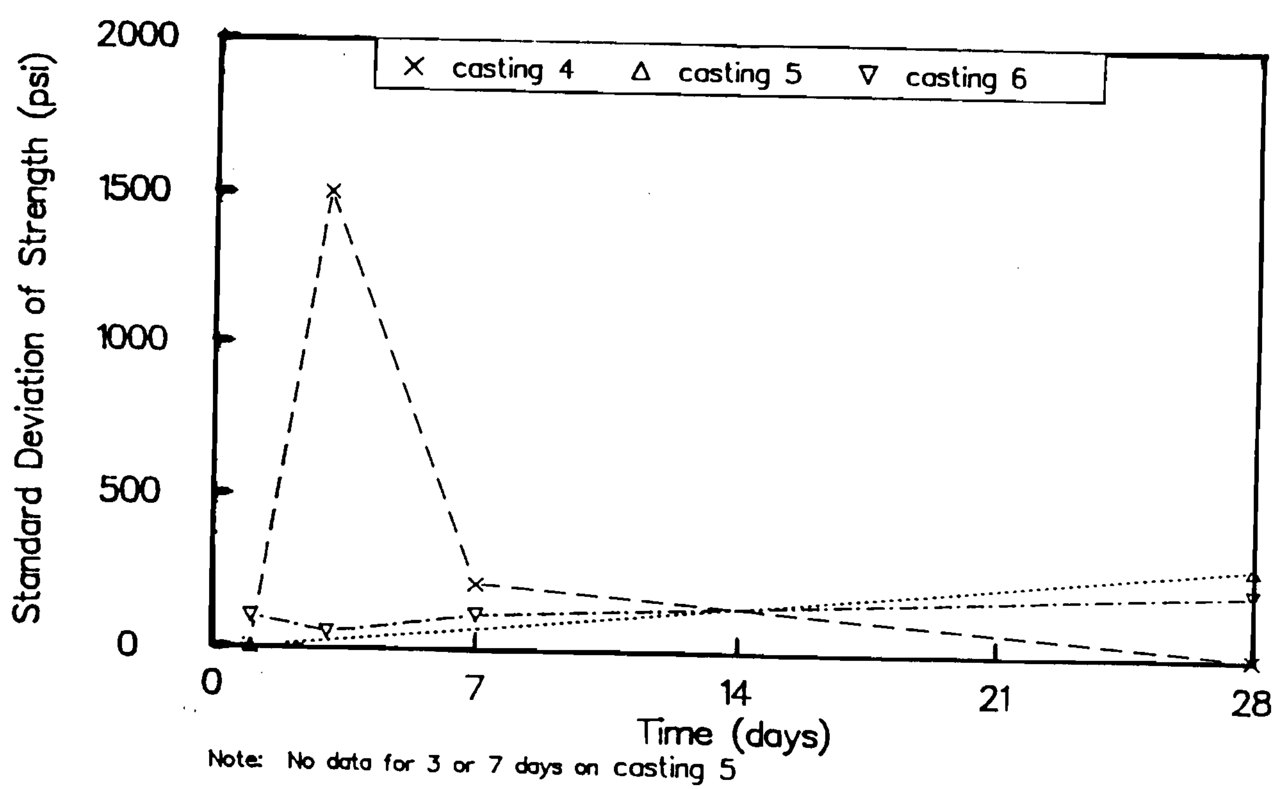


**Fig B-63. Standard deviation of Windsor probe penetration resistance strengths for castings 4, 5, and 6.**

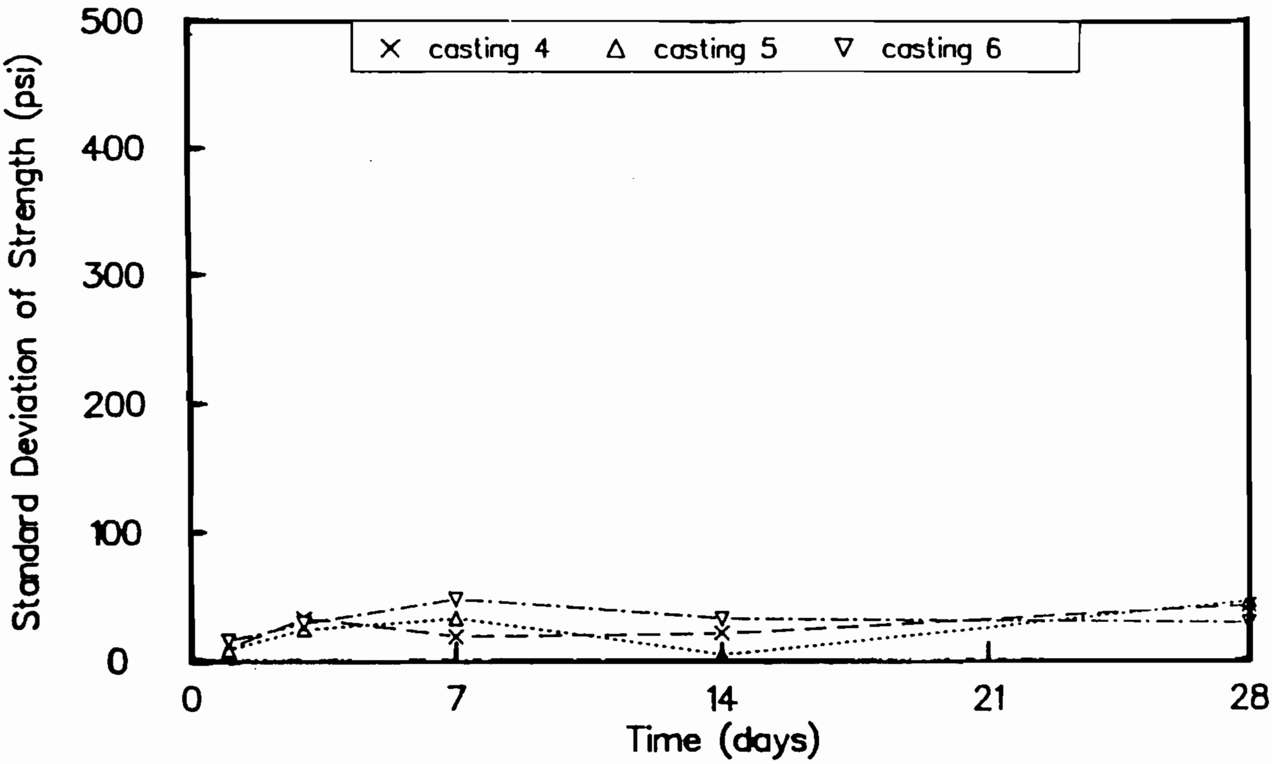


**Fig B-64. Standard deviation of maturity method strengths for castings 4, 5, and 6. Maturity curve based on wetroom-cured cylinders.**

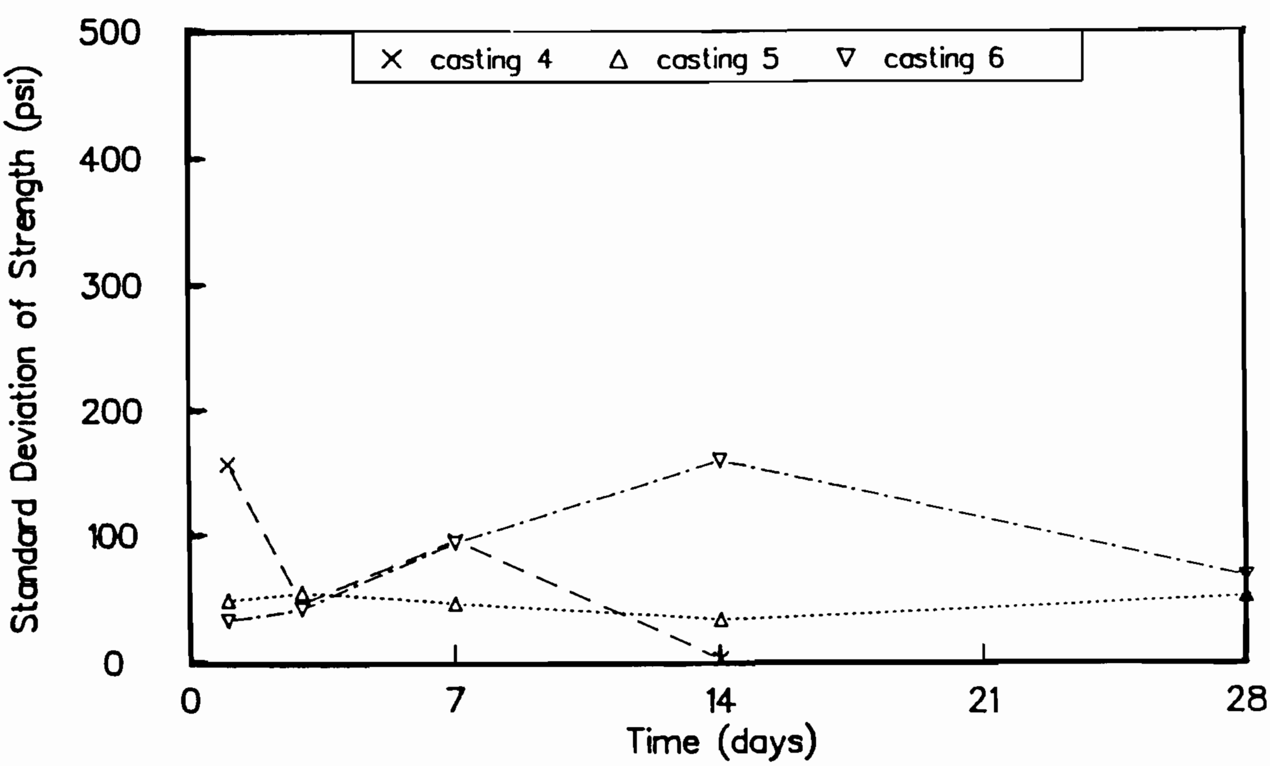


**Fig B-65. Standard deviation of maturity method strengths for castings 4, 5, and 6. Maturity curve based on outdoor-cured cylinders.**

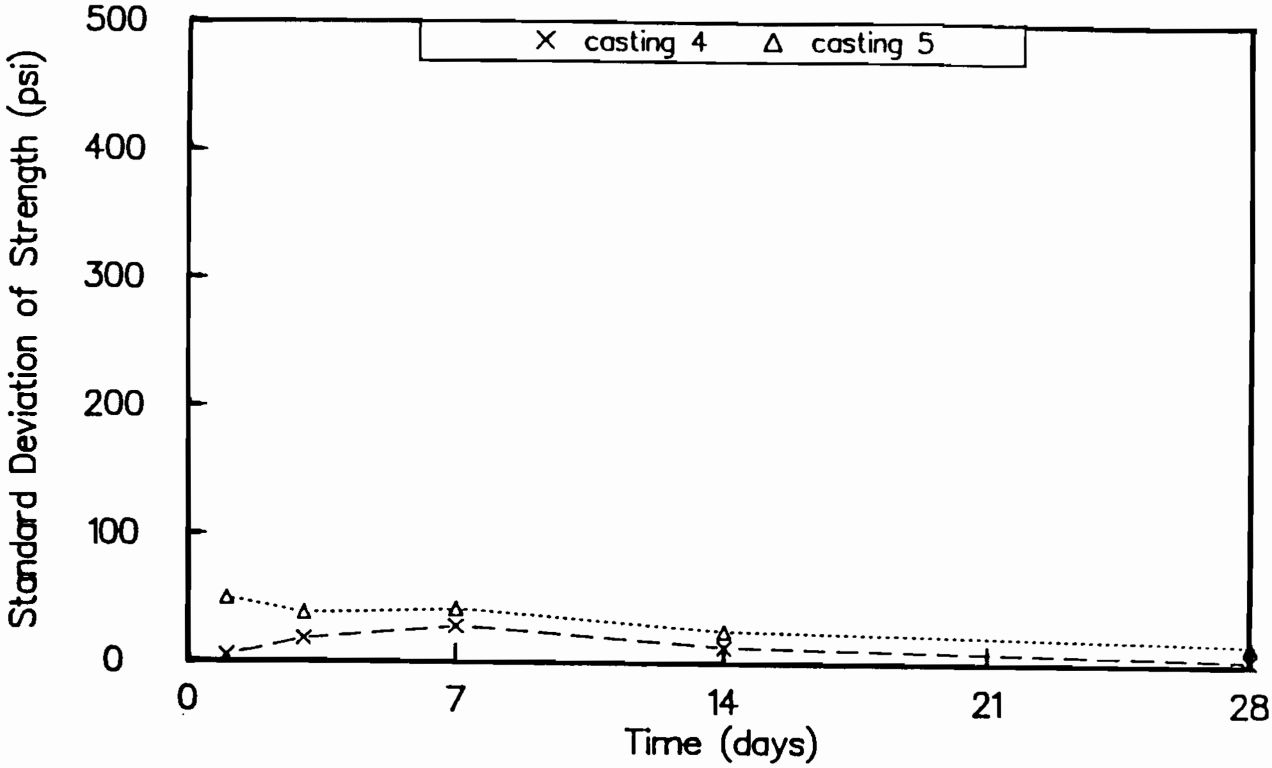


**Fig B-66. Standard deviation of maturity method strengths for castings 4 and 5. Maturity curve based on cores drilled from slabs.**

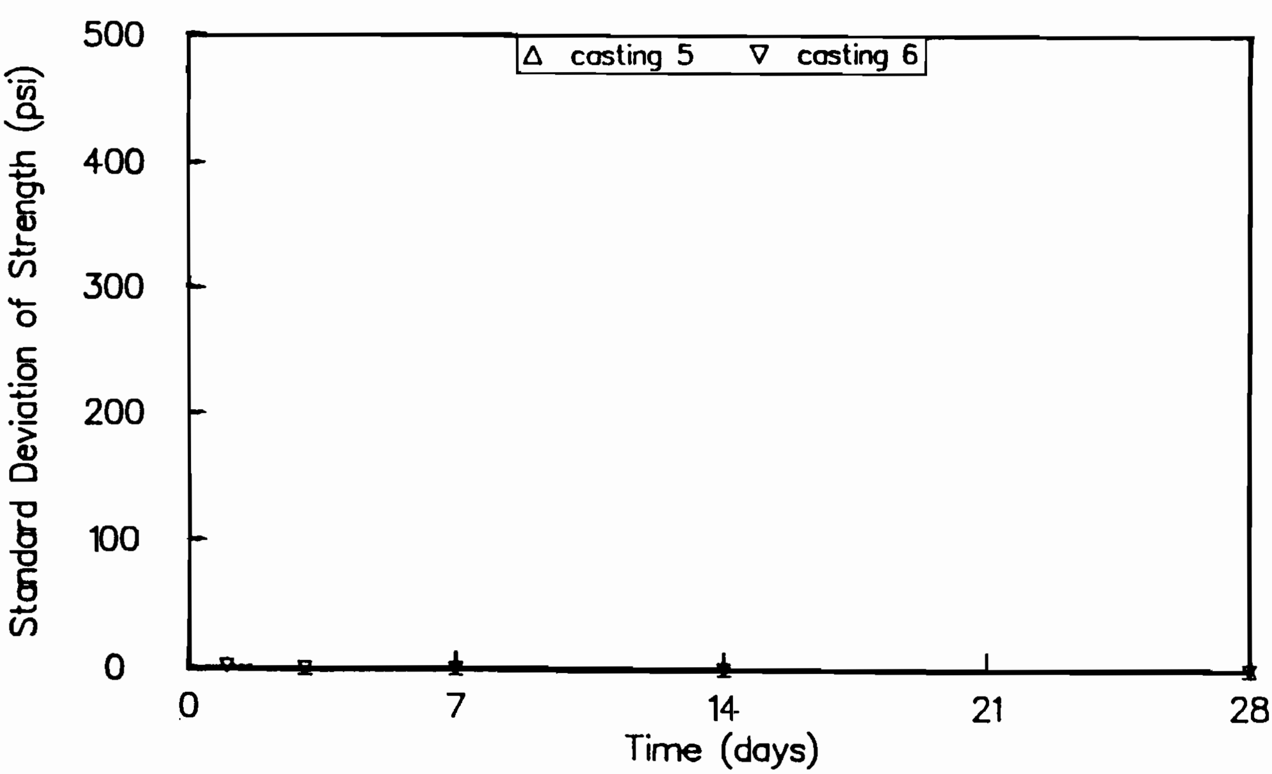


**Fig B-67. Standard deviation of maturity method strengths for castings 5 and 6. Maturity curve based on wetroom-cured beams.**

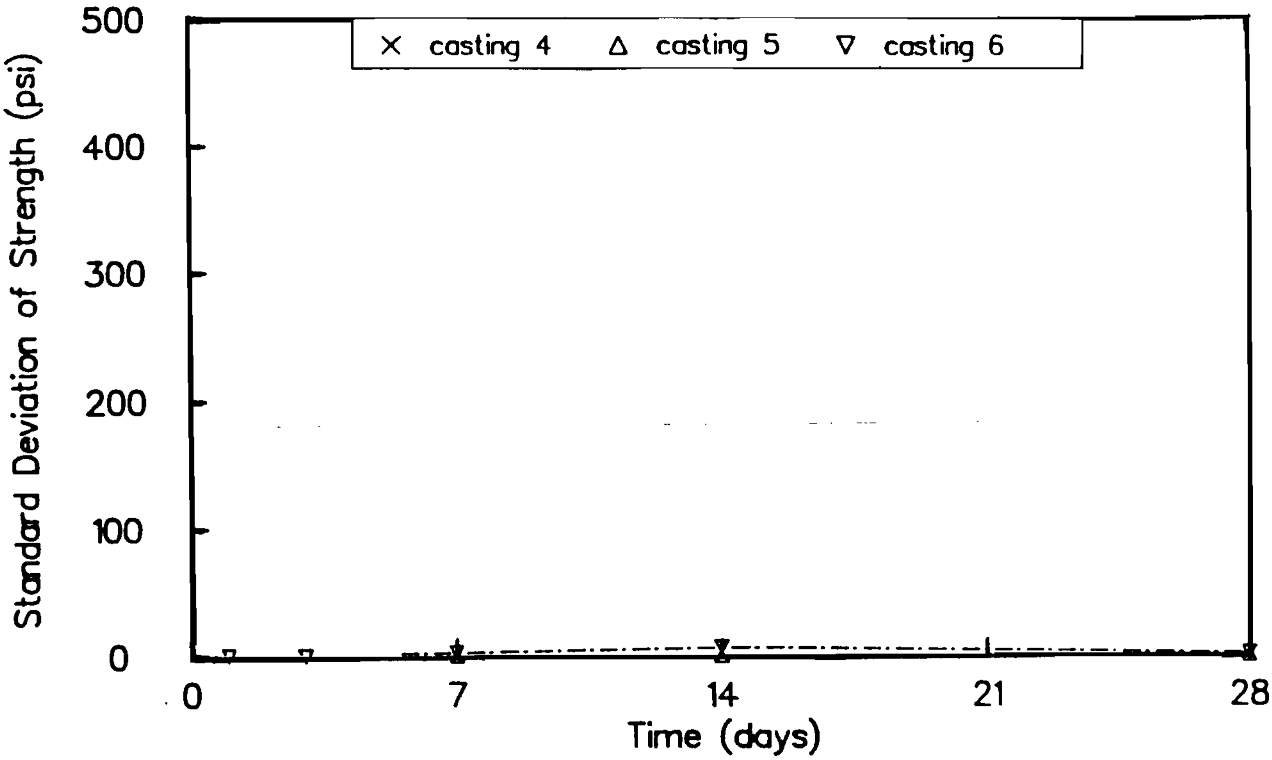


**Fig B-68. Standard deviation of maturity method strengths for castings 4, 5, and 6. Maturity curve based on outdoor-cured beams.**

# APPENDIX C

TABLE C-1. DATA FOR MATURITY CURVES OF CASTING 7

| | STRENGTH (psi) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 1764 | 4727 | 6483 | 7606 | 8119 |
| CYLINDERS cured outdoors | 1523 | 3894 | 6123 | 6945 | 7197 |
| CORES | 2398 | 4941 | 6777 | 7580 | 8004 |
| BEAMS cured in wetroom | 400 | 633 | 782 | 810 | 893 |
| BEAMS cured outdoors | 363 | 617 | 653 | 900 | 943 |

| | MATURITY (degree C-Hr) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 854 | 2159 | 5189 | 10169 | 20189 |
| CYLINDERS cured outdoors | 789 | 1981 | 4890 | 9083 | 18316 |
| CORES | 1025 | 2645 | 5930 | 10542 | 18963 |
| BEAMS cured in wetroom | 826 | 2161 | 5191 | 10141 | 20221 |
| BEAMS cured outdoors | 771 | 2019 | 4770 | 8977 | 17999 |

| | slope m (psi/log Mat) | intercept b (psi) |
|---|---|---|
| CYLINDERS cured in wetroom | 4633 | -11211 |
| CYLINDERS cured outdoors | 4329 | -10538 |
| CORES | 4522 | -10799 |
| BEAMS cured in wetroom | 255 | -201 |
| BEAMS cured outdoors | 426 | -844 |

NOTE: slope and intercept calculated using logarithmic curve fit. See chapter 5.

**TABLE C-2. DATA FOR MATURITY CURVES OF CASTING 8**

| | STRENGTH (psi) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 357 | 1305 | 1648 | 1788 | 2176 |
| CYLINDERS cured outdoors | 376 | 1146 | 1476 | 1708 | 1731 |
| CORES | --- | 1188 | 1510 | 1704 | 1993 |
| BEAMS cured in wetroom | 130 | 300 | 365 | 417 | 486 |
| BEAMS cured outdoors | 130 | 323 | 433 | 455 | 496 |

| | MATURITY (degree C-Hr) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 636 | 2046 | 5046 | 10026 | 20046 |
| CYLINDERS cured outdoors | 624 | 2149 | 4372 | 8833 | 18607 |
| CORES | --- | 2207 | 4686 | 8854 | 17723 |
| BEAMS cured in wetroom | 626 | 2034 | 4944 | 9984 | 15144 |
| BEAMS cured outdoors | 598 | 1947 | 4246 | 8626 | 18423 |

| | slope m (psi/log Mat) | intercept b (psi) |
|---|---|---|
| CYLINDERS cured in wetroom | 1144 | -2690 |
| CYLINDERS cured outdoors | 947 | -2120 |
| CORES | 874 | -1723 |
| BEAMS cured in wetroom | 203 | -377 |
| BEAMS cured outdoors | 245 | -510 |

NOTE: slope and intercept calculated using logarithmic curve fit. See chapter 5.

**TABLE C-3. DATA FOR MATURITY CURVES OF CASTING 9**

| | STRENGTH (psi) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 1324 | 3515 | 4551 | 5022 | 5549 |
| CYLINDERS cured outdoors | 1250 | 3348 | 4272 | 5982 | 5321 |
| CORES | 1443 | 3677 | 4716 | 5138 | 5430 |
| BEAMS cured in wetroom | 301 | 498 | 668 | 655 | 675 |
| BEAMS cured outdoors | 294 | 527 | 531 | 646 | 654 |

| | MATURITY (degree C-Hr) AT AGE OF | | | | |
|---|---|---|---|---|---|
| | 1 day | 3 days | 7 days | 14 days | 28 days |
| CYLINDERS cured in wetroom | 523 | 1843 | 4843 | 9823 | 19873 |
| CYLINDERS cured outdoors | 632 | 2143 | 5803 | 10972 | 23125 |
| CORES | 827 | --- | 6843 | 10901 | --- |
| BEAMS cured in wetroom | 531 | 1851 | 4851 | 9801 | 19851 |
| BEAMS cured outdoors | 663 | 2137 | 5534 | 10971 | 24528 |

| | slope m (psi/log Mat) | intercept b (psi) |
|---|---|---|
| CYLINDERS cured in wetroom | 2635 | -5473 |
| CYLINDERS cured outdoors | 2865 | -6450 |
| CORES | 3382 | -8398 |
| BEAMS cured in wetroom | 161 | 9 |
| BEAMS cured outdoors | 223 | -288 |

NOTE: slope and intercept calculated using logarithmic curve fit. See chapter 5.

NOTE: curve for cores not used because of lack of data points.

TABLE C-4. CASTINGS 7, 8, AND 9, RATIOS OF VARIOUS STRENGTHS TO FOUR STANDARDS, AT 1 DAY

| | CYLINDERS | | CORES | BEAMS | | PULLOUT | PEN. RESIST. |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors | LOK-Test | Windsor Probe |
| **CASTING 7** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.86 | 1.36 | 0.23 | 0.21 | 0.93 | 0.79 |
| ratio to strength of outdoor cylinders | 1.16 | 1.00 | 1.57 | 0.26 | 0.24 | 1.08 | 0.92 |
| ratio to strength of wetroom beams | 4.41 | 3.81 | 5.99 | 1.00 | 0.91 | 4.10 | 3.50 |
| ratio to strength of outdoor beams | 4.85 | 4.19 | 6.60 | 1.10 | 1.00 | 4.52 | 3.85 |
| **CASTING 8** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 1.05 | 0.00 | 0.37 | 0.37 | 0.33 | 0.00 |
| ratio to strength of outdoor cylinders | 0.95 | 1.00 | 0.00 | 0.35 | 0.35 | 0.31 | 0.00 |
| ratio to strength of wetroom beams | 2.74 | 2.89 | 0.00 | 1.00 | 1.00 | 0.89 | 0.01 |
| ratio to strength of outdoor beams | 2.74 | 2.89 | 0.00 | 1.00 | 1.00 | 0.89 | 0.01 |
| **CASTING 9** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.94 | 1.09 | 0.23 | 0.22 | 0.00 | 0.00 |
| ratio to strength of outdoor cylinders | 1.06 | 1.00 | 1.15 | 0.24 | 0.23 | 0.00 | 0.00 |
| ratio to strength of wetroom beams | 4.41 | 4.16 | 4.80 | 1.00 | 0.98 | 0.00 | 0.00 |
| ratio to strength of outdoor beams | 4.51 | 4.26 | 4.91 | 1.02 | 1.00 | 0.00 | 0.00 |

TABLE C-5. CASTINGS 7, 8, AND 9, RATIOS OF MATURITY STRENGTHS TO FOUR STANDARDS, AT 1 DAY

| | MATURITY STRENGTHS LISTED BY BASIS OF MATURITY CURVE | | | | |
|---|---|---|---|---|---|
| | CYLINDERS | | CORES | BEAMS | |
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors |
| **CASTING 7** | | | | | |
| ratio to strength of wetroom cylinders | 1.34 | 1.14 | 1.57 | 0.31 | 0.22 |
| ratio to strength of outdoor cylinders | 1.56 | 1.32 | 1.81 | 0.36 | 0.25 |
| ratio to strength of wetroom beams | 5.93 | 5.01 | 6.91 | 1.36 | 0.96 |
| ratio to strength of outdoor beams | 6.53 | 5.52 | 7.60 | 1.50 | 1.06 |
| **CASTING 8** | | | | | |
| ratio to strength of wetroom cylinders | 1.45 | 1.48 | 0.00 | 0.53 | 0.48 |
| ratio to strength of outdoor cylinders | 1.37 | 1.40 | 0.00 | 0.50 | 0.45 |
| ratio to strength of wetroom beams | 3.96 | 4.05 | 0.01 | 1.45 | 1.31 |
| ratio to strength of outdoor beams | 3.96 | 4.05 | 0.01 | 1.45 | 1.31 |
| **CASTING 9** | | | | | |
| ratio to strength of wetroom cylinders | 1.28 | 1.19 | 0.00 | 0.34 | 0.26 |
| ratio to strength of outdoor cylinders | 1.35 | 1.26 | 0.00 | 0.36 | 0.27 |
| ratio to strength of wetroom beams | 5.63 | 5.24 | 0.00 | 1.49 | 1.14 |
| ratio to strength of outdoor beams | 5.76 | 5.36 | 0.00 | 1.53 | 1.16 |

TABLE C-6. CASTINGS 7, 8, AND 9, RATIOS OF VARIOUS STRENGTHS TO FOUR STANDARDS, AT 3 DAYS

| | CYLINDERS | | CORES | BEAMS | | PULLOUT | PEN. RESIST. |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors | LOK-Test | Windsor Probe |
| **CASTING 7** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.82 | 1.05 | 0.13 | 0.13 | 0.90 | 0.69 |
| ratio to strength of outdoor cylinders | 1.21 | 1.00 | 1.27 | 0.16 | 0.16 | 1.09 | 0.83 |
| ratio to strength of wetroom beams | 7.46 | 6.15 | 7.80 | 1.00 | 0.97 | 6.69 | 5.13 |
| ratio to strength of outdoor beams | 7.67 | 6.32 | 8.01 | 1.03 | 1.00 | 6.87 | 5.27 |
| **CASTING 8** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.88 | 0.91 | 0.23 | 0.25 | 0.71 | 0.57 |
| ratio to strength of outdoor cylinders | 1.14 | 1.00 | 1.04 | 0.26 | 0.28 | 0.81 | 0.65 |
| ratio to strength of wetroom beams | 4.35 | 3.82 | 3.96 | 1.00 | 1.08 | 3.11 | 2.50 |
| ratio to strength of outdoor beams | 4.04 | 3.55 | 3.67 | 0.93 | 1.00 | 2.88 | 2.32 |
| **CASTING 9** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.95 | 1.05 | 0.14 | 0.15 | 0.00 | 0.00 |
| ratio to strength of outdoor cylinders | 1.05 | 1.00 | 1.10 | 0.15 | 0.16 | 0.00 | 0.00 |
| ratio to strength of wetroom beams | 7.06 | 6.73 | 7.39 | 1.00 | 1.06 | 0.00 | 0.00 |
| ratio to strength of outdoor beams | 6.67 | 6.35 | 6.98 | 0.94 | 1.00 | 0.00 | 0.00 |

TABLE C-7. CASTINGS 7, 8, AND 9, RATIOS OF MATURITY STRENGTHS TO FOUR STANDARDS, AT 3 DAYS

| | MATURITY STRENGTHS LISTED BY BASIS OF MATURITY CURVE | | | | |
|---|---|---|---|---|---|
| | CYLINDERS | | CORES | BEAMS | |
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors |
| **CASTING 7** | | | | | |
| ratio to strength of wetroom cylinders | 0.90 | 0.79 | 0.99 | 0.14 | 0.12 |
| ratio to strength of outdoor cylinders | 1.09 | 0.96 | 1.21 | 0.17 | 0.14 |
| ratio to strength of wetroom beams | 6.69 | 5.90 | 7.41 | 1.03 | 0.89 |
| ratio to strength of outdoor beams | 6.87 | 6.06 | 7.61 | 1.06 | 0.91 |
| **CASTING 8** | | | | | |
| ratio to strength of wetroom cylinders | 0.84 | 0.79 | 0.92 | 0.22 | 0.23 |
| ratio to strength of outdoor cylinders | 0.96 | 0.90 | 1.04 | 0.26 | 0.26 |
| ratio to strength of wetroom beams | 3.66 | 3.46 | 3.99 | 0.98 | 0.99 |
| ratio to strength of outdoor beams | 3.39 | 3.21 | 3.71 | 0.91 | 0.92 |
| **CASTING 9** | | | | | |
| ratio to strength of wetroom cylinders | 0.89 | 0.88 | 0.00 | 0.15 | 0.13 |
| ratio to strength of outdoor cylinders | 0.94 | 0.92 | 0.00 | 0.16 | 0.14 |
| ratio to strength of wetroom beams | 6.29 | 6.21 | 0.00 | 1.08 | 0.92 |
| ratio to strength of outdoor beams | 5.94 | 5.87 | 0.00 | 1.02 | 0.86 |

TABLE C-8. CASTINGS 7, 8, AND 9, RATIOS OF VARIOUS STRENGTHS TO FOUR STANDARDS, AT 7 DAYS

| | CYLINDERS | | CORES | BEAMS | | PULLOUT | PEN. RESIST. |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors | LOK-Test | Windsor Probe |
| **CASTING 7** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.94 | 1.05 | 0.12 | 0.10 | 0.87 | 0.56 |
| ratio to strength of outdoor cylinders | 1.06 | 1.00 | 1.11 | 0.13 | 0.11 | 0.93 | 0.60 |
| ratio to strength of wetroom beams | 8.29 | 7.83 | 8.66 | 1.00 | 0.83 | 7.25 | 4.67 |
| ratio to strength of outdoor beams | 9.93 | 9.38 | 10.38 | 1.20 | 1.00 | 8.68 | 5.59 |
| **CASTING 8** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.90 | 0.92 | 0.22 | 0.26 | 0.62 | 0.49 |
| ratio to strength of outdoor cylinders | 1.12 | 1.00 | 1.02 | 0.25 | 0.29 | 0.70 | 0.54 |
| ratio to strength of wetroom beams | 4.52 | 4.04 | 4.14 | 1.00 | 1.19 | 2.82 | 2.19 |
| ratio to strength of outdoor beams | 3.80 | 3.41 | 3.48 | 0.84 | 1.00 | 2.38 | 1.85 |
| **CASTING 9** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.94 | 1.04 | 0.15 | 0.12 | 0.00 | 0.00 |
| ratio to strength of outdoor cylinders | 1.07 | 1.00 | 1.10 | 0.16 | 0.12 | 0.00 | 0.00 |
| ratio to strength of wetroom beams | 6.82 | 6.40 | 7.06 | 1.00 | 0.80 | 0.00 | 0.00 |
| ratio to strength of outdoor beams | 8.57 | 8.04 | 8.88 | 1.26 | 1.00 | 0.00 | 0.00 |

TABLE C-9. CASTINGS 7, 8, AND 9, RATIOS OF MATURITY STRENGTHS TO FOUR STANDARDS, AT 7 DAYS

| | MATURITY STRENGTHS LISTED BY BASIS OF MATURITY CURVE | | | | |
|---|---|---|---|---|---|
| | CYLINDERS | | CORES | BEAMS | |
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors |
| **CASTING 7** | | | | | |
| ratio to strength of wetroom cylinders | 0.93 | 0.84 | 0.96 | 0.12 | 0.11 |
| ratio to strength of outdoor cylinders | 0.98 | 0.89 | 1.02 | 0.12 | 0.12 |
| ratio to strength of wetroom beams | 7.67 | 6.95 | 7.98 | 0.96 | 0.92 |
| ratio to strength of outdoor beams | 9.19 | 8.32 | 9.56 | 1.15 | 1.11 |
| **CASTING 8** | | | | | |
| ratio to strength of wetroom cylinders | 0.94 | 0.81 | 0.90 | 0.23 | 0.23 |
| ratio to strength of outdoor cylinders | 1.05 | 0.90 | 1.01 | 0.25 | 0.26 |
| ratio to strength of wetroom beams | 4.23 | 3.64 | 4.06 | 1.02 | 1.04 |
| ratio to strength of outdoor beams | 3.57 | 3.07 | 3.42 | 0.86 | 0.88 |
| **CASTING 9** | | | | | |
| ratio to strength of wetroom cylinders | 0.93 | 0.95 | 0.00 | 0.13 | 0.12 |
| ratio to strength of outdoor cylinders | 0.99 | 1.01 | 0.00 | 0.14 | 0.13 |
| ratio to strength of wetroom beams | 6.35 | 6.49 | 0.00 | 0.90 | 0.82 |
| ratio to strength of outdoor beams | 7.98 | 8.16 | 0.00 | 1.14 | 1.03 |

TABLE C-10. CASTINGS 7, 8, AND 9, RATIOS OF VARIOUS STRENGTHS TO FOUR STANDARDS, AT 14 DAYS

| | CYLINDERS | | CORES | BEAMS | | PULLOUT | PEN. RESIST. |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors | LOK-Test | Windsor Probe |
| CASTING 7 | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.91 | 1.00 | 0.11 | 0.12 | 0.00 | 0.00 |
| ratio to strength of outdoor cylinders | 1.10 | 1.00 | 1.09 | 0.12 | 0.13 | 0.00 | 0.00 |
| ratio to strength of wetroom beams | 9.39 | 8.57 | 9.36 | 1.00 | 1.11 | 0.00 | 0.00 |
| ratio to strength of outdoor beams | 8.45 | 7.72 | 8.42 | 0.90 | 1.00 | 0.00 | 0.00 |
| CASTING 8 | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.96 | 0.95 | 0.23 | 0.25 | 0.00 | 0.00 |
| ratio to strength of outdoor cylinders | 1.05 | 1.00 | 1.00 | 0.24 | 0.27 | 0.00 | 0.00 |
| ratio to strength of wetroom beams | 4.29 | 4.10 | 4.09 | 1.00 | 1.09 | 0.00 | 0.00 |
| ratio to strength of outdoor beams | 3.93 | 3.75 | 3.74 | 0.92 | 1.00 | 0.00 | 0.00 |
| CASTING 9 | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 1.19 | 1.02 | 0.13 | 0.13 | 0.00 | 0.00 |
| ratio to strength of outdoor cylinders | 0.84 | 1.00 | 0.86 | 0.11 | 0.11 | 0.00 | 0.00 |
| ratio to strength of wetroom beams | 7.67 | 9.13 | 7.84 | 1.00 | 0.99 | 0.00 | 0.00 |
| ratio to strength of outdoor beams | 7.77 | 9.26 | 7.95 | 1.01 | 1.00 | 0.00 | 0.00 |

TABLE C-11. CASTINGS 7, 8, AND 9, RATIOS OF MATURITY STRENGTHS TO FOUR STANDARDS, AT 14 DAYS

| | MATURITY STRENGTHS LISTED BY BASIS OF MATURITY CURVE | | | | |
|---|---|---|---|---|---|
| | CYLINDERS | | CORES | BEAMS | |
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors |
| CASTING 7 | | | | | |
| ratio to strength of wetroom cylinders | 0.97 | 0.87 | 0.96 | 0.11 | 0.11 |
| ratio to strength of outdoor cylinders | 1.06 | 0.95 | 1.06 | 0.12 | 0.12 |
| ratio to strength of wetroom beams | 9.08 | 8.14 | 9.05 | 1.01 | 1.04 |
| ratio to strength of outdoor beams | 8.17 | 7.33 | 8.15 | 0.91 | 0.93 |
| CASTING 8 | | | | | |
| ratio to strength of wetroom cylinders | 1.05 | 0.90 | 0.96 | 0.24 | 0.25 |
| ratio to strength of outdoor cylinders | 1.10 | 0.95 | 1.01 | 0.25 | 0.27 |
| ratio to strength of wetroom beams | 4.53 | 3.88 | 4.14 | 1.04 | 1.09 |
| ratio to strength of outdoor beams | 4.15 | 3.56 | 3.79 | 0.95 | 1.00 |
| CASTING 9 | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 1.02 | 0.00 | 0.13 | 0.12 |
| ratio to strength of outdoor cylinders | 0.84 | 0.86 | 0.00 | 0.11 | 0.10 |
| ratio to strength of wetroom beams | 7.71 | 7.82 | 0.00 | 1.00 | 0.94 |
| ratio to strength of outdoor beams | 7.81 | 7.93 | 0.00 | 1.01 | 0.95 |

**TABLE C-12. CASTINGS 7, 8, AND 9, RATIOS OF VARIOUS STRENGTHS TO FOUR STANDARDS, AT 28 DAYS**

| | CYLINDERS | | CORES | BEAMS | | PULLOUT | PEN. RESIST. |
|---|---|---|---|---|---|---|---|
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors | LOK-Test | Windsor Probe |
| **CASTING 7** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.89 | 0.99 | 0.11 | 0.12 | 0.88 | 0.54 |
| ratio to strength of outdoor cylinders | 1.13 | 1.00 | 1.11 | 0.12 | 0.13 | 1.00 | 0.61 |
| ratio to strength of wetroom beams | 9.09 | 8.06 | 8.96 | 1.00 | 1.06 | 8.02 | 4.93 |
| ratio to strength of outdoor beams | 8.61 | 7.63 | 8.49 | 0.95 | 1.00 | 7.59 | 4.66 |
| **CASTING 8** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.80 | 0.92 | 0.22 | 0.23 | 0.86 | 0.00 |
| ratio to strength of outdoor cylinders | 1.26 | 1.00 | 1.15 | 0.28 | 0.29 | 1.08 | 0.00 |
| ratio to strength of wetroom beams | 4.48 | 3.57 | 4.10 | 1.00 | 1.02 | 3.84 | 0.00 |
| ratio to strength of outdoor beams | 4.39 | 3.49 | 4.02 | 0.98 | 1.00 | 3.76 | 0.00 |
| **CASTING 9** | | | | | | | |
| ratio to strength of wetroom cylinders | 1.00 | 0.96 | 0.98 | 0.12 | 0.12 | 0.00 | 0.00 |
| ratio to strength of outdoor cylinders | 1.04 | 1.00 | 1.02 | 0.13 | 0.12 | 0.00 | 0.00 |
| ratio to strength of wetroom beams | 8.22 | 7.88 | 8.04 | 1.00 | 0.97 | 0.00 | 0.00 |
| ratio to strength of outdoor beams | 8.48 | 8.13 | 8.30 | 1.03 | 1.00 | 0.00 | 0.00 |

TABLE C-13. CASTINGS 7, 8, AND 9, RATIOS OF MATURITY STRENGTHS TO FOUR STANDARDS AT 28 DAYS

| | MATURITY STRENGTHS LISTED BY BASIS OF MATURITY CURVE | | | | |
|---|---|---|---|---|---|
| | CYLINDERS | | CORES | BEAMS | |
| | Cured in wetroom | Cured outdoors | | Cured in wetroom | Cured outdoors |
| **CASTING 7** | | | | | |
| ratio to strength of wetroom cylinders | 1.08 | 0.97 | 1.04 | 0.11 | 0.12 |
| ratio to strength of outdoor cylinders | 1.21 | 1.10 | 1.17 | 0.12 | 0.13 |
| ratio to strength of wetroom beams | 9.78 | 8.86 | 9.42 | 1.01 | 1.08 |
| ratio to strength of outdoor beams | 9.26 | 8.39 | 8.92 | 0.95 | 1.03 |
| **CASTING 8** | | | | | |
| ratio to strength of wetroom cylinders | 1.02 | 0.88 | 0.91 | 0.22 | 0.25 |
| ratio to strength of outdoor cylinders | 1.29 | 1.11 | 1.15 | 0.27 | 0.31 |
| ratio to strength of wetroom beams | 4.59 | 3.96 | 4.09 | 0.97 | 1.10 |
| ratio to strength of outdoor beams | 4.50 | 3.88 | 4.01 | 0.95 | 1.08 |
| **CASTING 9** | | | | | |
| ratio to strength of wetroom cylinders | 1.05 | 1.09 | 0.00 | 0.13 | 0.12 |
| ratio to strength of outdoor cylinders | 1.10 | 1.14 | 0.00 | 0.13 | 0.13 |
| ratio to strength of wetroom beams | 8.67 | 8.95 | 0.00 | 1.04 | 1.02 |
| ratio to strength of outdoor beams | 8.95 | 9.24 | 0.00 | 1.07 | 1.06 |

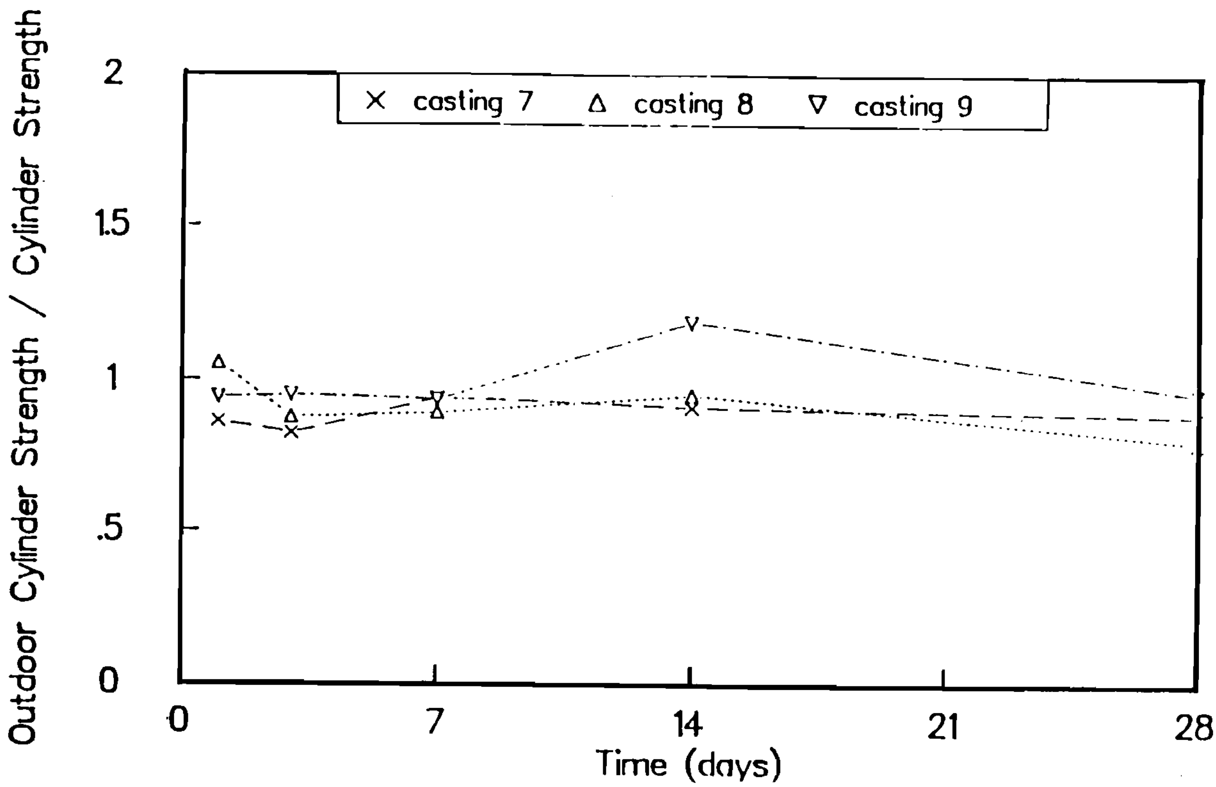


**Fig C-1. Ratio of strengths of outdoor-cured cylinders to strengths of wetroom-cured cylinders for castings 7, 8, and 9.**

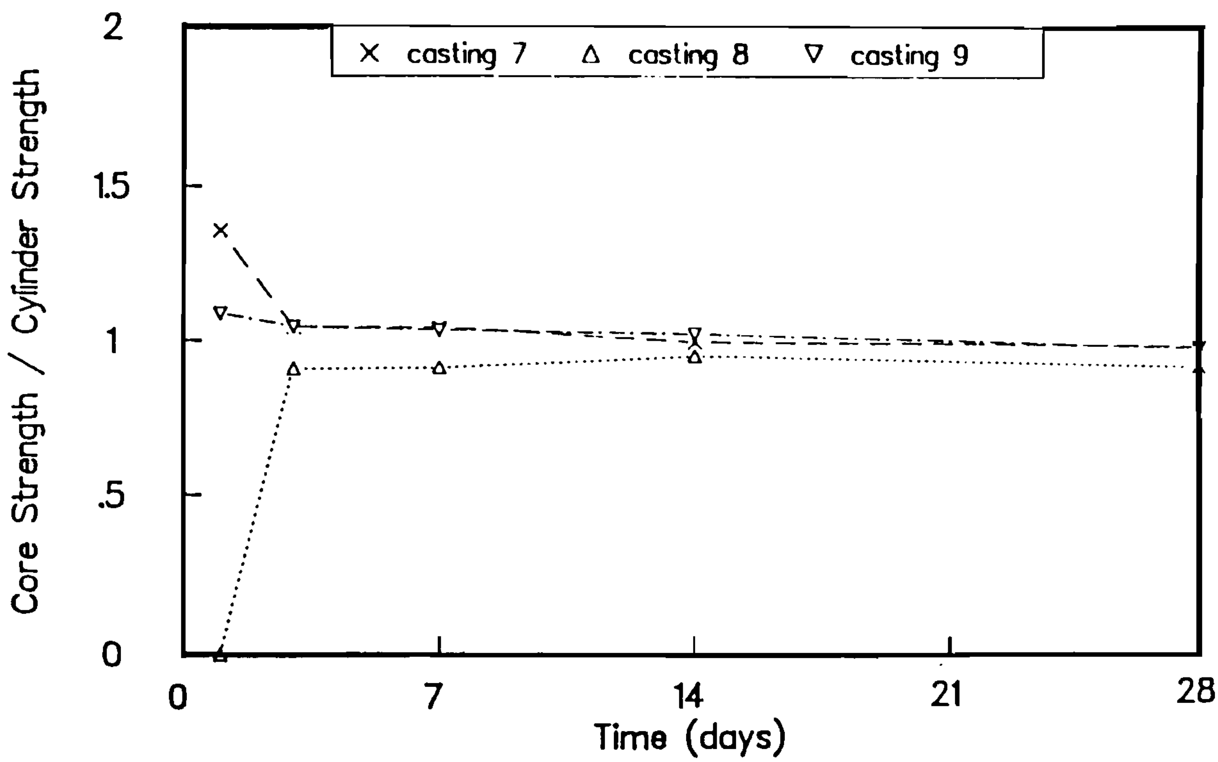


**Fig C-2. Ratio of strengths of cores to strengths of wetroom-cured cylinders for castings 7, 8, and 9.**

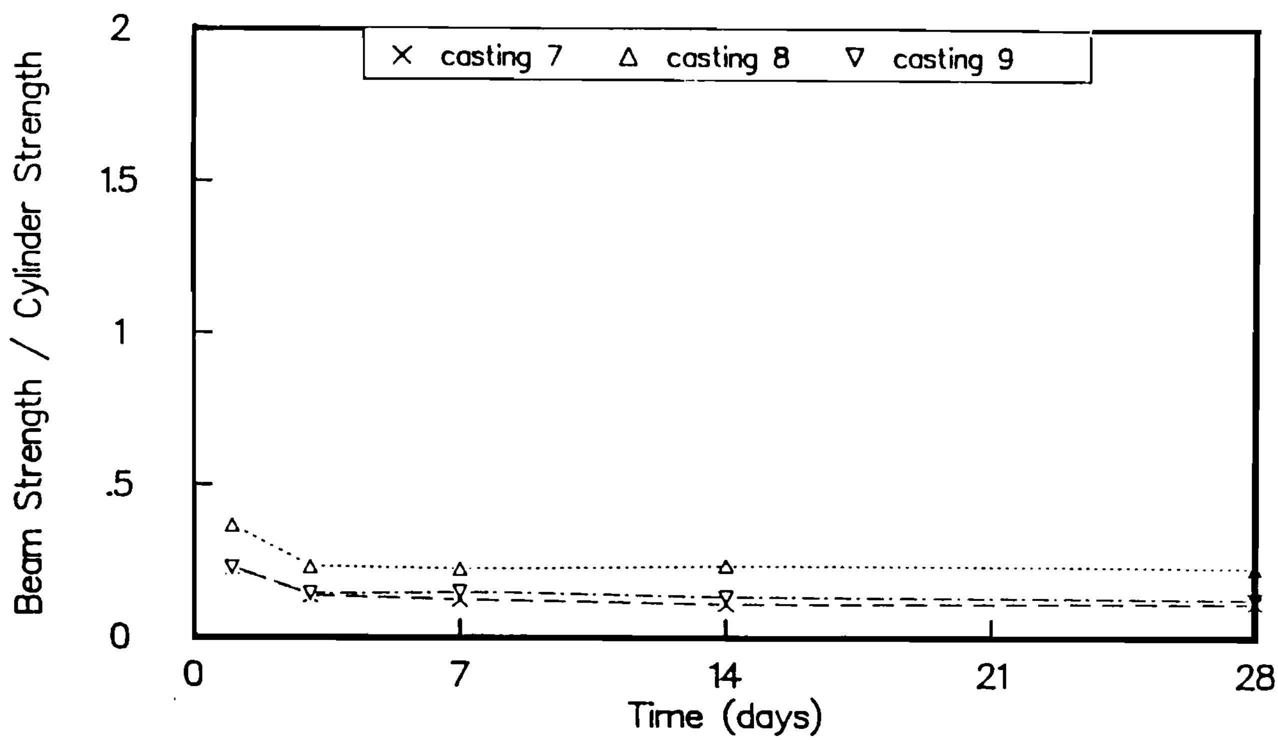


**Fig C-3. Ratio of flexural strengths of wetroom-cured beams to strengths of wetroom-cured cylinders for castings 7, 8, and 9.**

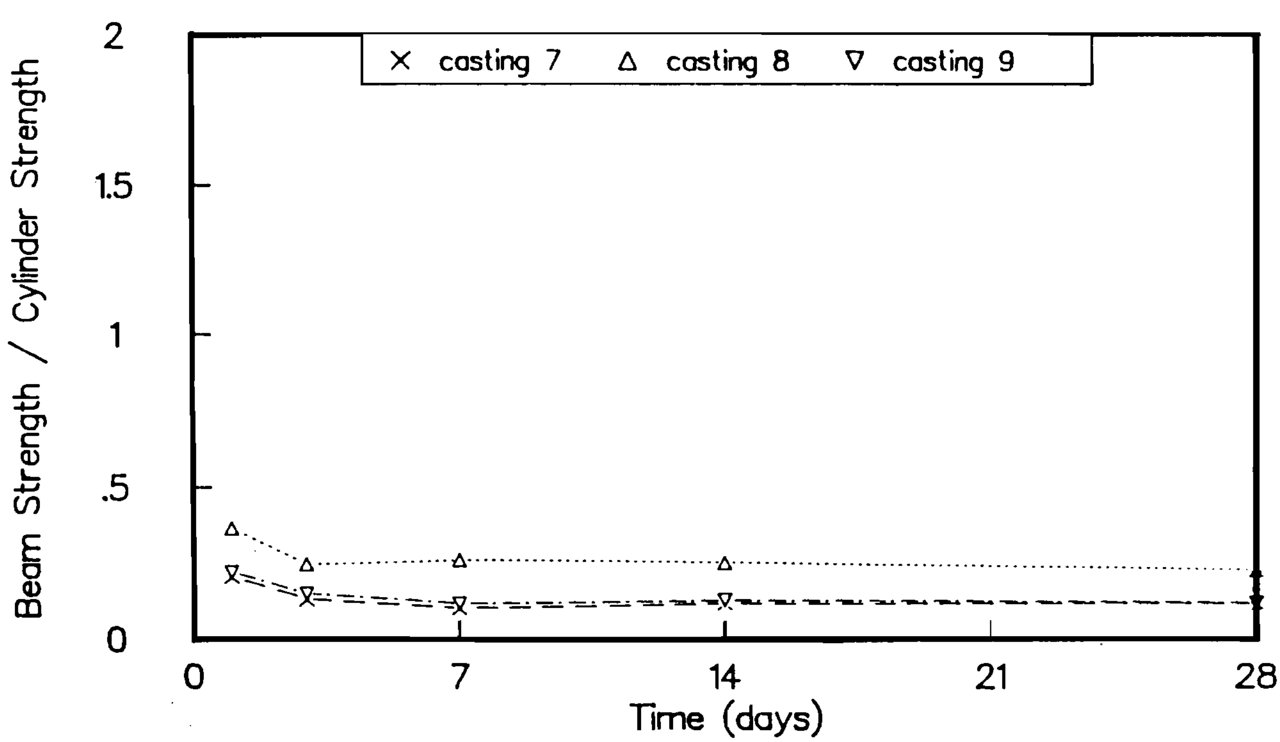


**Fig C-4. Ratio of flexural strengths of outdoor-cured beams to strengths of wetroom-cured cylinders for castings 7, 8, and 9.**

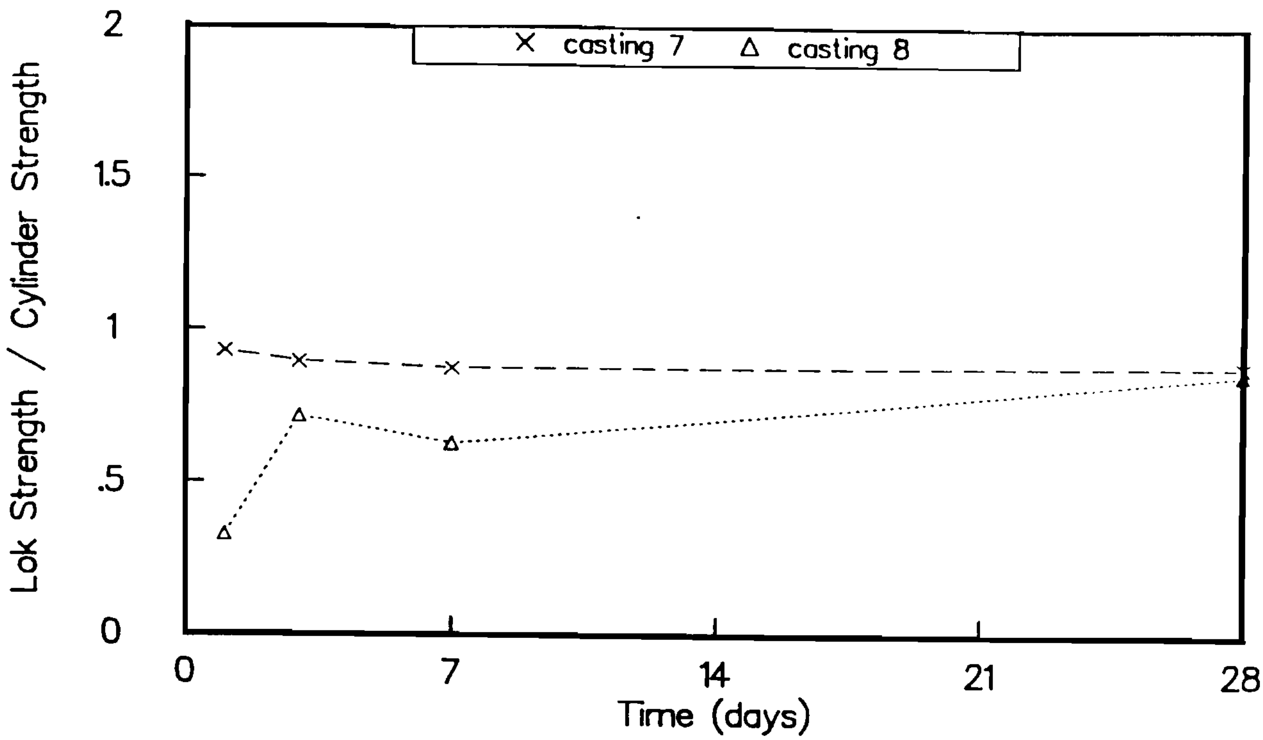


**Fig C-5. Ratio of "Lok-Test" pullout strengths to strengths of wetroom-cured cylinders for castings 7, 8, and 9.**

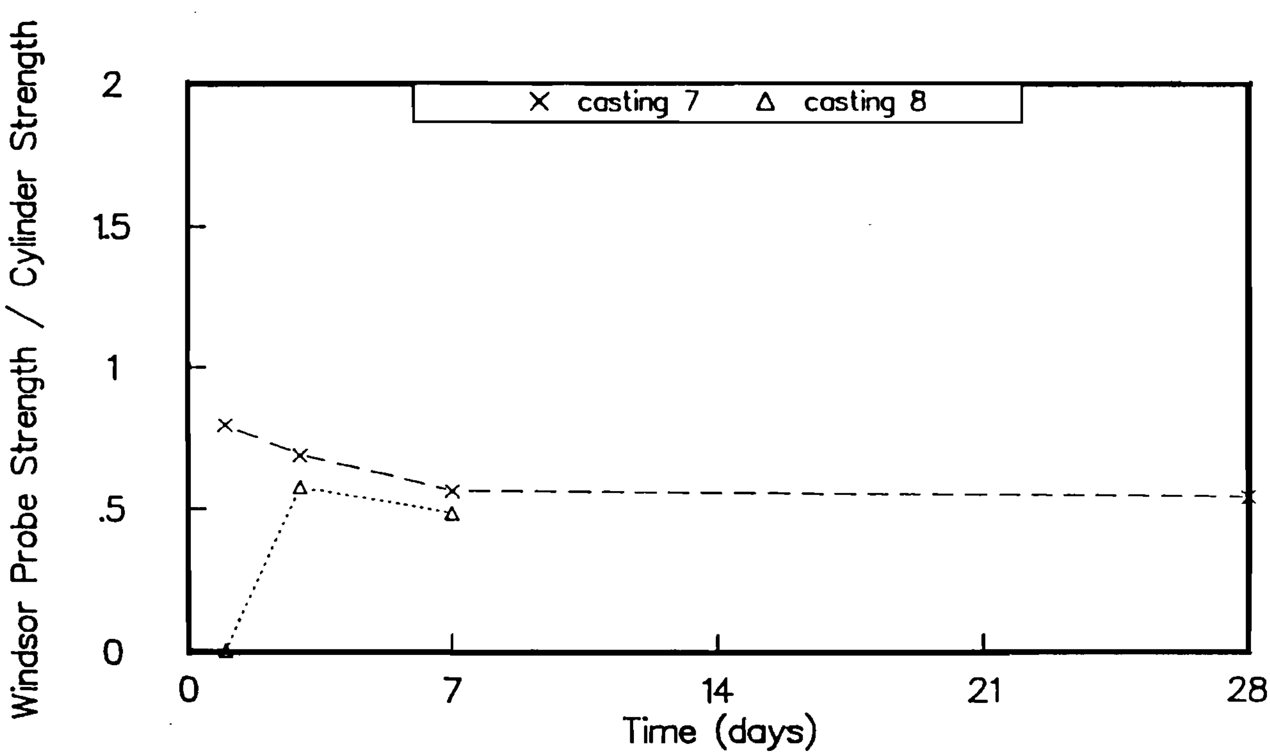


**Fig C-6. Ratio of Windsor probe penetration resistance strengths to strengths of wetroom-cured cylinders for castings 7 and 8.**

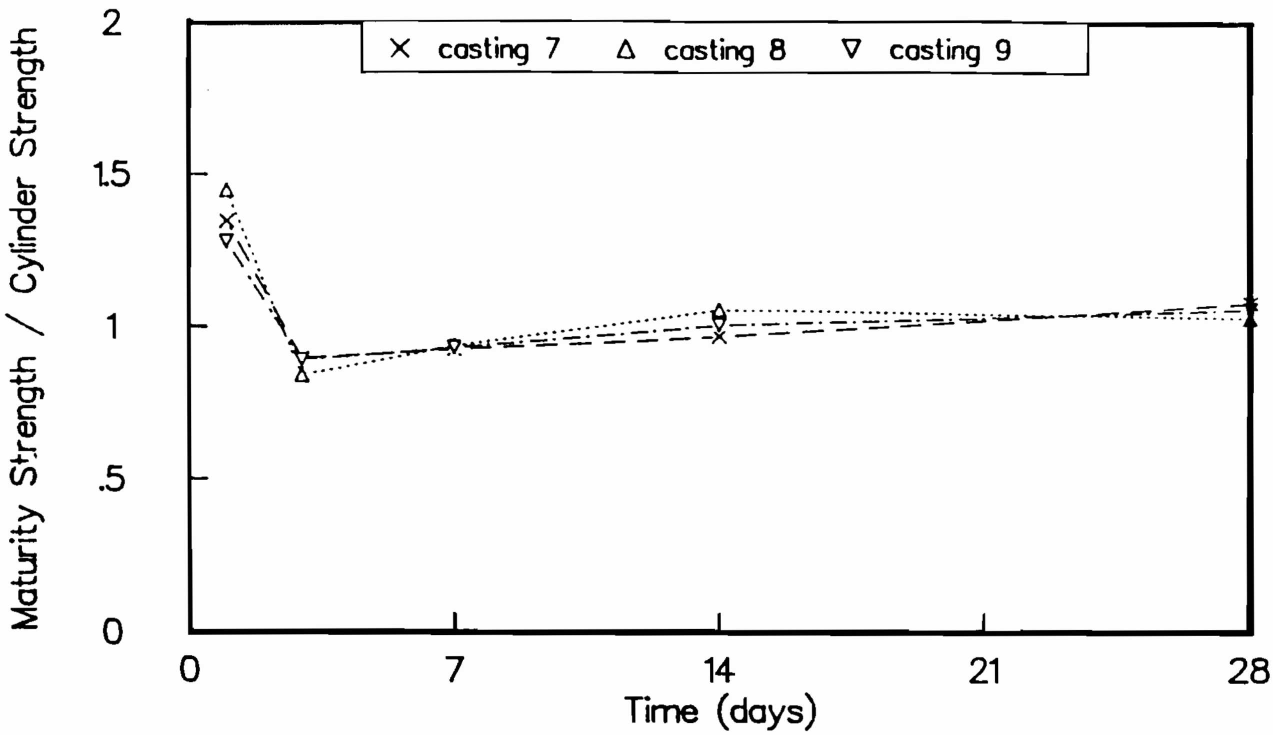


**Fig C-7. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 7, 8, and 9. Maturity curve based on wetroom-cured cylinders.**

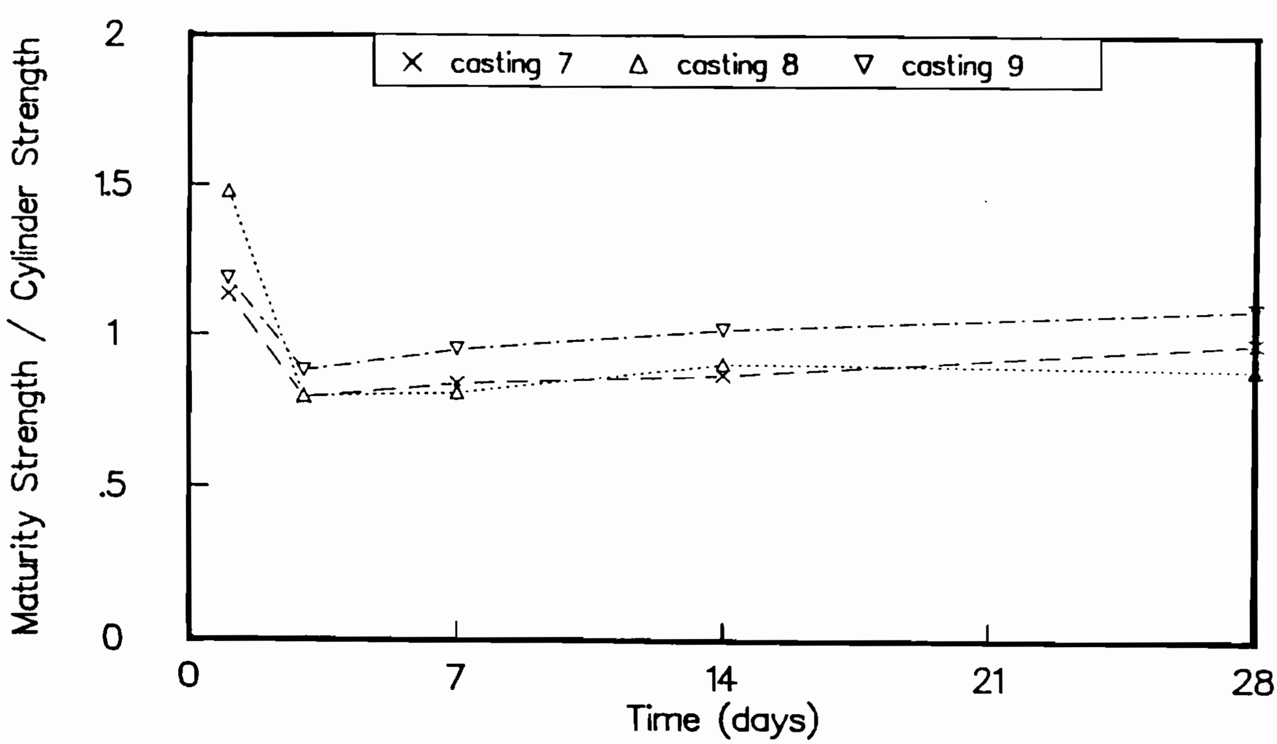


**Fig C-8. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 7, 8, and 9. Maturity curve based on outdoor-cured cylinders.**

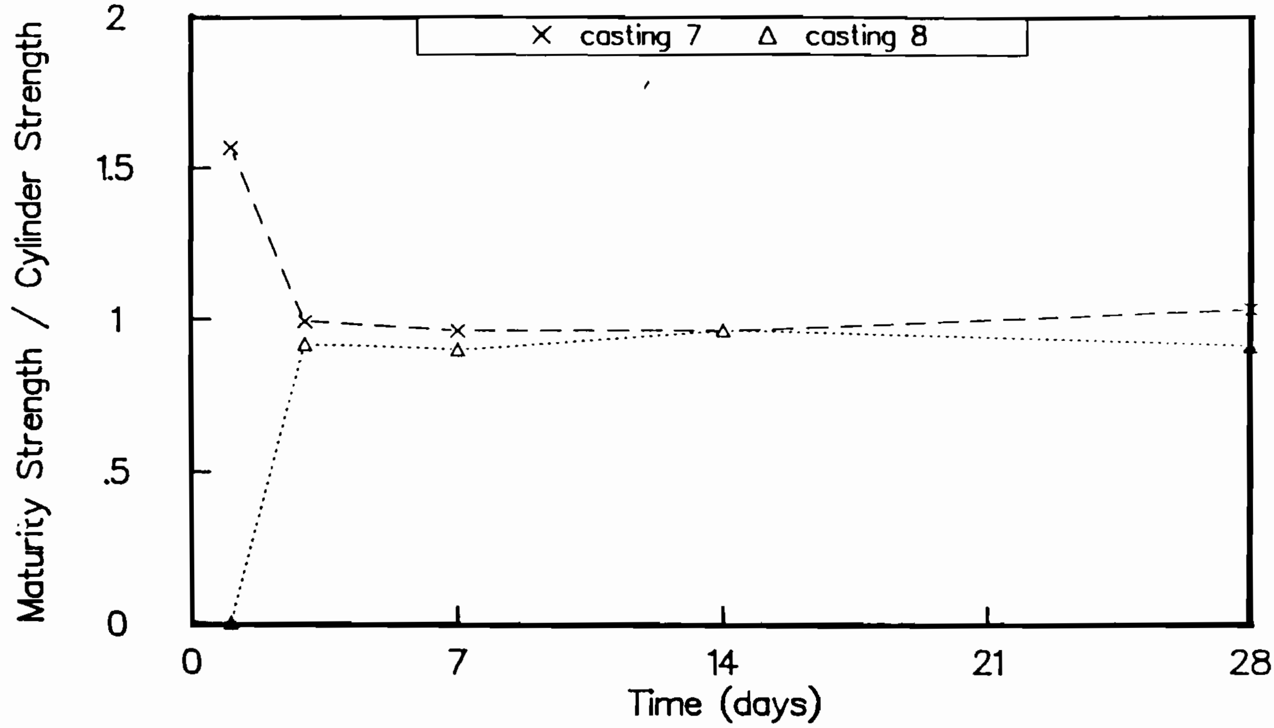


**Fig C-9. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 7 and 8. Maturity curve based on cores drilled from slabs.**

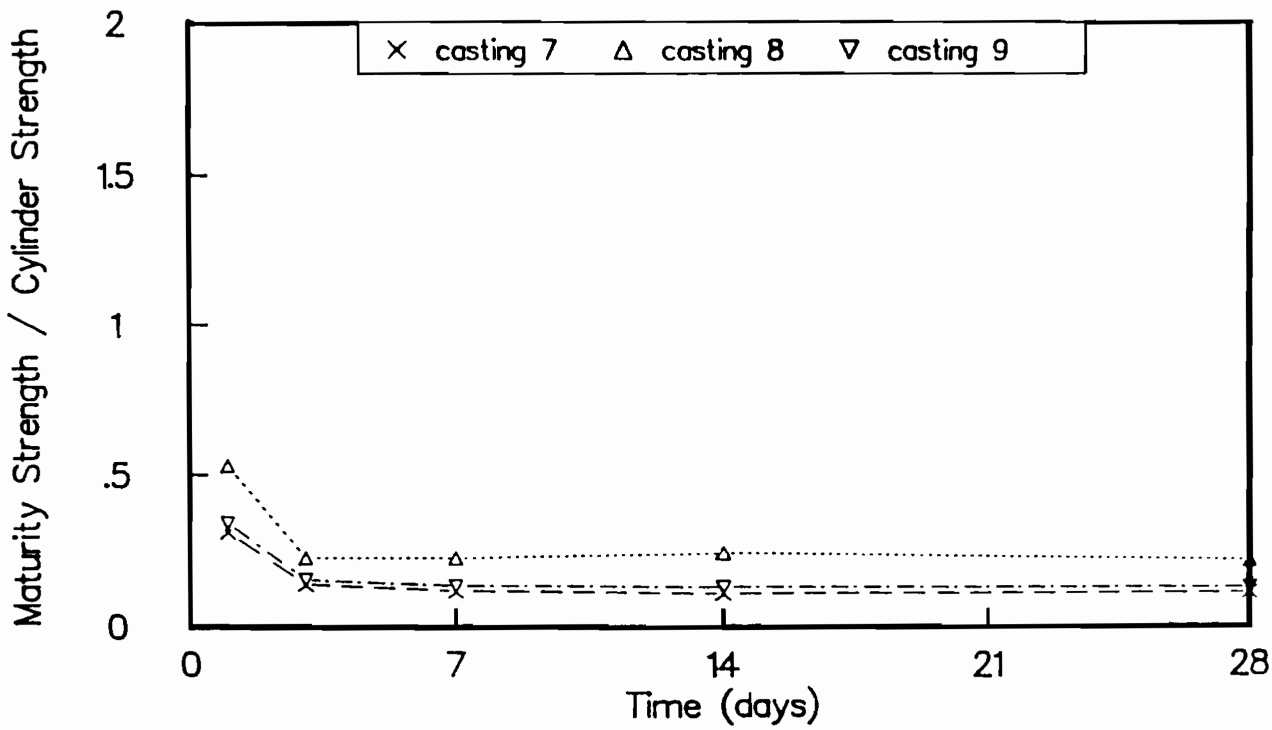


**Fig C-10. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 7, 8, and 9. Maturity curve based on wetroom-cured beams.**

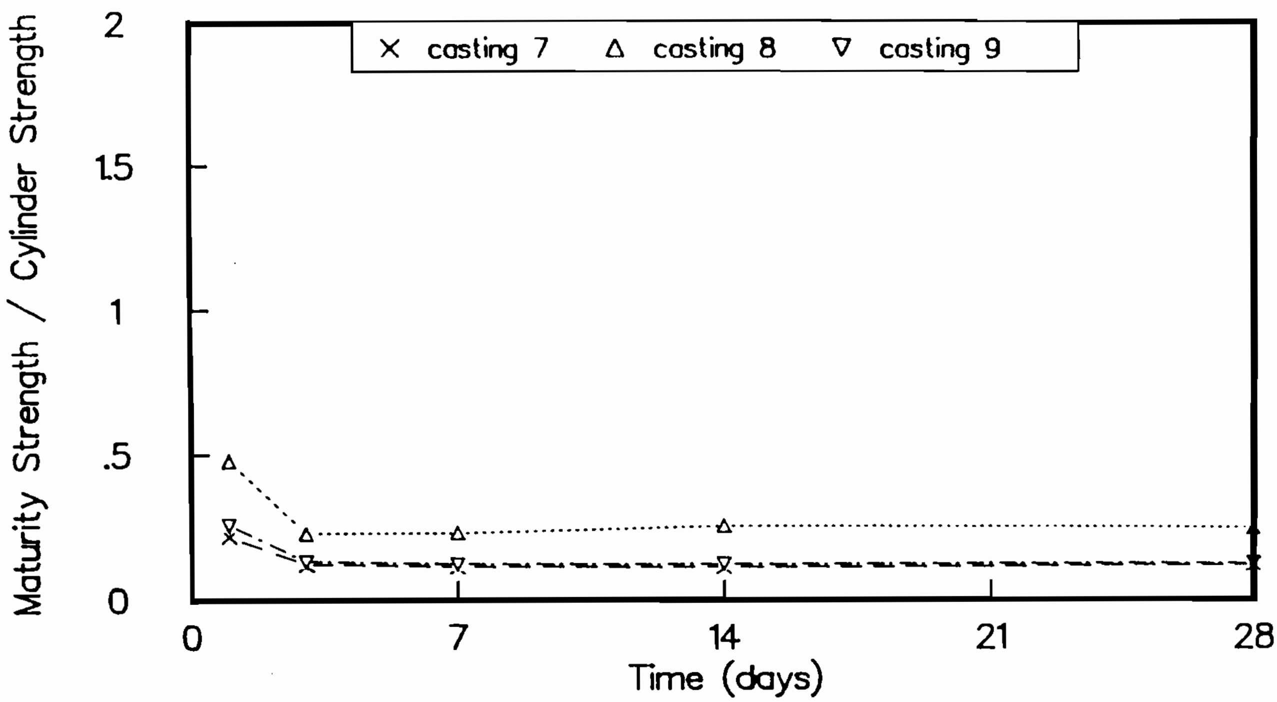


**Fig C-11. Ratio of maturity method strengths to strengths of wetroom-cured cylinders for castings 7, 8, and 9. Maturity curve based on outdoor-cured beams.**

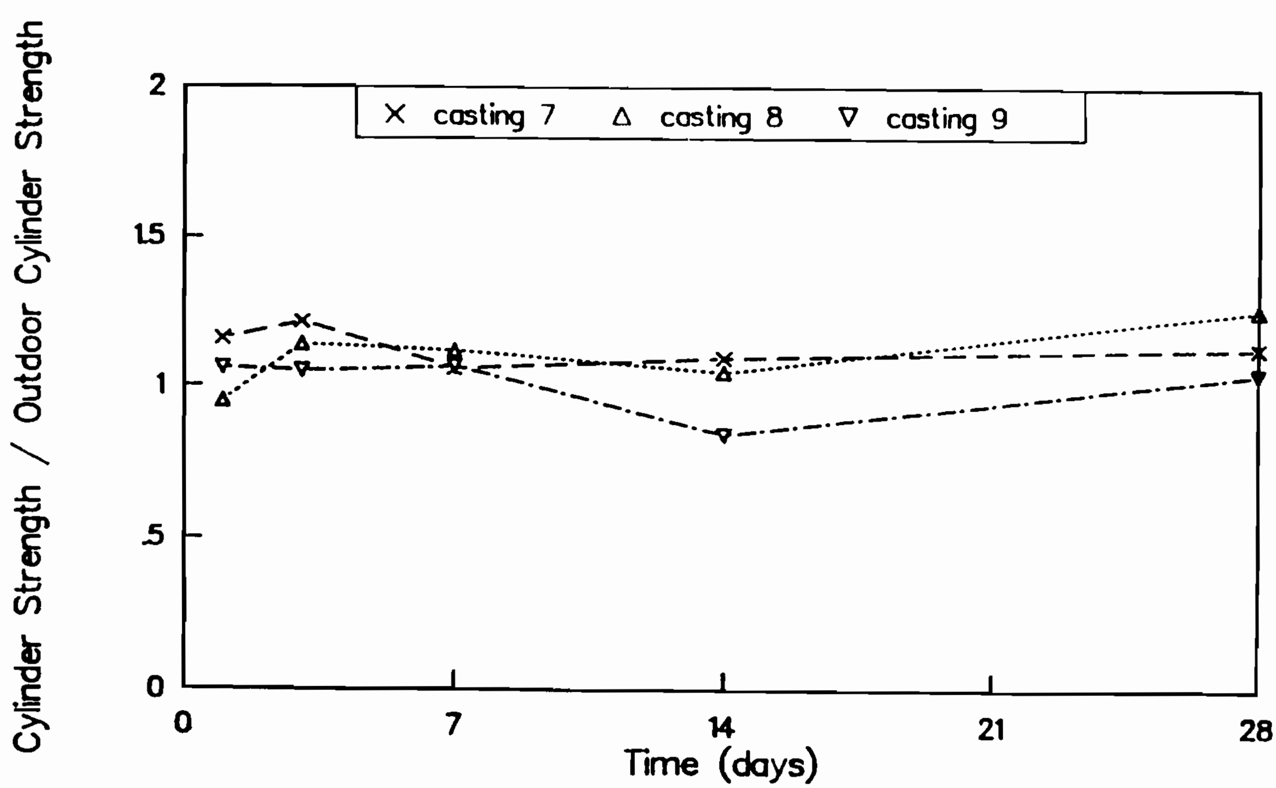


**Fig C-12. Ratio of strengths of wetroom-cured cylinders to strengths of outdoor-cured cylinders for castings 7, 8, and 9.**

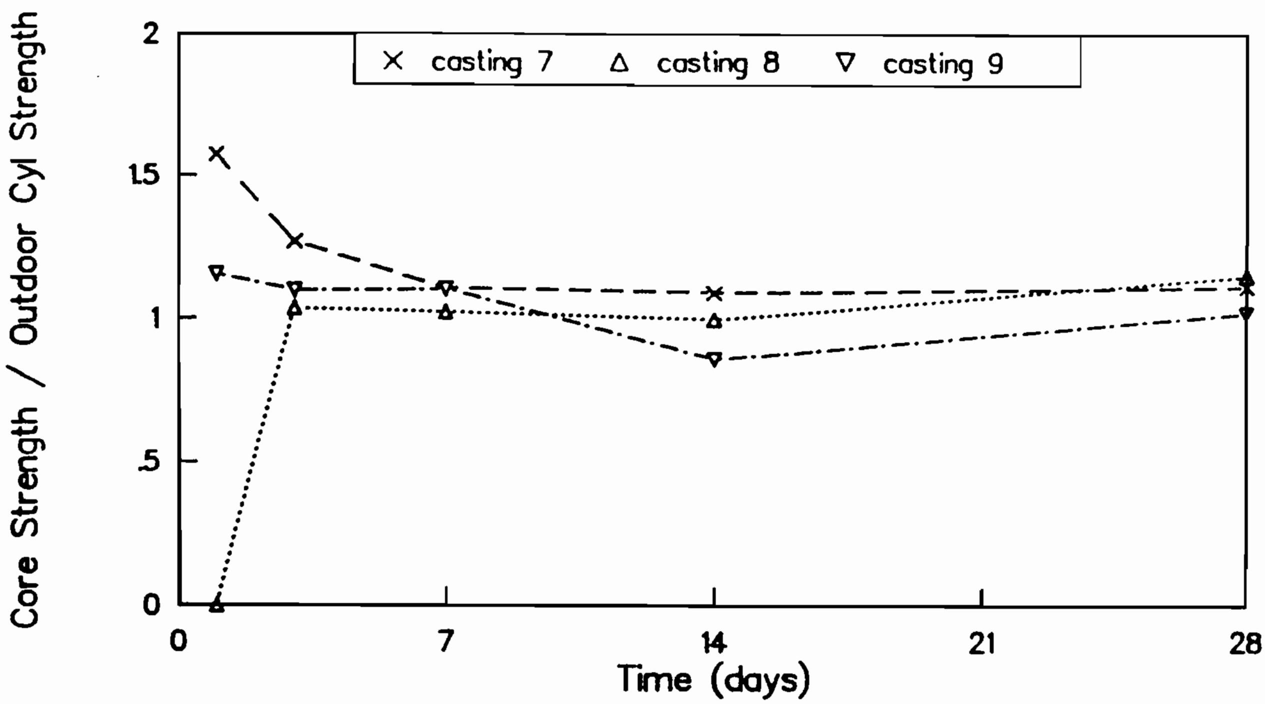


Fig C-13. Ratio of strengths of cores to strengths of outdoor-cured cylinders for castings 7, 8, and 9.

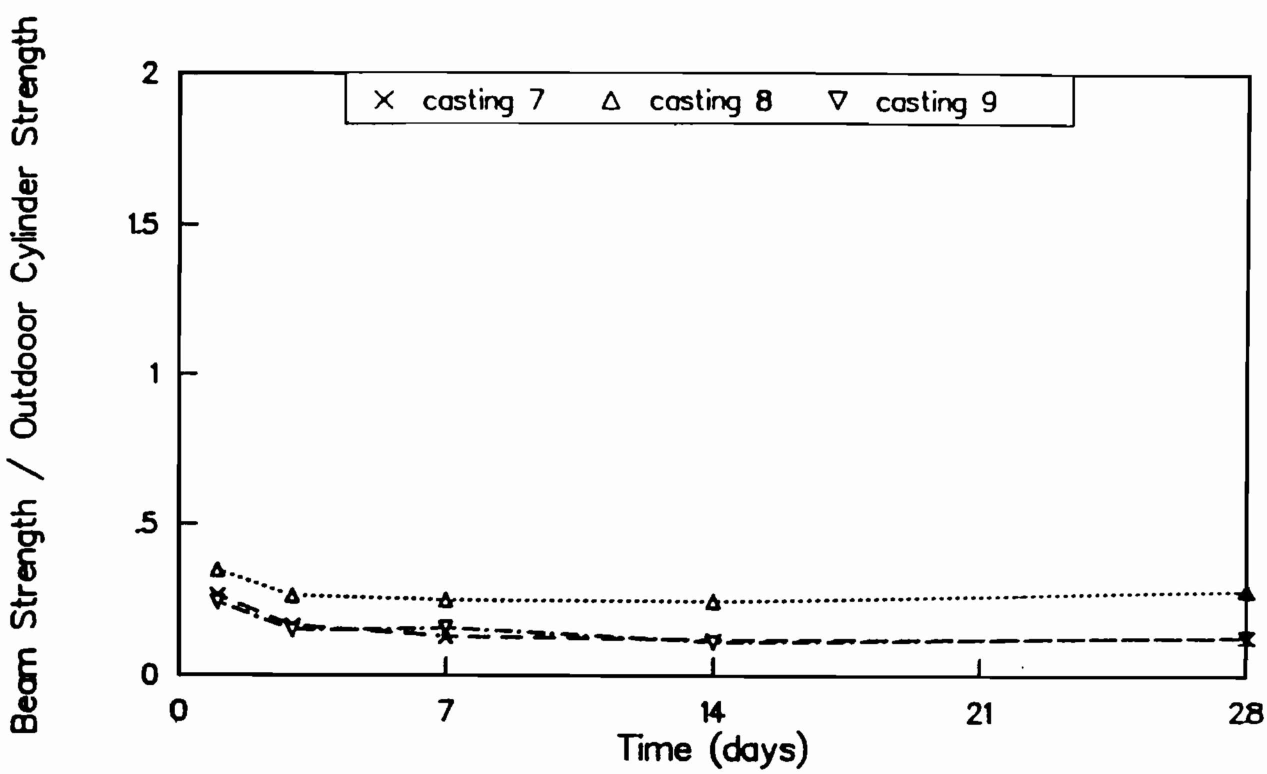


Fig C-14. Ratio of flexural strengths of wetroom-cured beams to strengths of outdoor-cured cylinders for castings 7, 8, and 9.

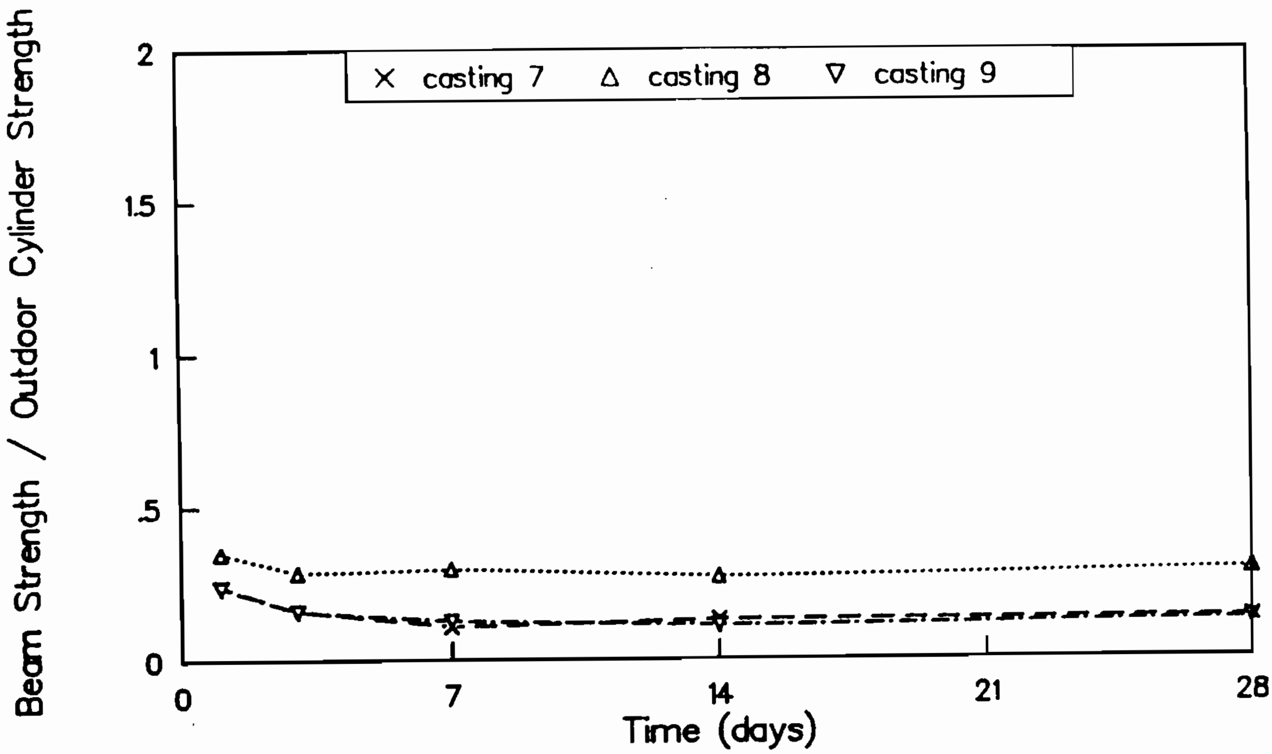


**Fig C-15. Ratio of flexural strengths of outdoor-cured beams to strengths of outdoor-cured cylinders for castings 7, 8, and 9.**

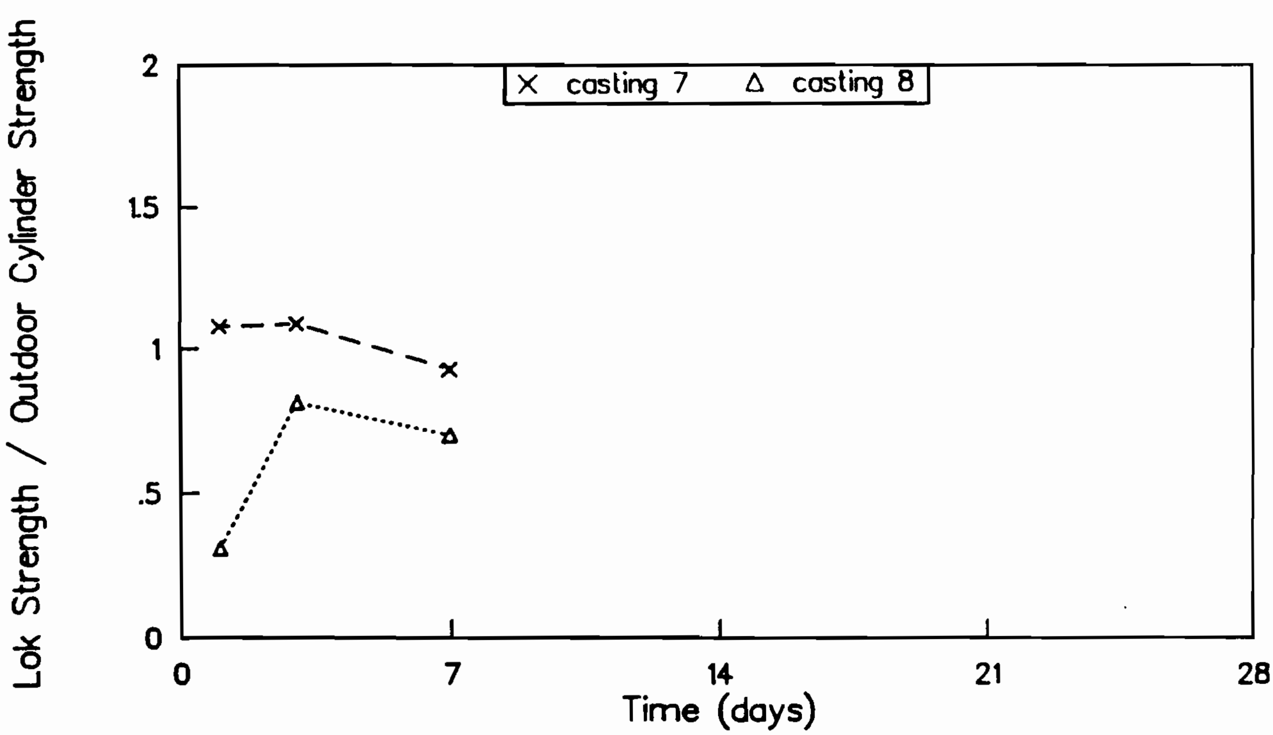


**Fig C-16. Ratio of "Lok-Test" pullout strengths to strengths of outdoor-cured cylinders for castings 7 and 8.**

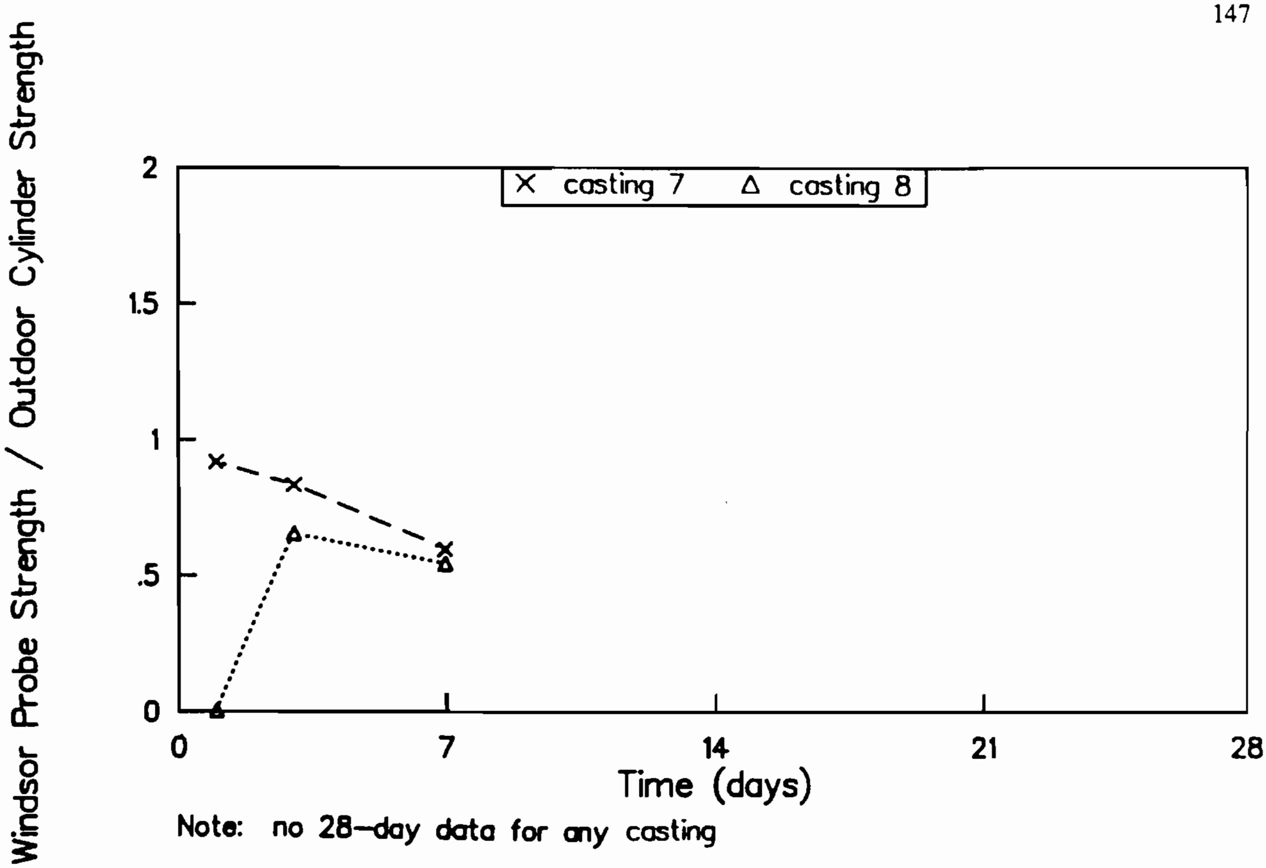


**Fig C-17. Ratio of Windsor probe penetration resistance strengths to strengths of outdoor-cured cylinders for castings 7 and 8.**

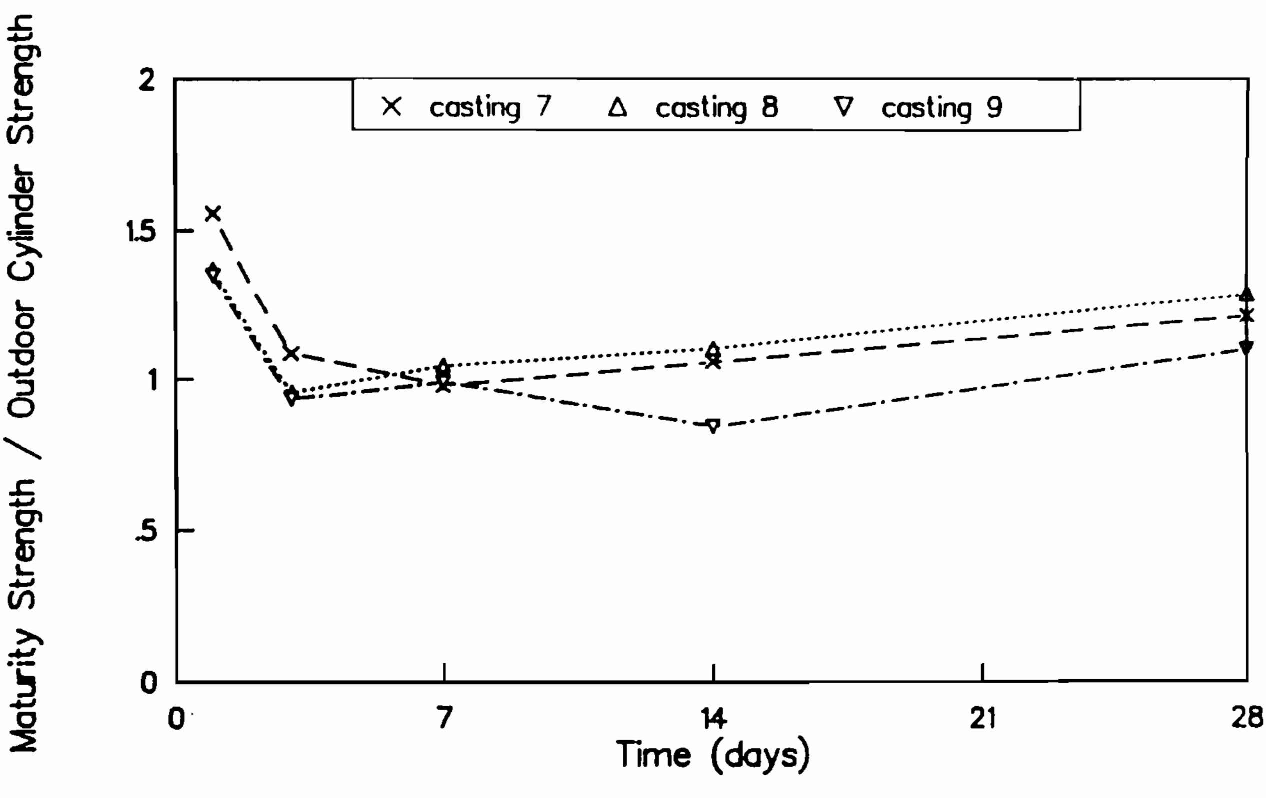


**Fig C-18. Ratio of maturity method strengths to strengths of outdoor-cured cylinders for castings 7, 8, and 9. Maturity curve based on wetroom-cured cylinders.**

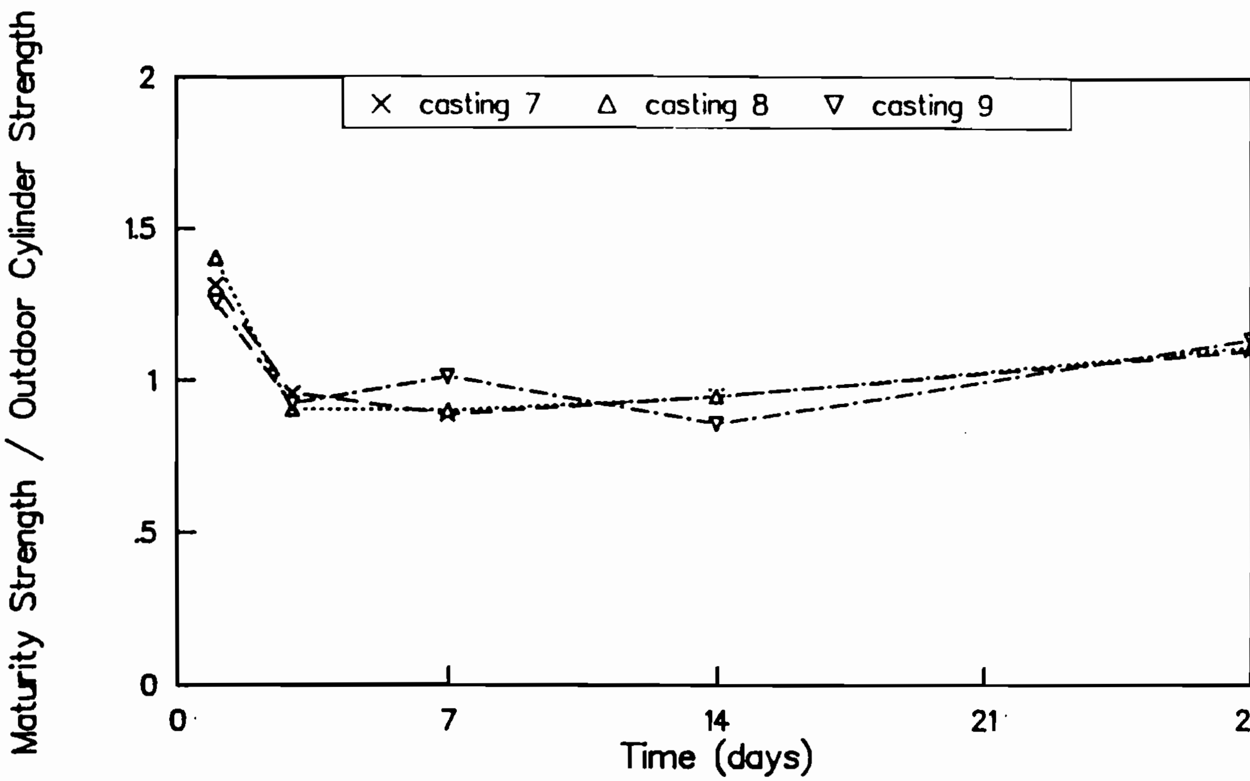


**Fig C-19. Ratio of maturity method strengths to strengths of outdoor-cured cylinders for castings 7, 8, and 9. Maturity curve based on outdoor-cured cylinders.**

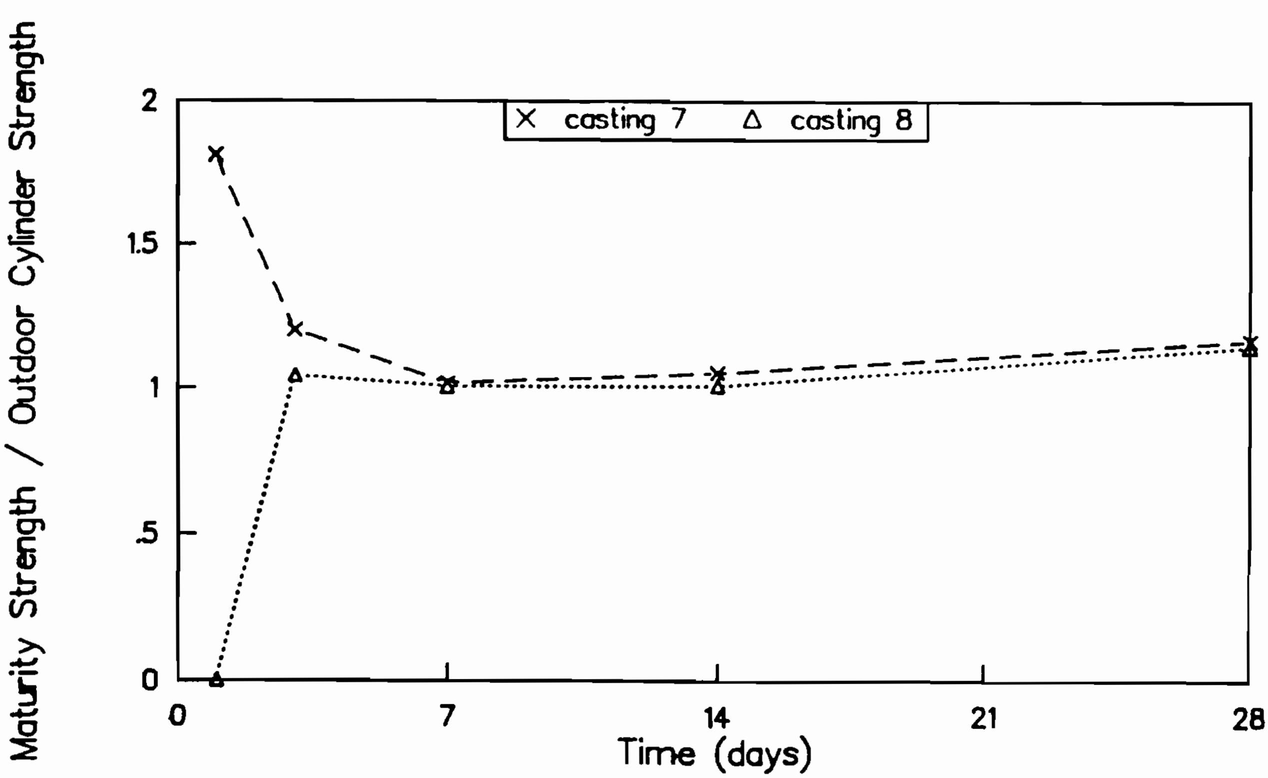


**Fig C-20. Ratio of maturity method strengths to strengths of outdoor-cured cylinders for castings 7 and 8. Maturity curve based on cores drilled from slabs.**

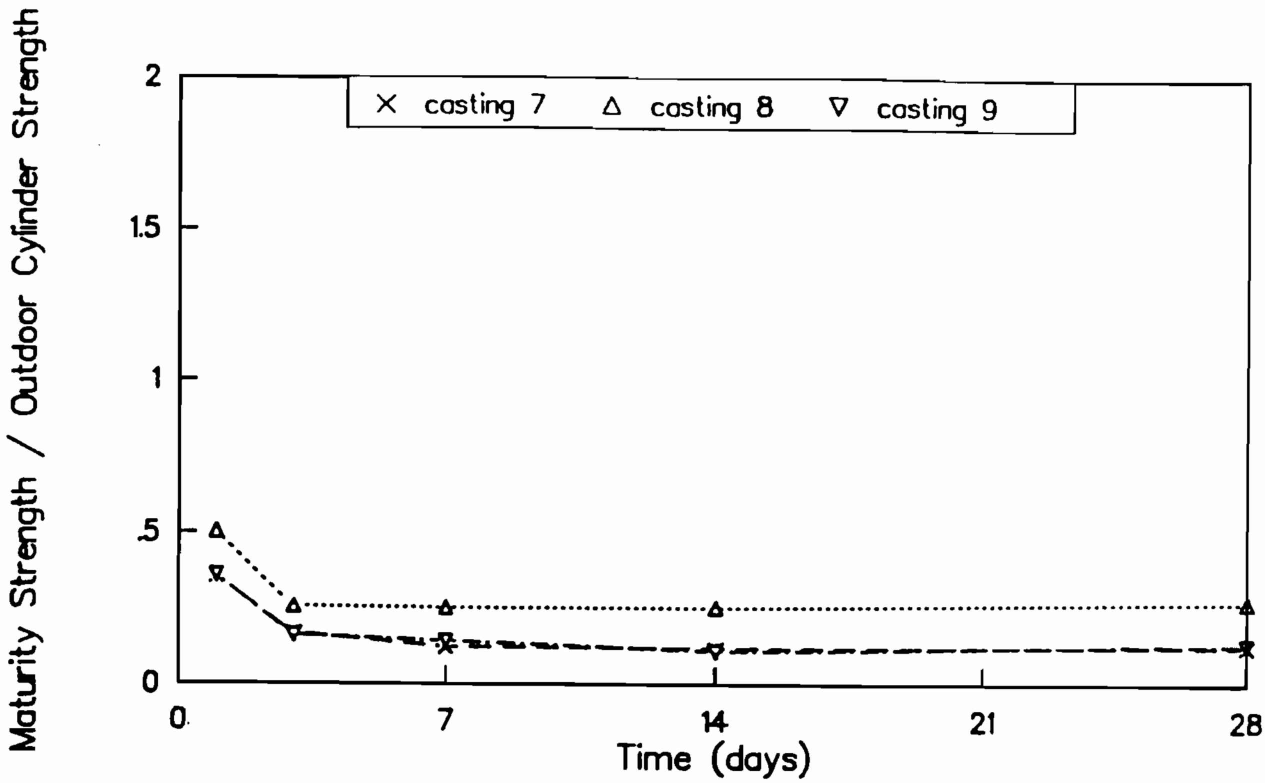


**Fig C-21. Ratio of maturity method strengths to strengths of outdoor-cured cylinders for castings 7, 8, and 9. Maturity curve based on wetroom-cured beams.**

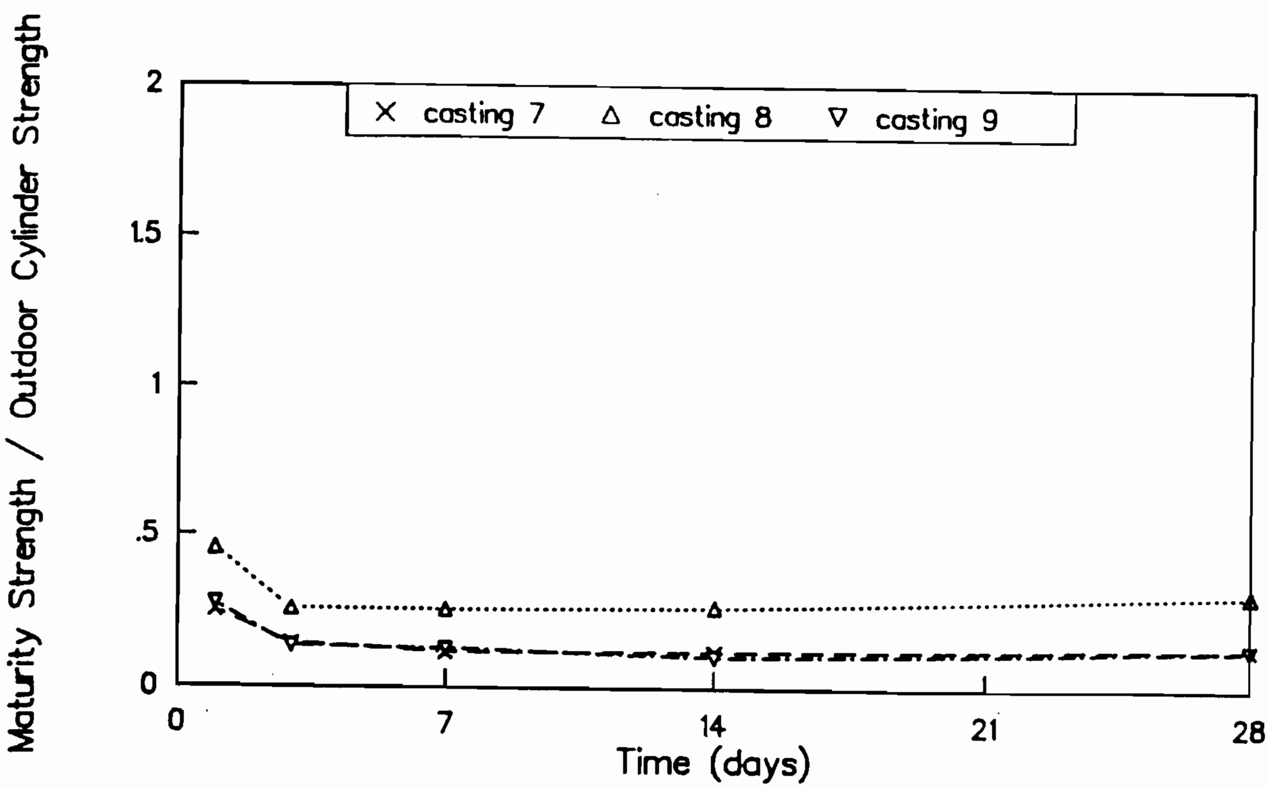


**Fig C-22. Ratio of maturity method strengths to strengths of outdoor-cured cylinders for castings 7, 8, and 9. Maturity curve based on outdoor-cured beams.**

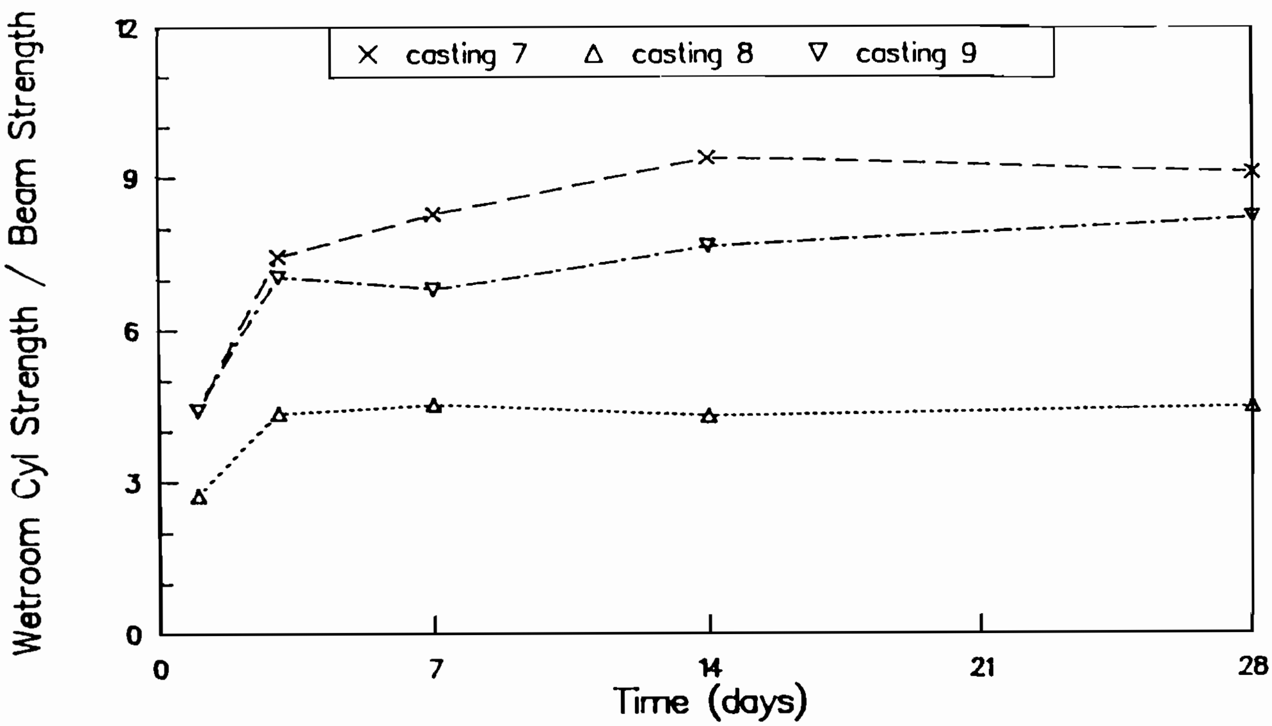


**Fig C-23. Ratio of strengths of wetroom-cured cylinders to flexural strengths of wetroom-cured beams for castings 7, 8, and 9.**

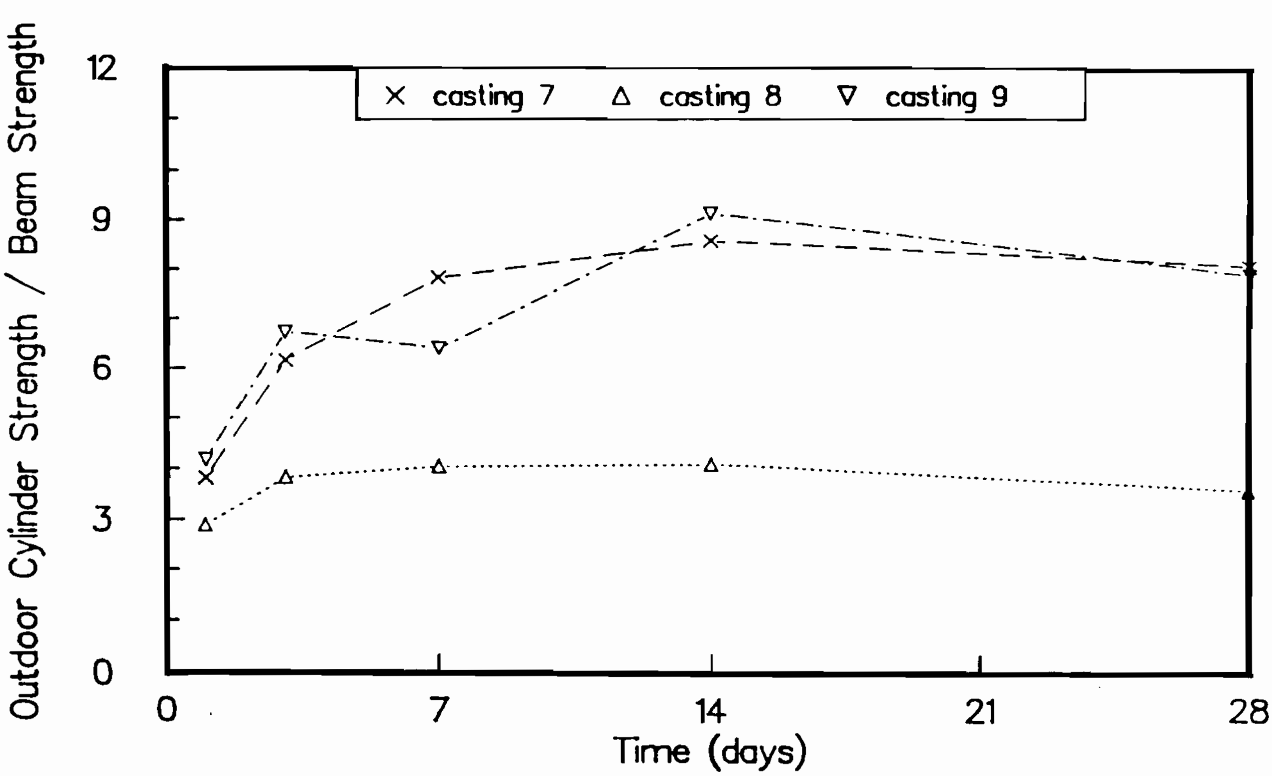


**Fig C-24. Ratio of strengths of outdoor-cured cylinders to flexural strengths of wetroom-cured beams for castings 7, 8, and 9.**

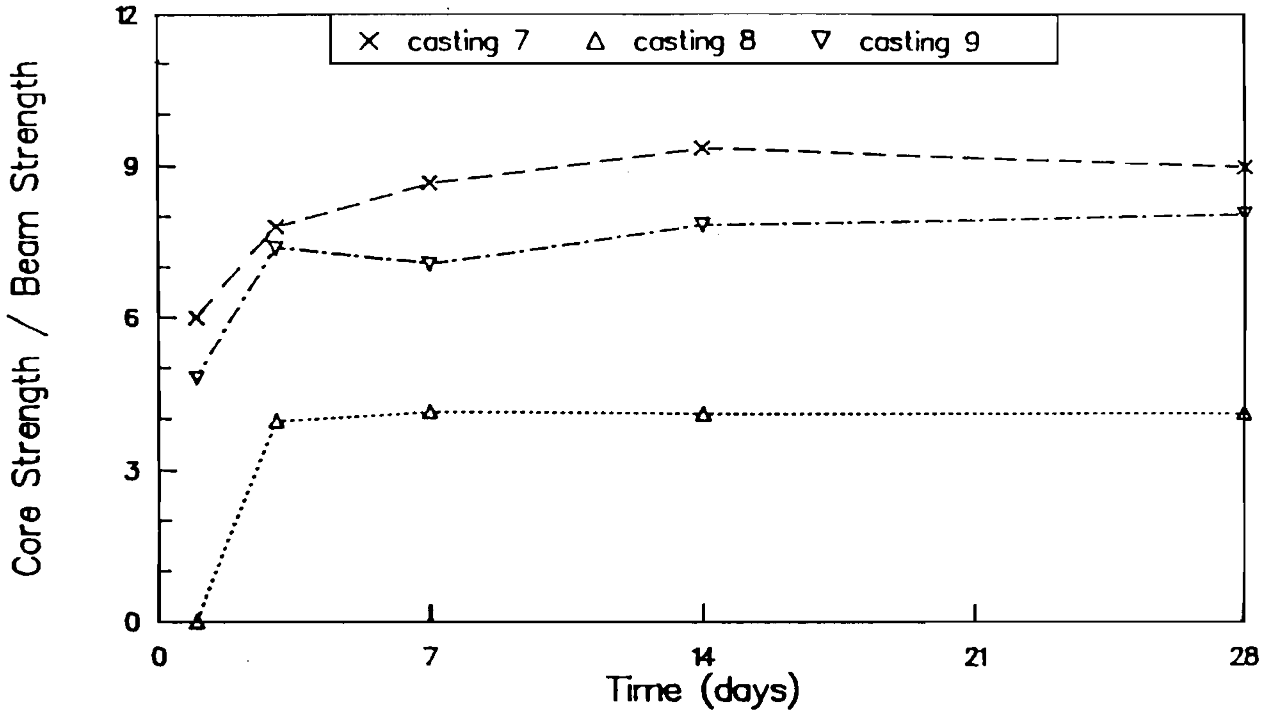


**Fig C-25. Ratio of strengths of cores to flexural strengths of wetroom-cured beams for castings 7, 8, and 9.**

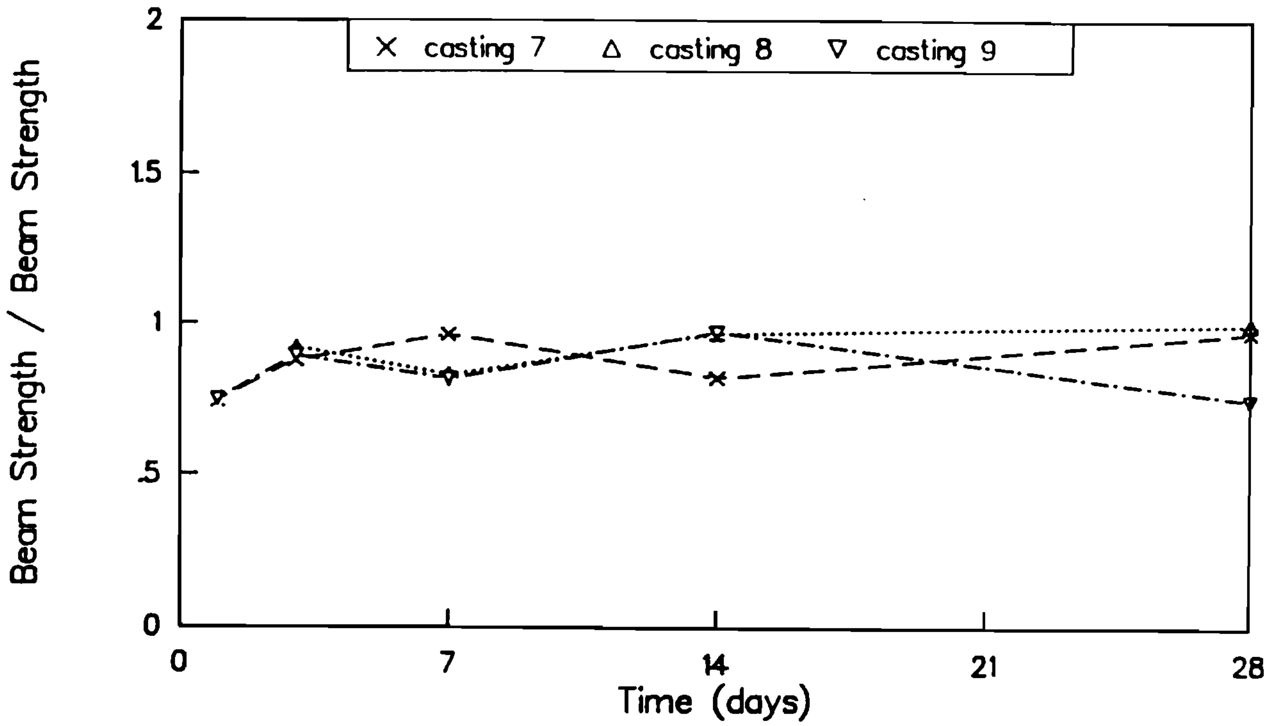


**Fig C-26. Ratio of flexural strengths of outdoor-cured beams to flexural strengths of wetroom-cured beams for castings 7, 8, and 9.**

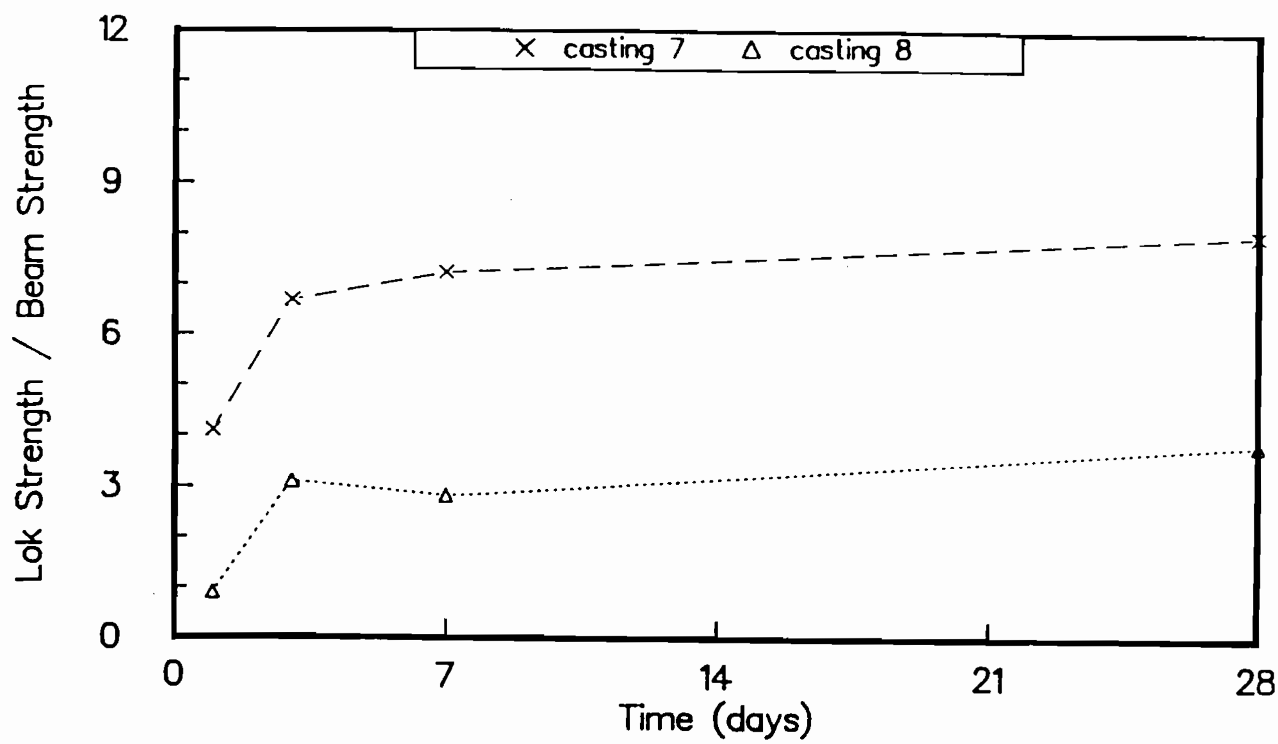


**Fig C-27. Ratio of "Lok-Test" pullout strengths to flexural strengths of wetroom-cured beams for castings 7 and 8.**

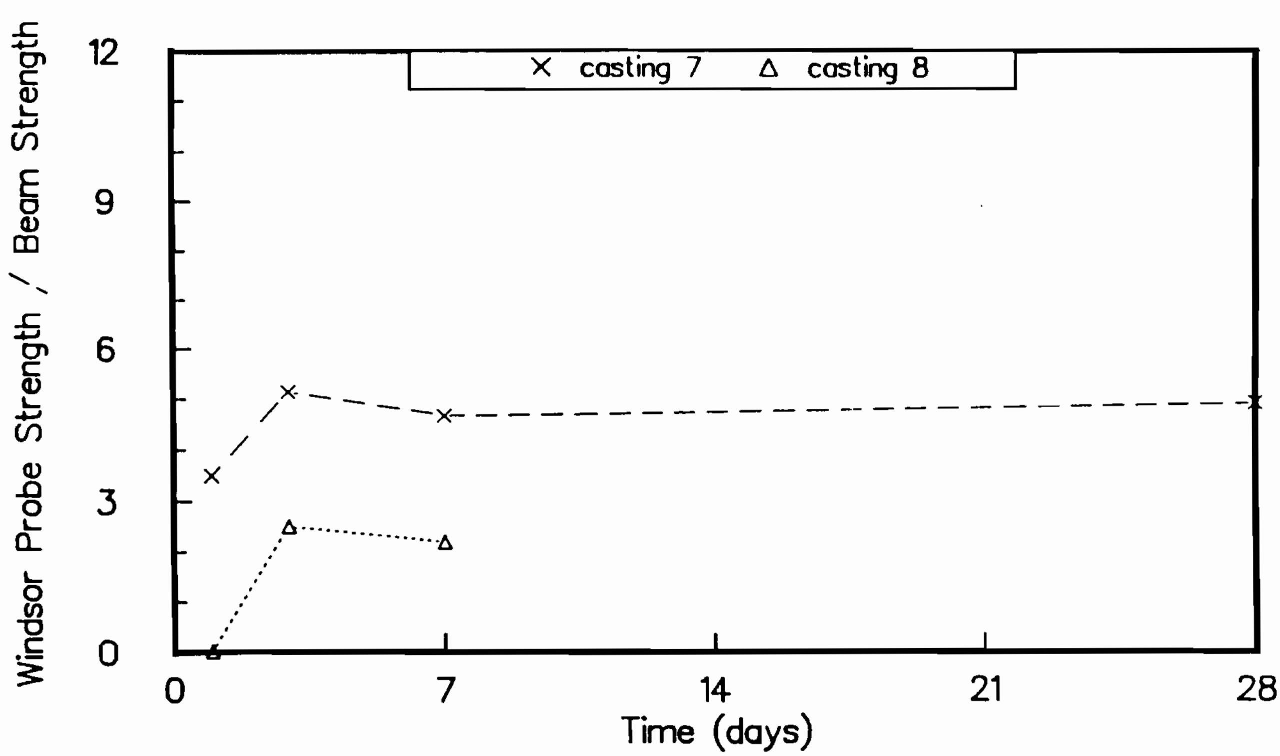


**Fig C-28. Ratio of Windsor probe penetration resistance strengths to flexural strengths of wetroom-cured beams for castings 7 and 8.**

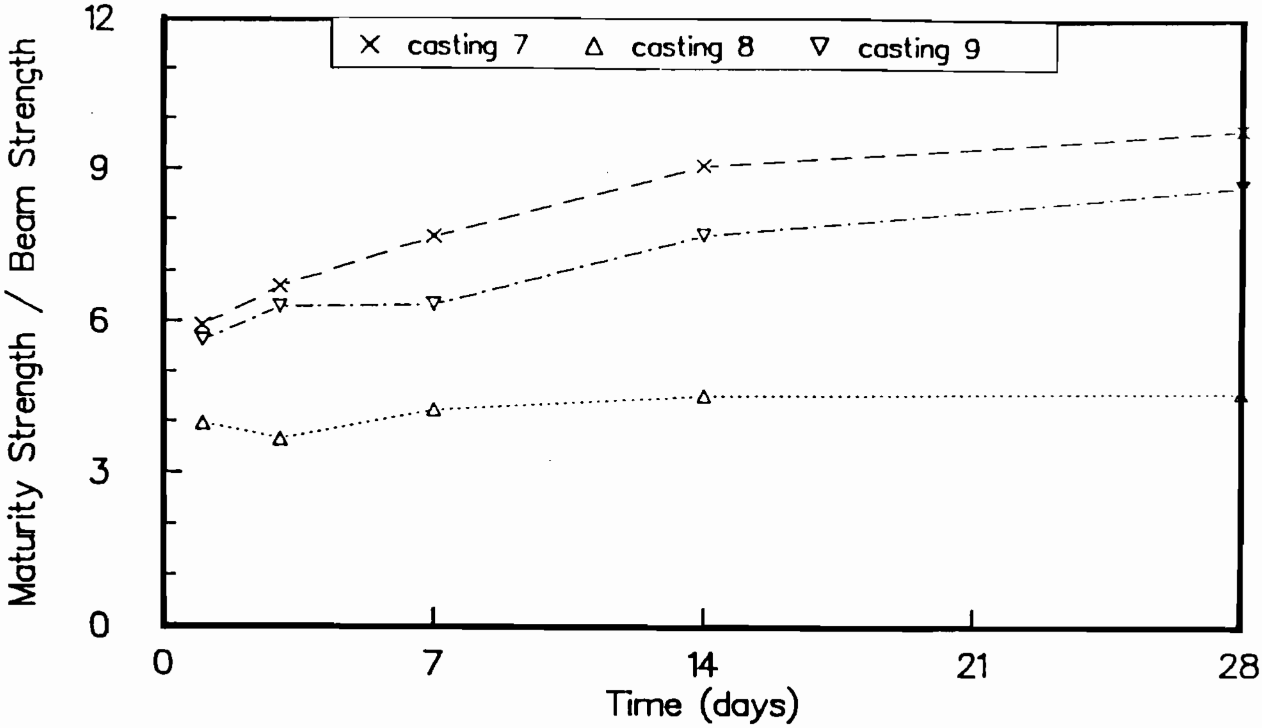


**Fig C-29. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 7, 8, and 9. Maturity curve based on wetroom-cured cylinders.**

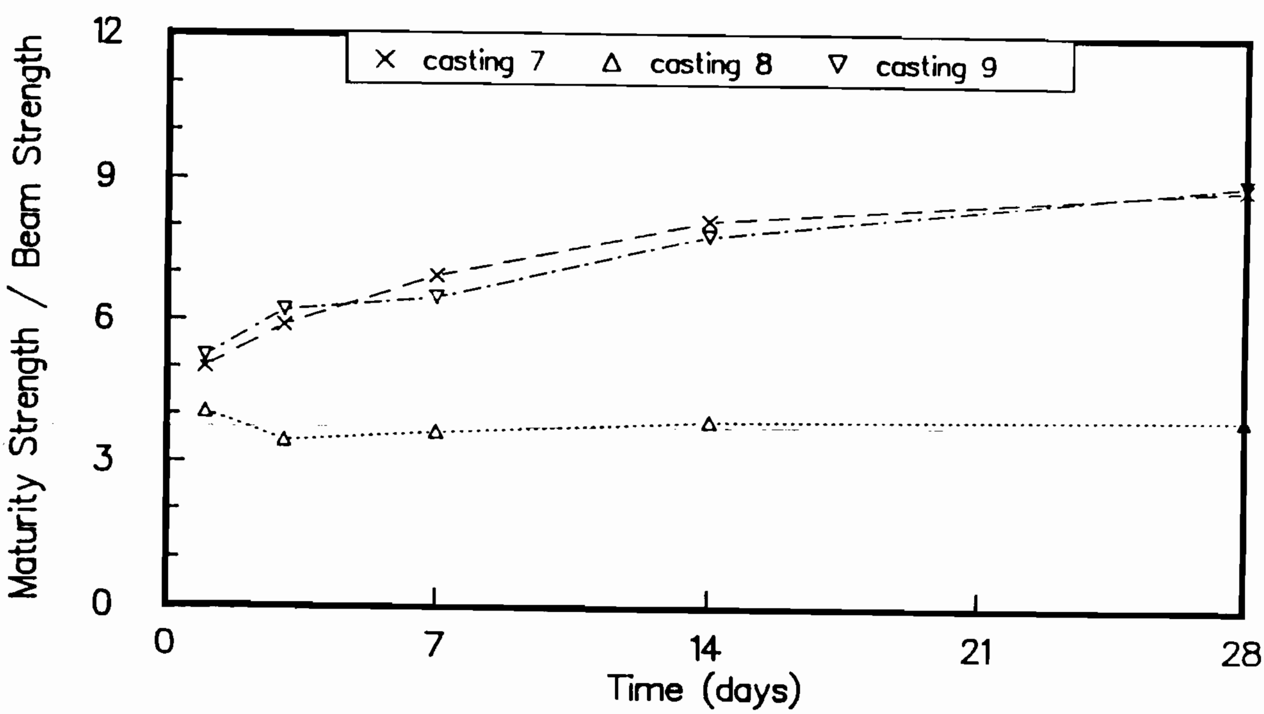


**Fig C-30. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 7, 8, and 9. Maturity curve based on outdoor-cured cylinders.**

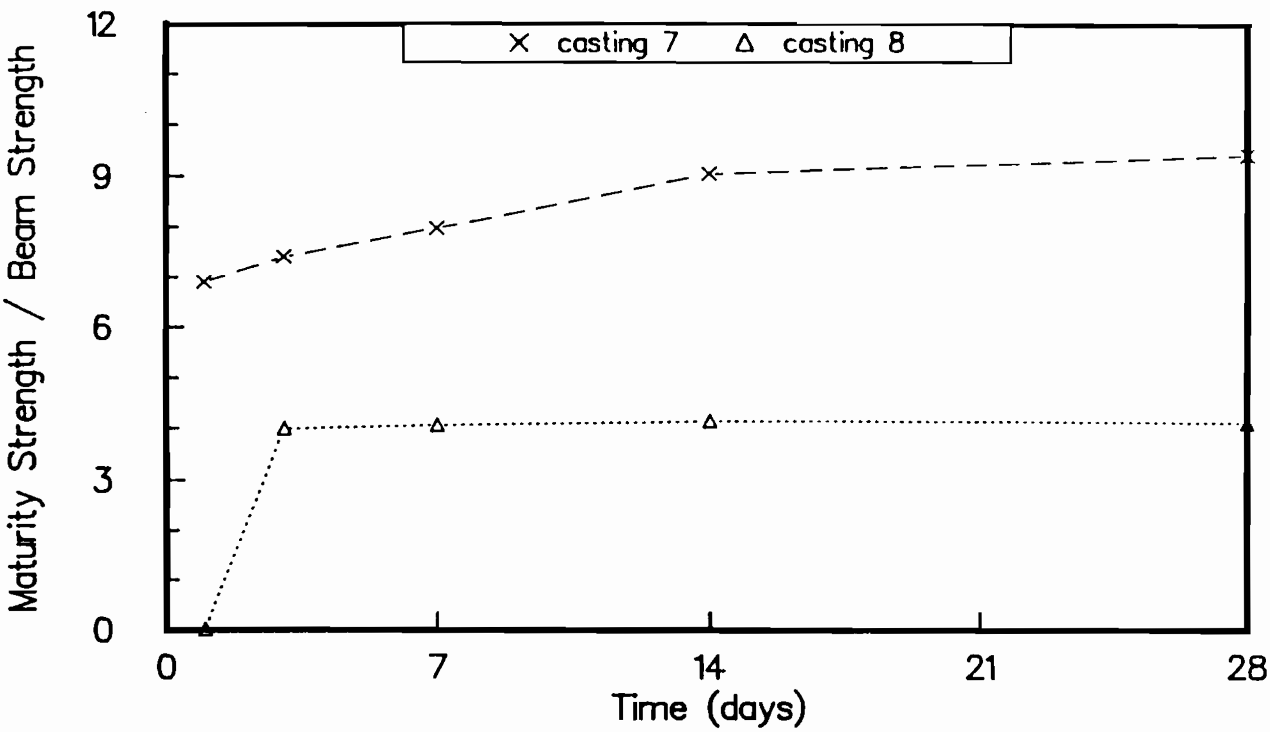


**Fig C-31. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 7 and 8. Maturity curve based on cores drilled from slabs.**

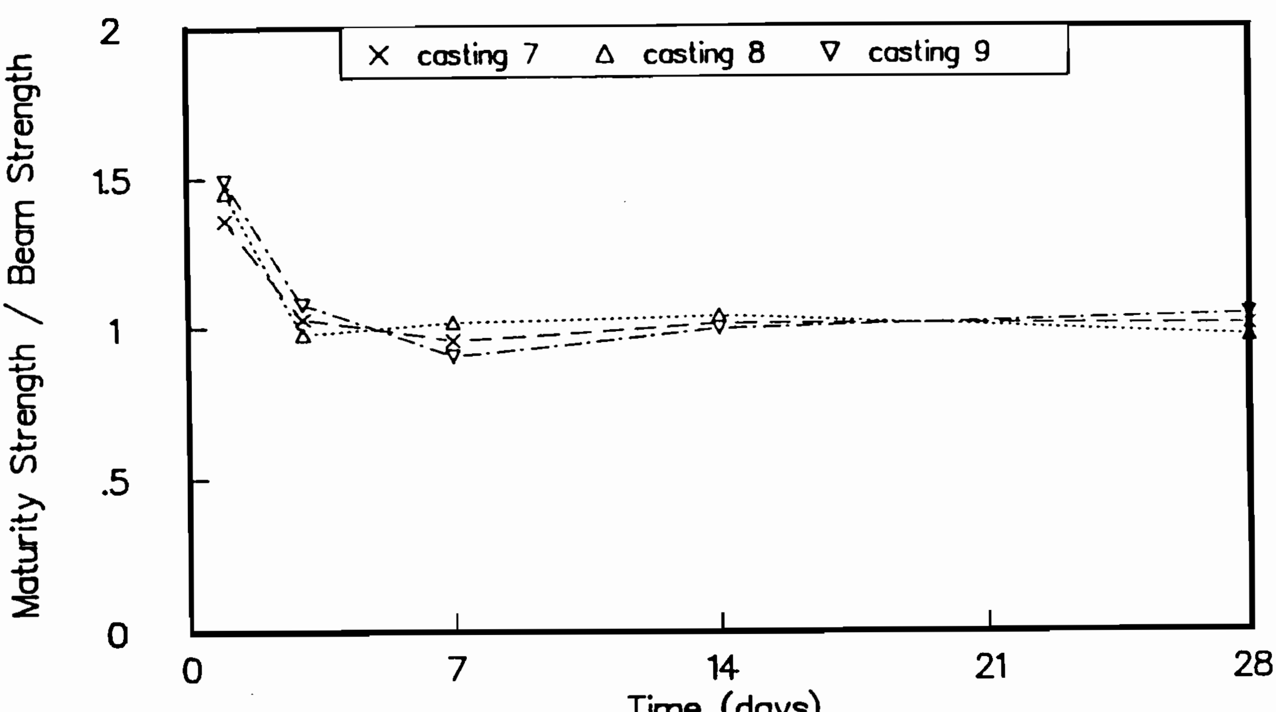


**Fig C-32. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 7, 8, and 9. Maturity curve based on wetroom-cured beams.**

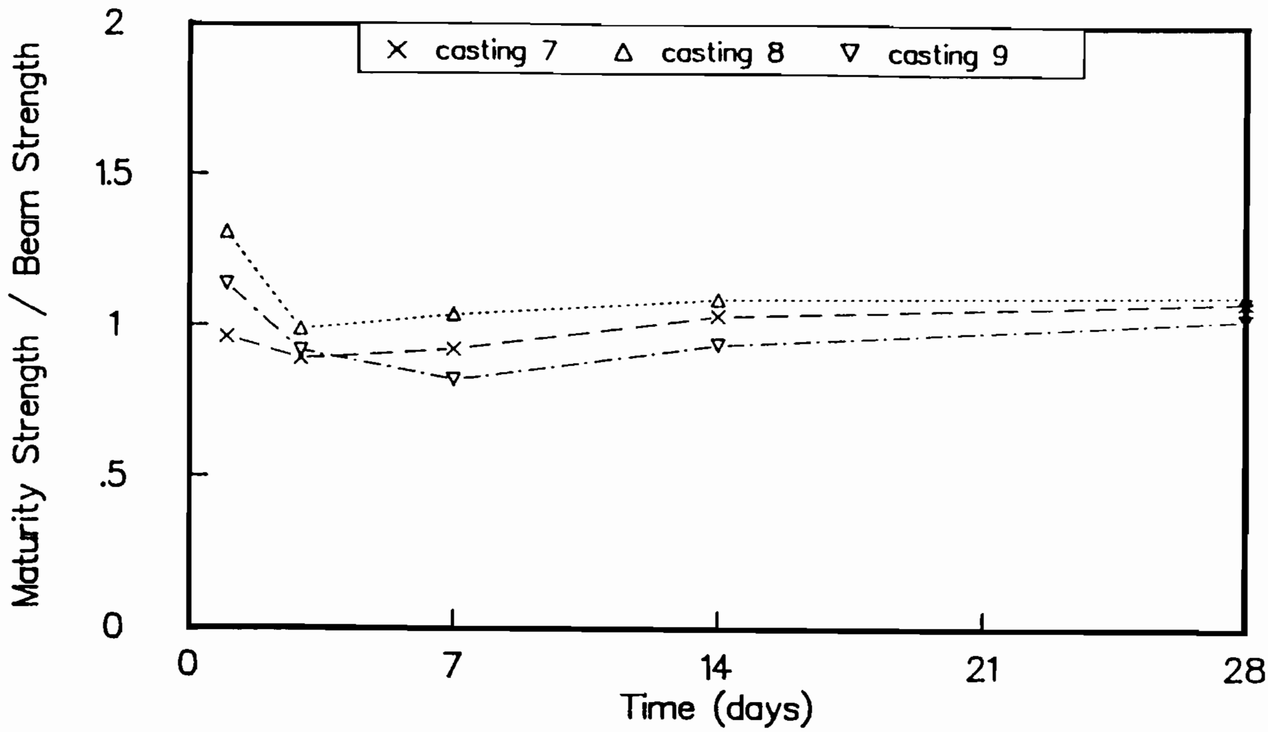


**Fig C-33. Ratio of maturity method strengths to flexural strengths of wetroom-cured beams for castings 7, 8, and 9. Maturity curve based on outdoor-cured beams.**

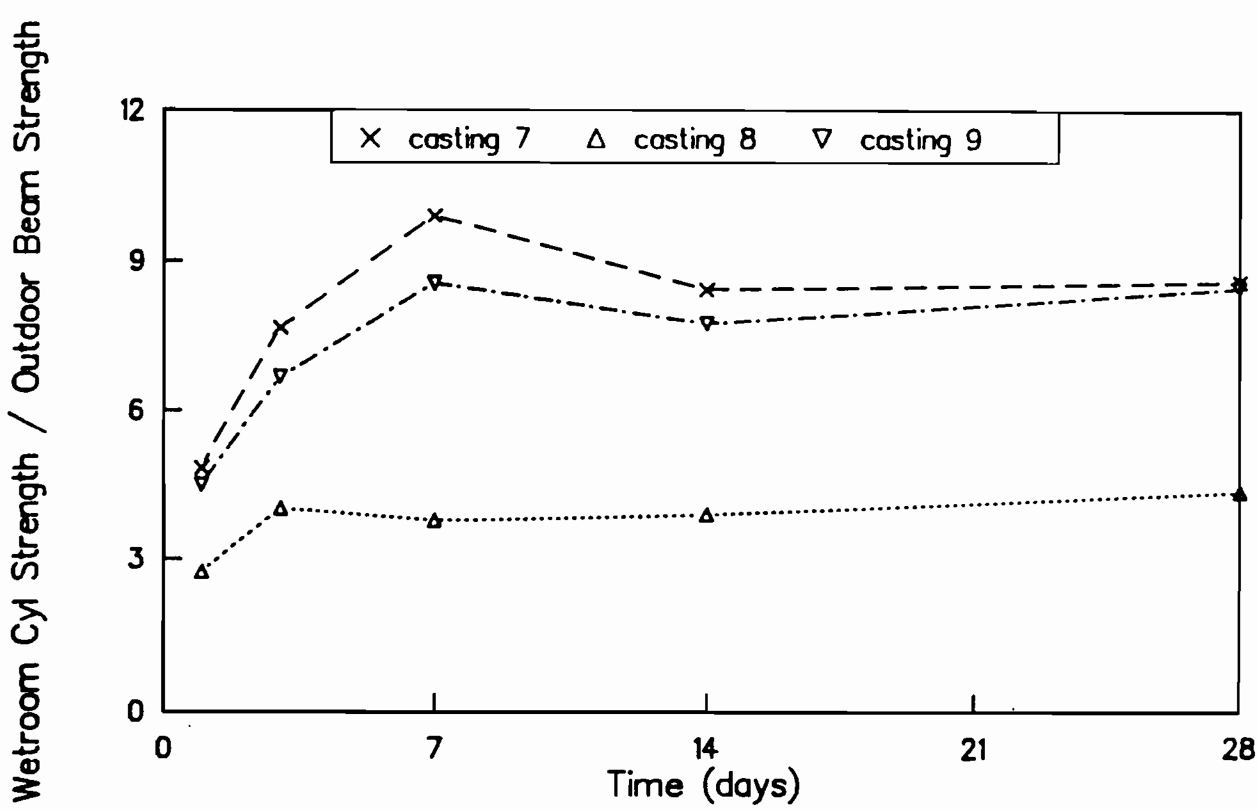


**Fig C-34. Ratio of strengths of wetroom-cured cylinders to flexural strengths of outdoor-cured beams for castings 7, 8, and 9.**

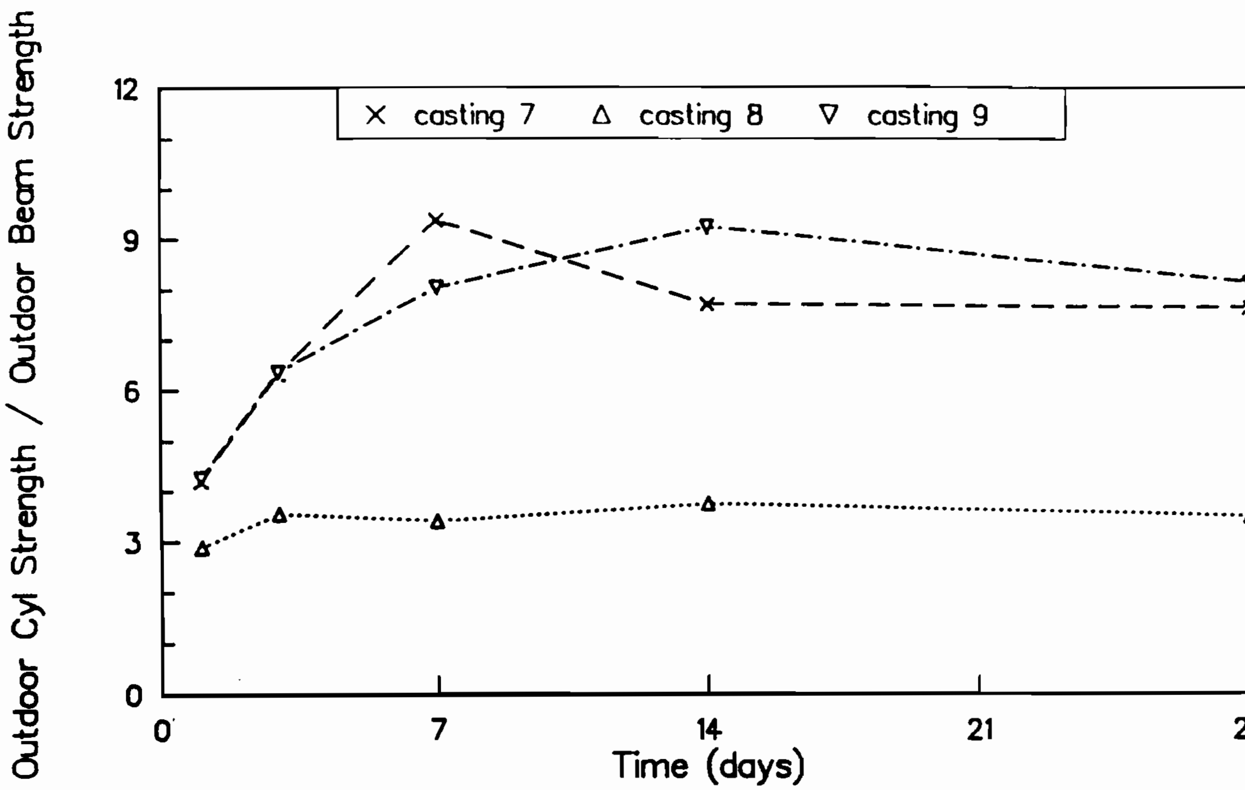


**Fig C-35. Ratio of strengths of outdoor-cured cylinders to flexural strengths of outdoor-cured beams for castings 7, 8, and 9.**

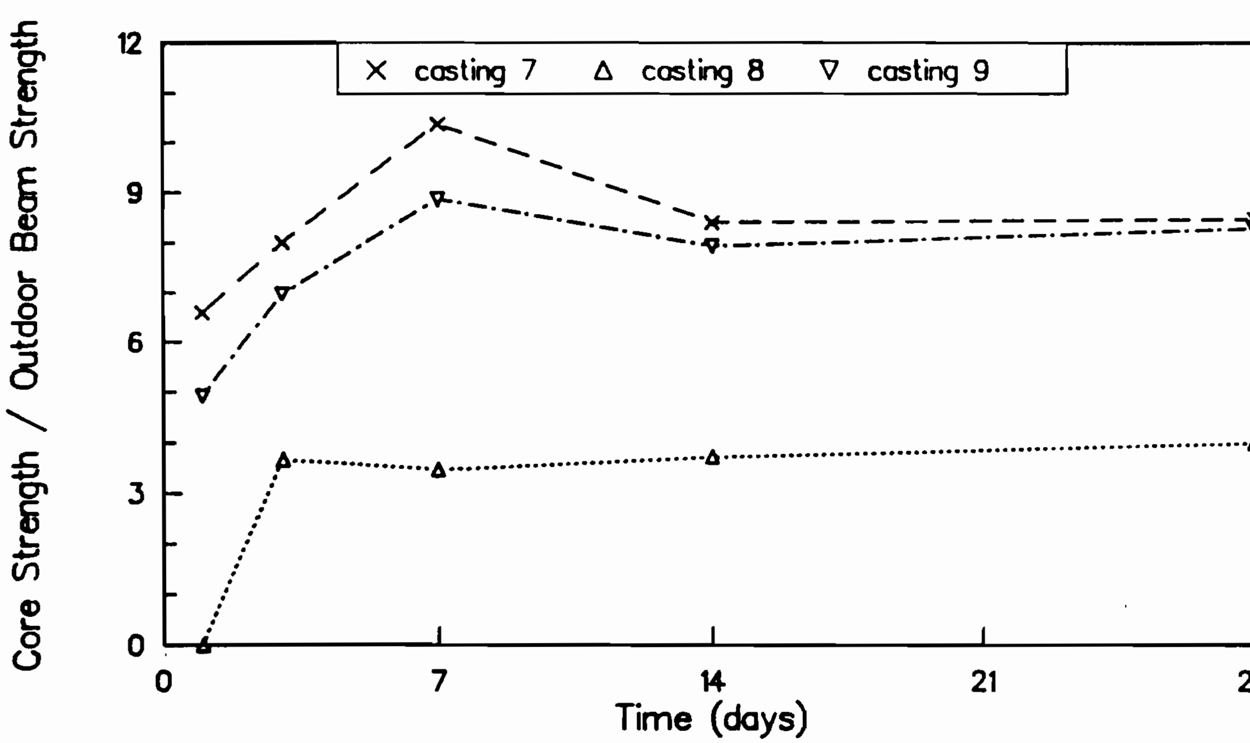


**Fig C-36. Ratio of strengths of cores to flexural strengths of outdoor-cured beams for castings 7, 8, and 9.**

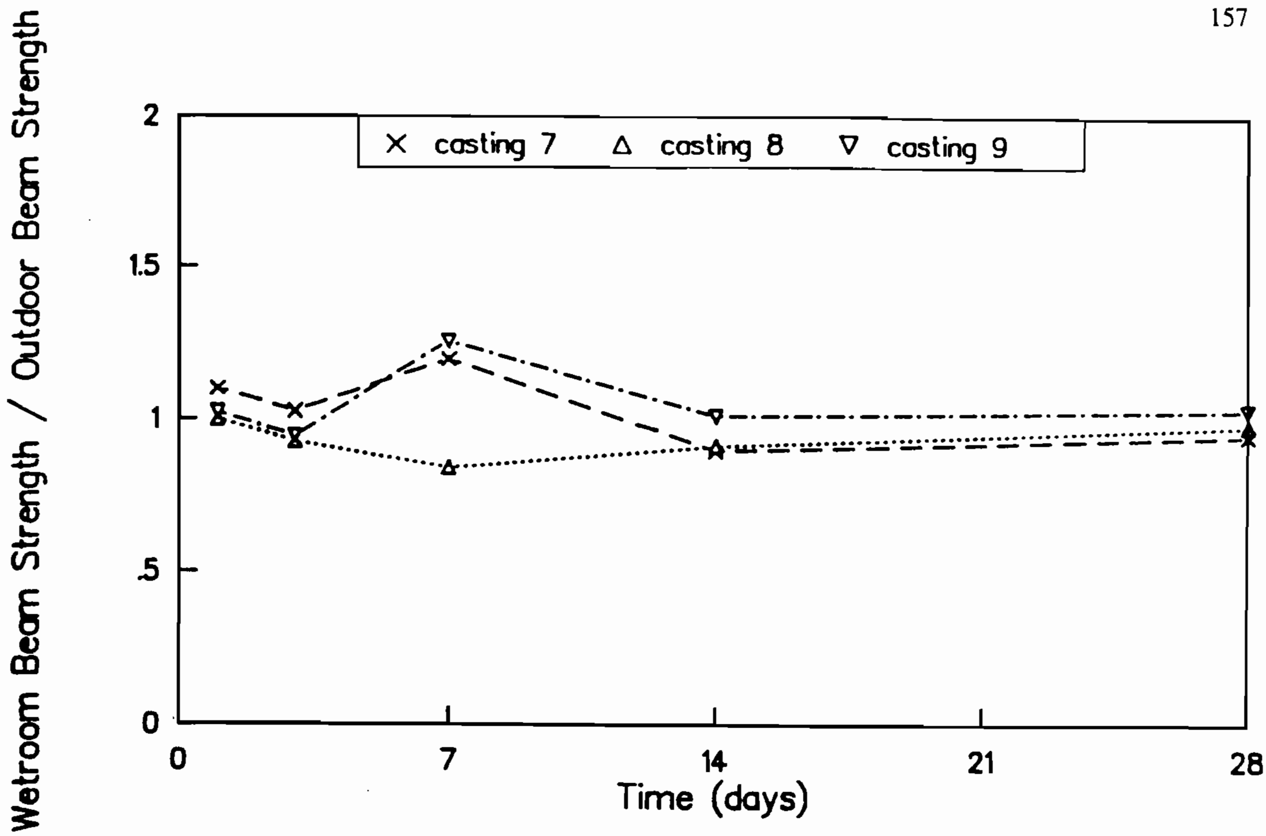


Fig C-37. Ratio of flexural strengths of wetroom-cured beams to flexural strengths of outdoor-cured beams for castings 7, 8, and 9.

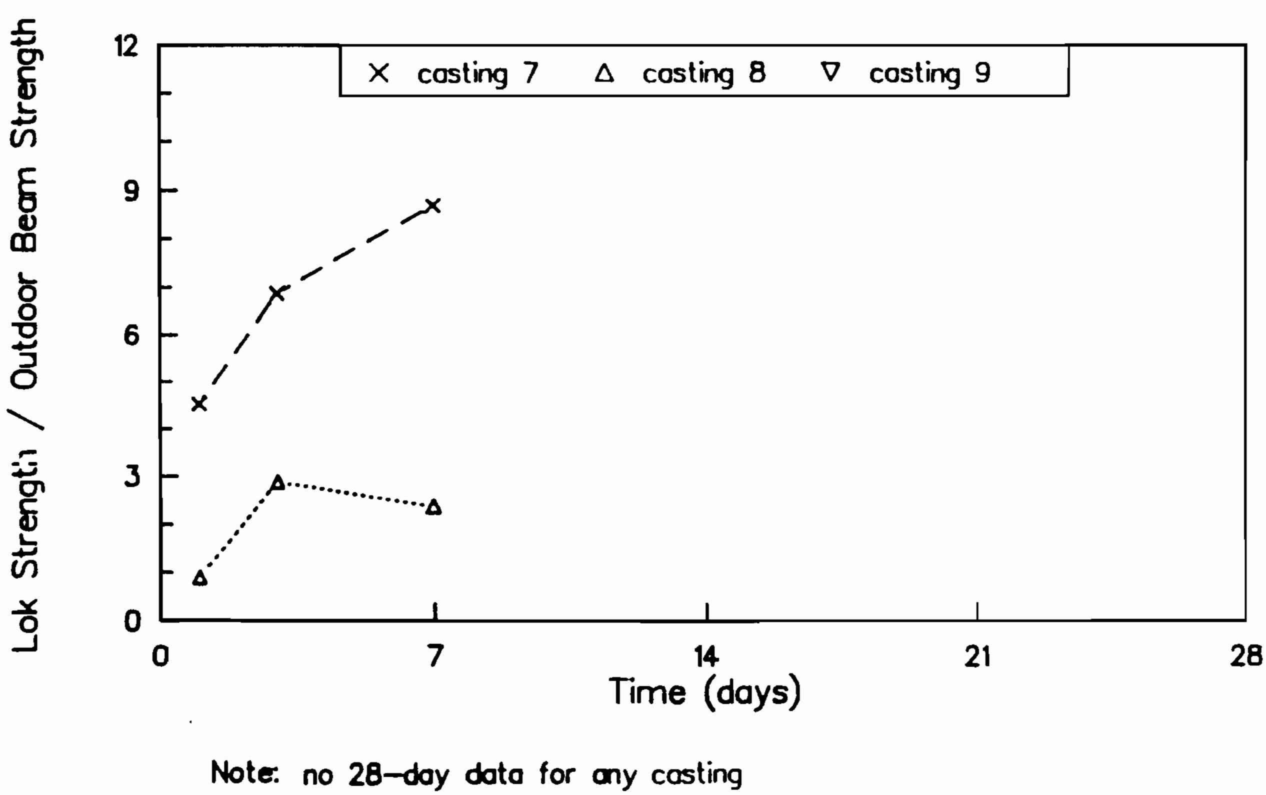


Fig C-38. Ratio of "Lok-Test" pullout strengths to flexural strengths of outdoor-cured beams for castings 7 and 8.

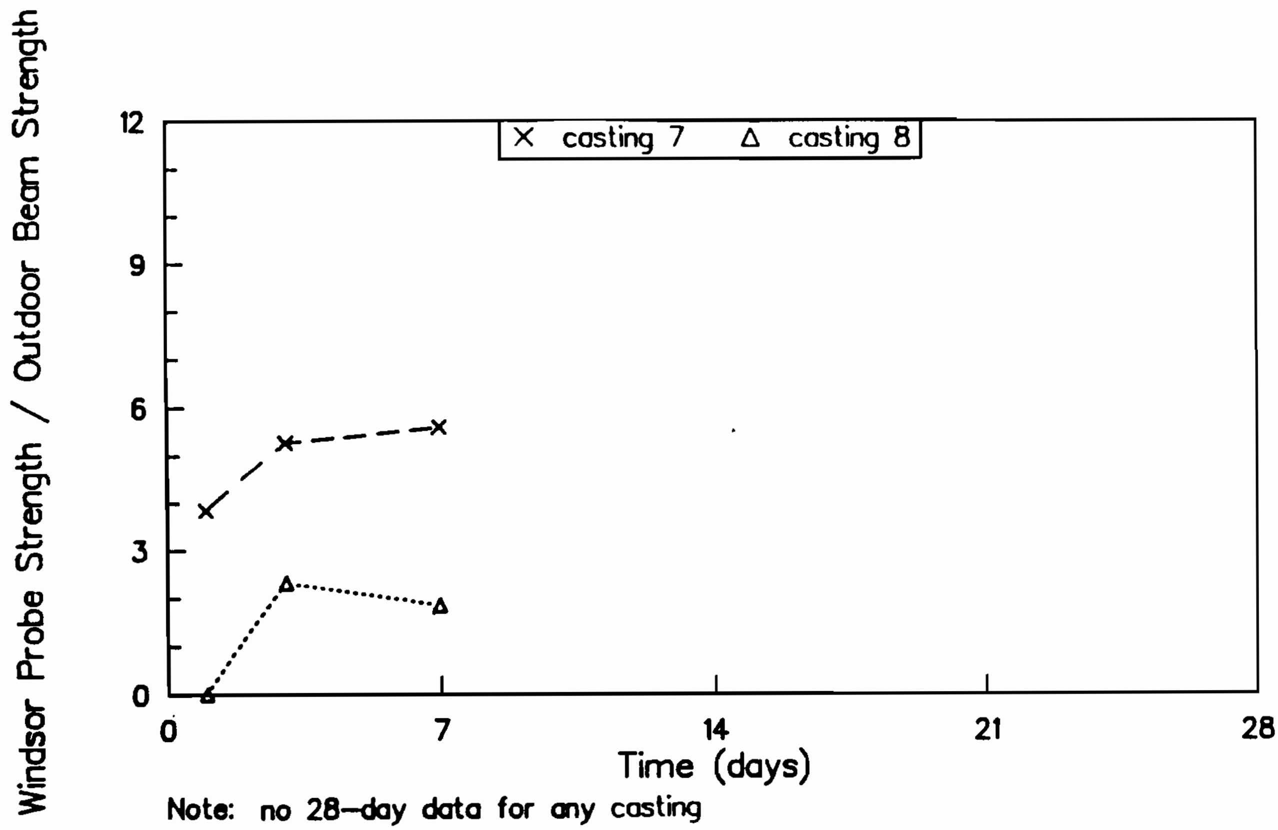


Fig C-39. Ratio of Windsor probe penetration resistance strengths to flexural strengths of outdoor-cured beams for castings 7 and 8.

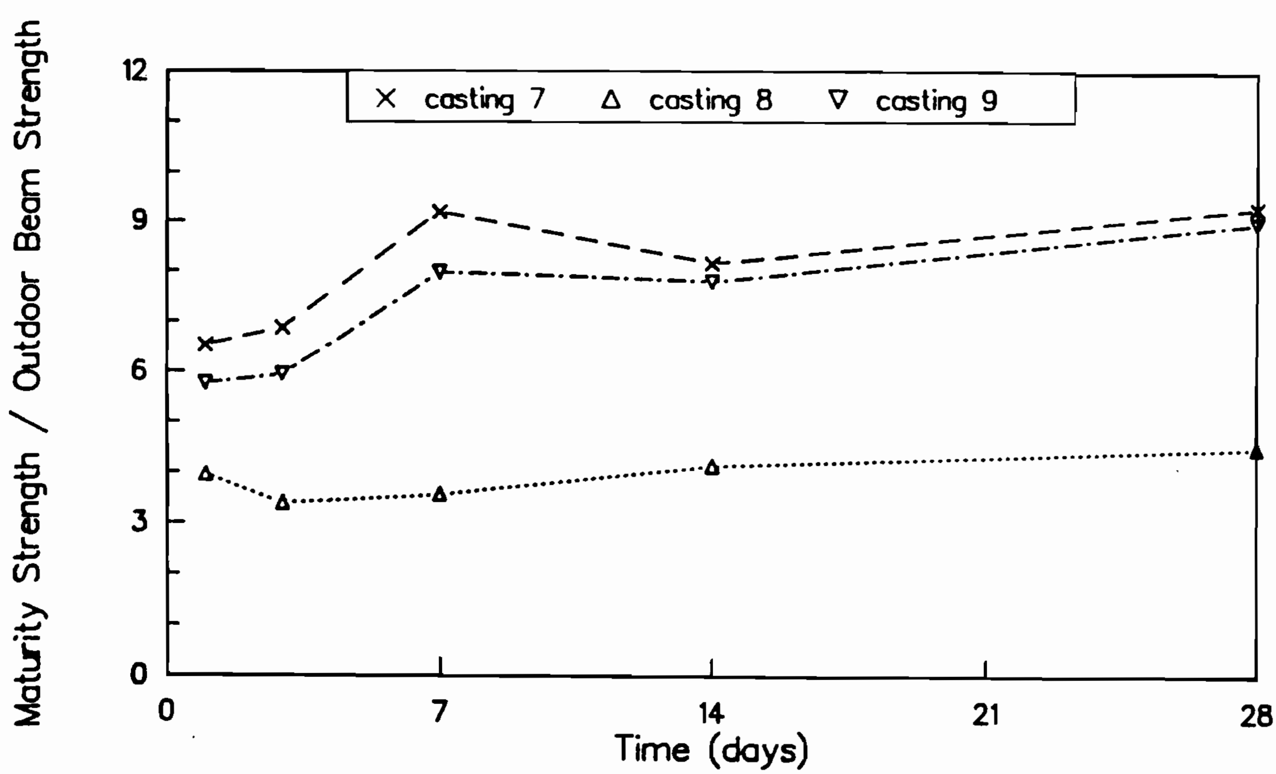


Fig C-40. Ratio of maturity method strengths to flexural strengths of outdoor-cured beams for castings 7, 8, and 9. Maturity curve based on wetroom-cured cylinders.

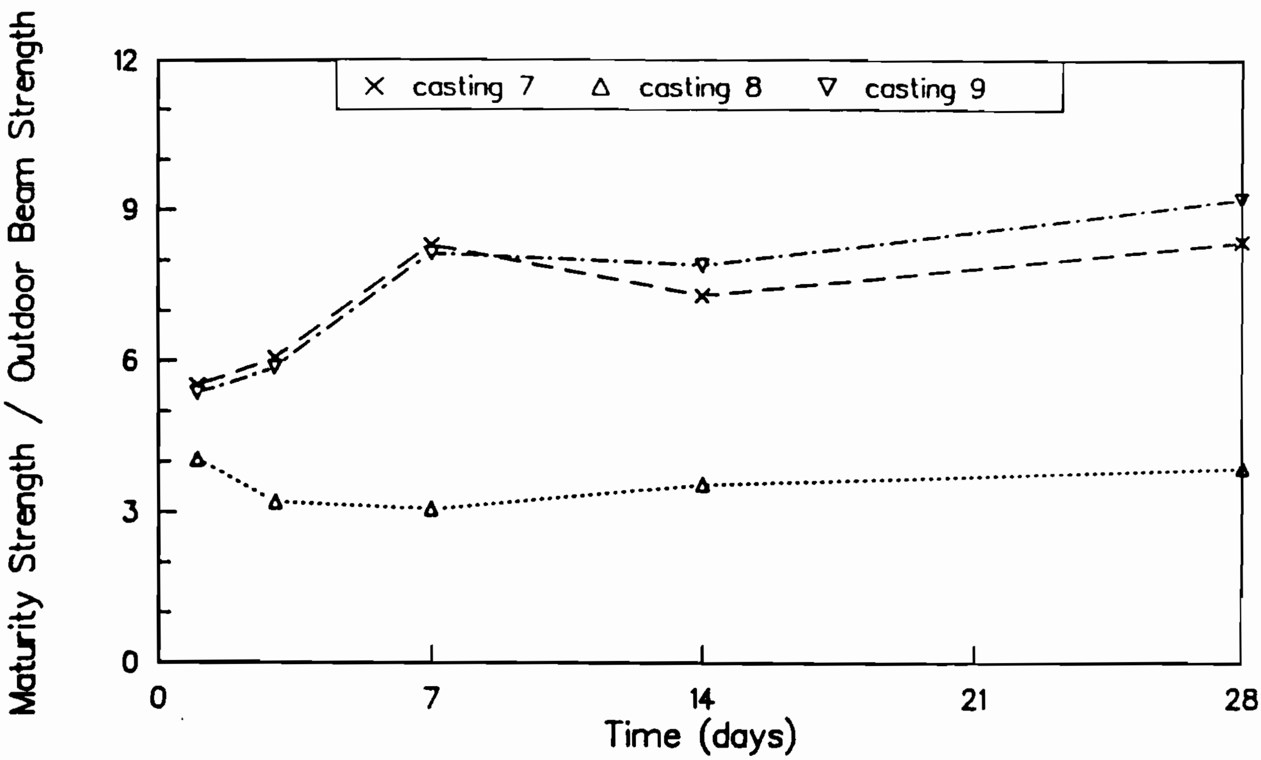


**Fig C-41. Ratio of maturity method strengths to flexural strengths of outdoor-cured beams for castings 7, 8, and 9. Maturity curve based on outdoor-cured cylinders.**

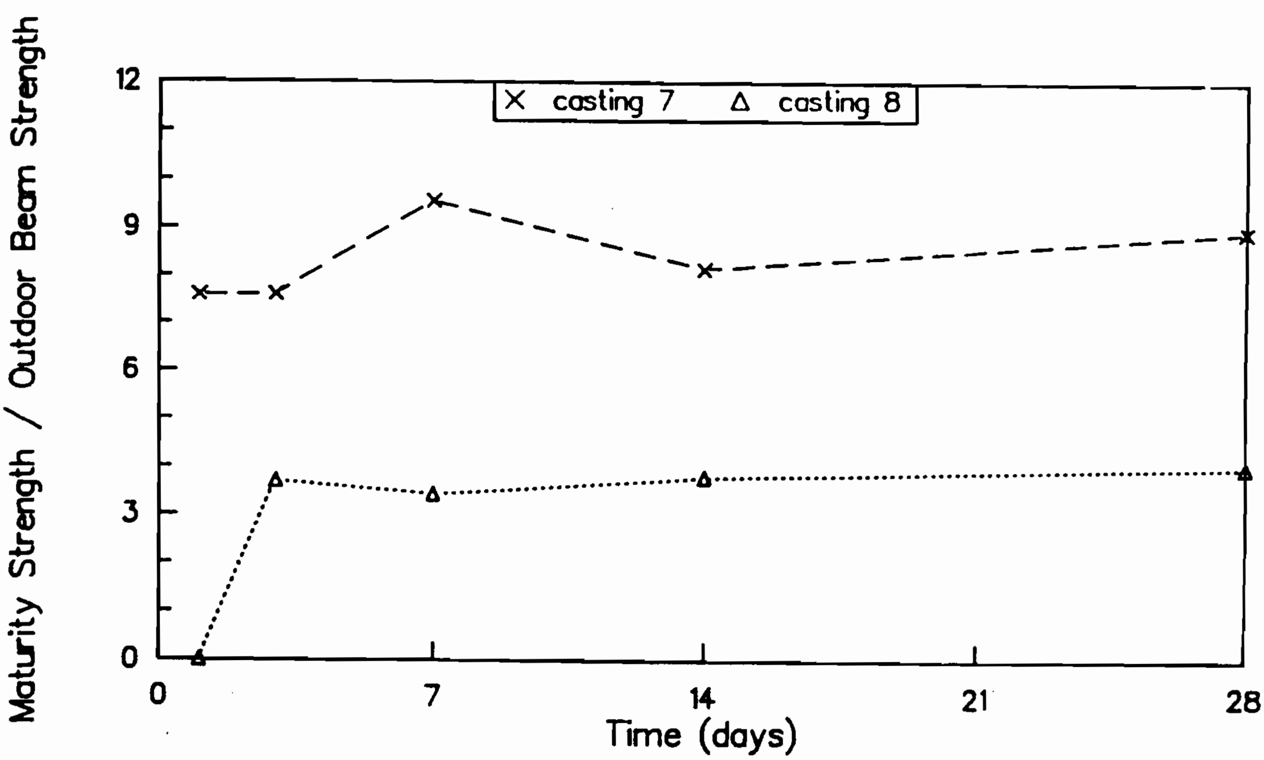


**Fig C-42. Ratio of maturity method strengths to flexural strengths of outdoor-cured beams for castings 7 and 8. Maturity curve based on cores drilled from slabs.**

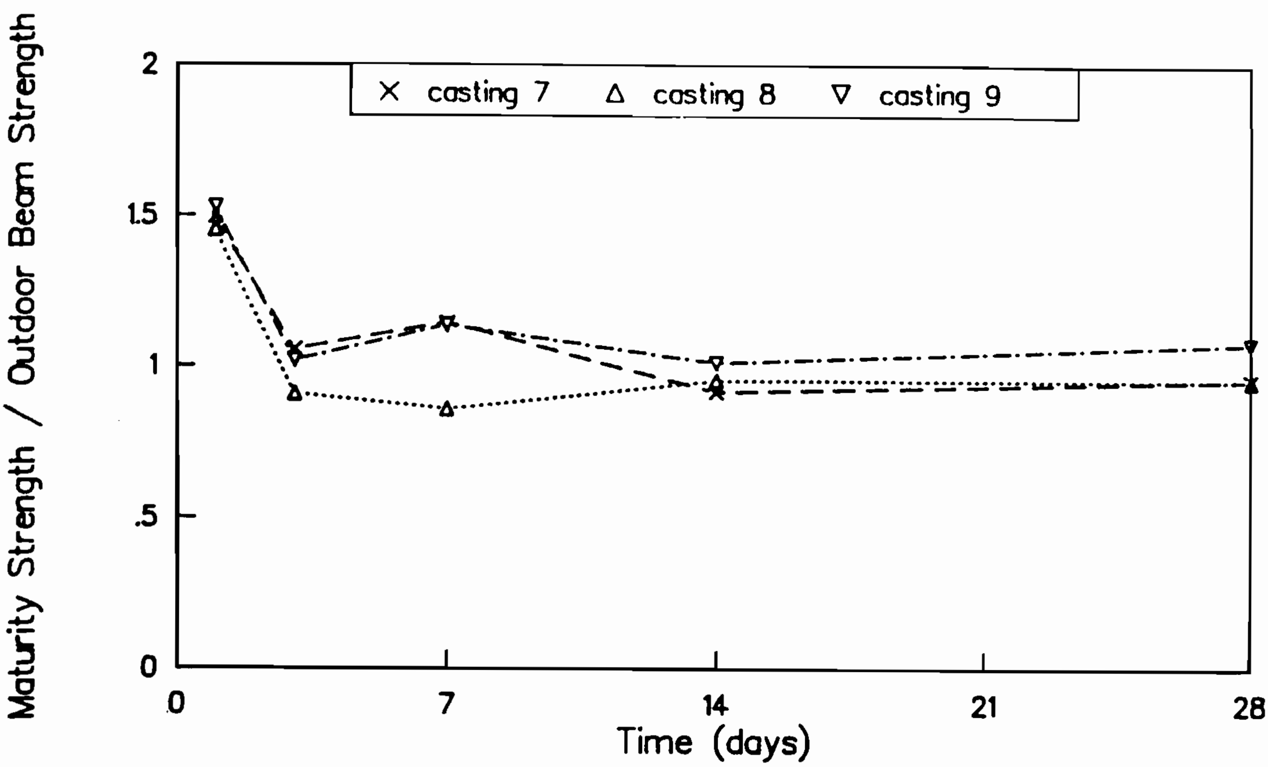


**Fig C-43. Ratio of maturity method strengths to flexural strengths of outdoor-cured beams for castings 7, 8, and 9. Maturity curve based on wetroom-cured beams.**

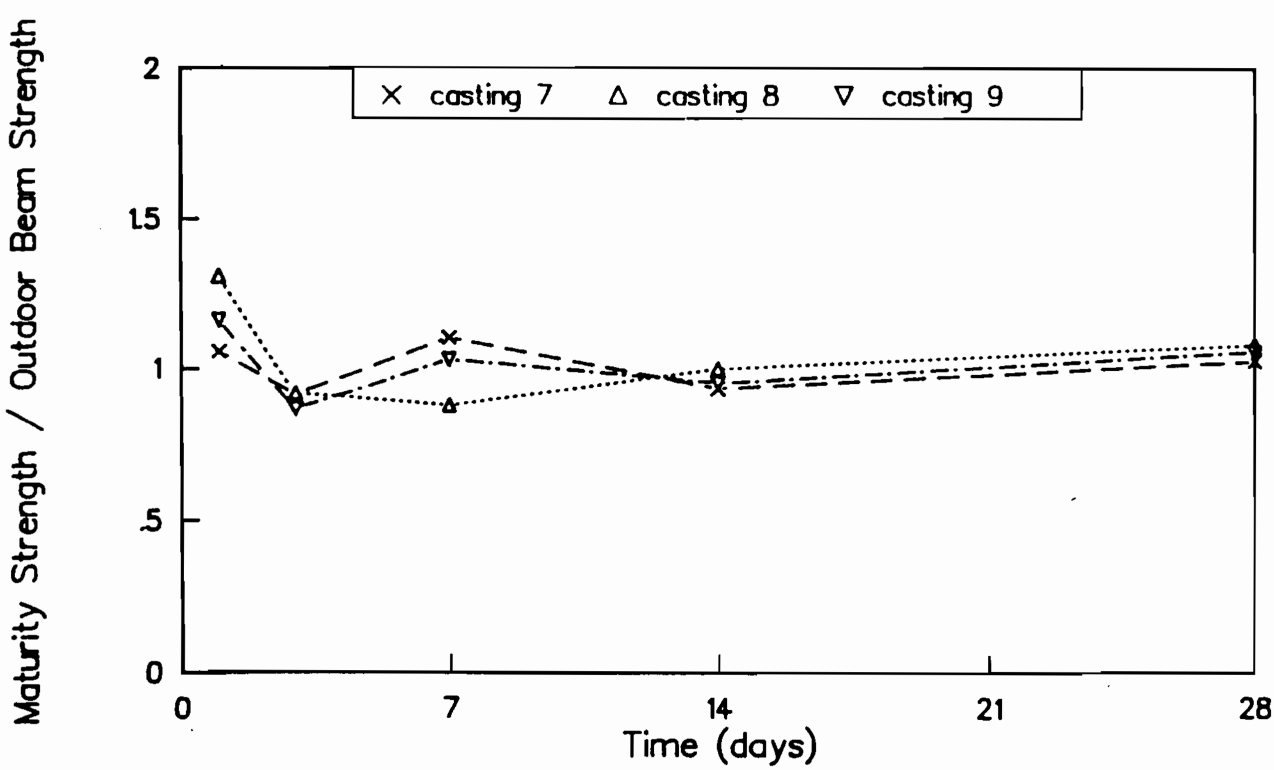


**Fig C-44. Ratio of maturity method strengths to flexural strengths of outdoor-cured beams for castings 7, 8, and 9. Maturity curve based on outdoor-cured beams.**

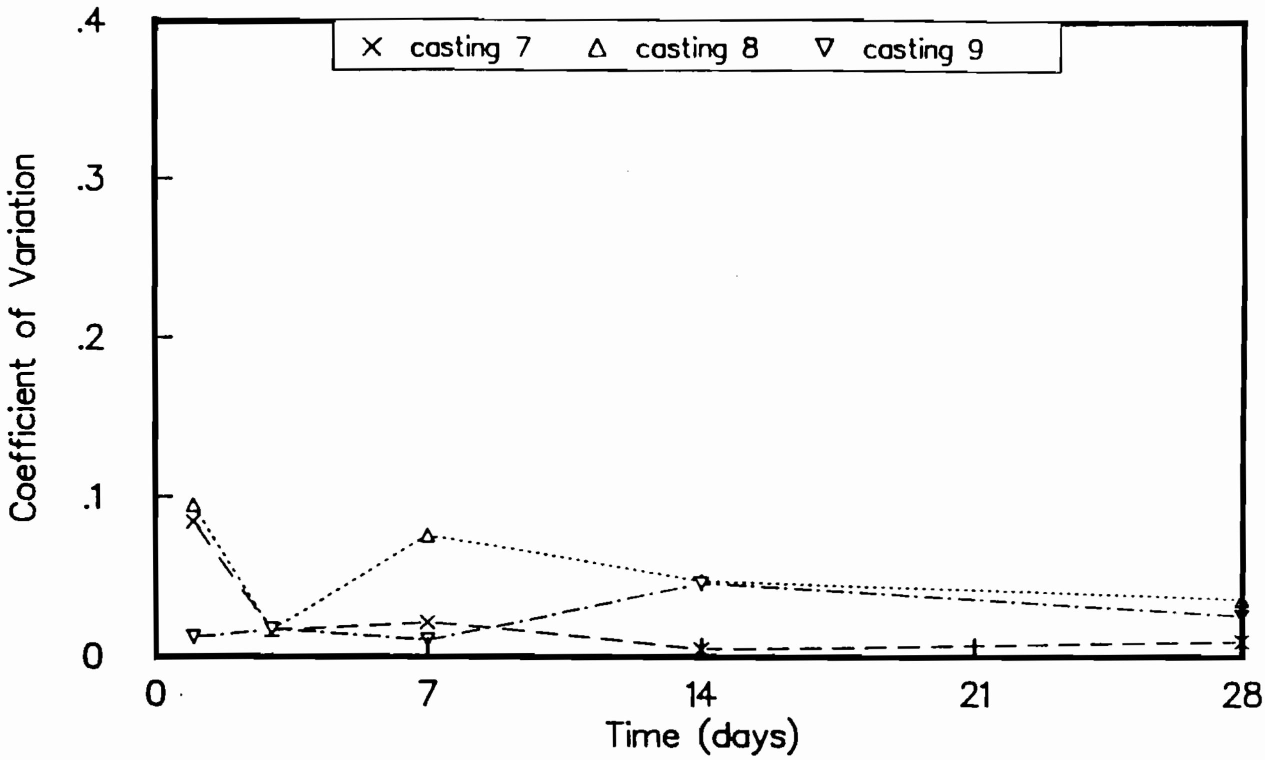


**Fig C-45. Coefficient of variation of wetroom-cured cylinder strengths for castings 7, 8, and 9.**

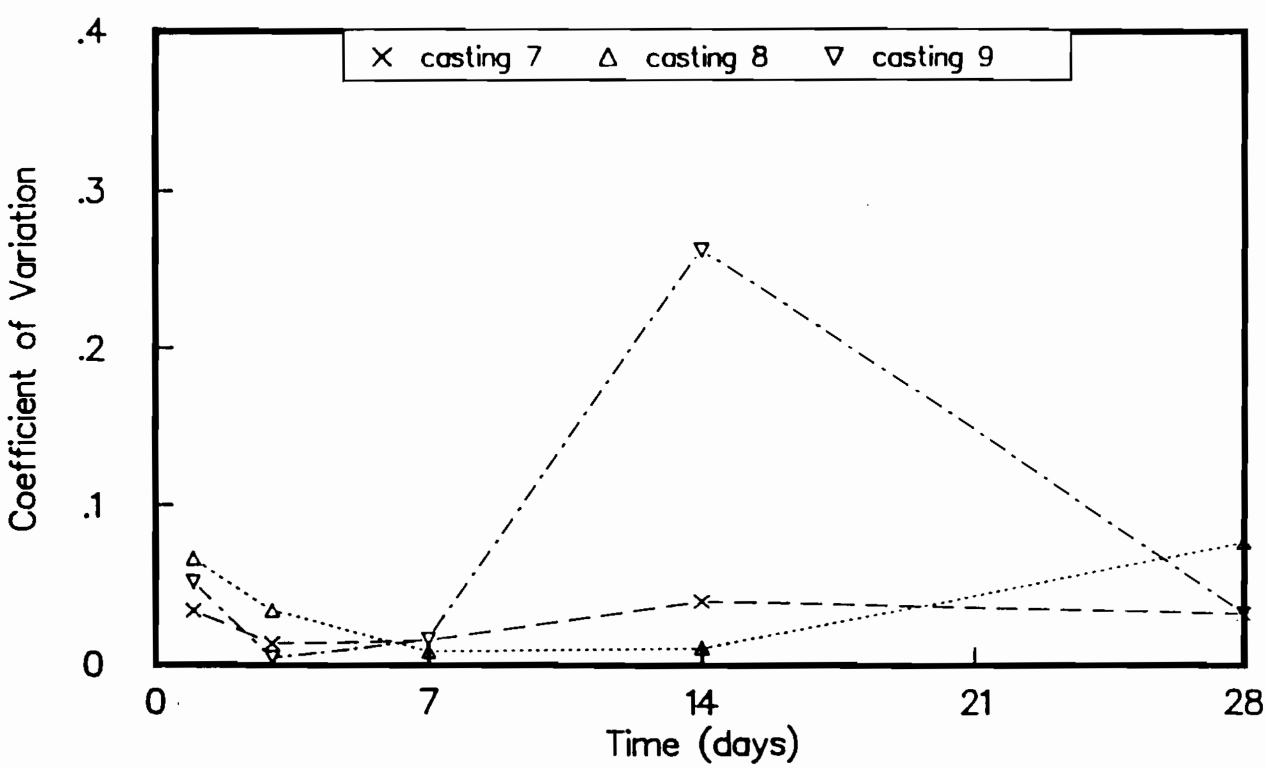


**Fig C-46. Coefficient of variation of outdoor-cured cylinder strengths for castings 7, 8, and 9.**

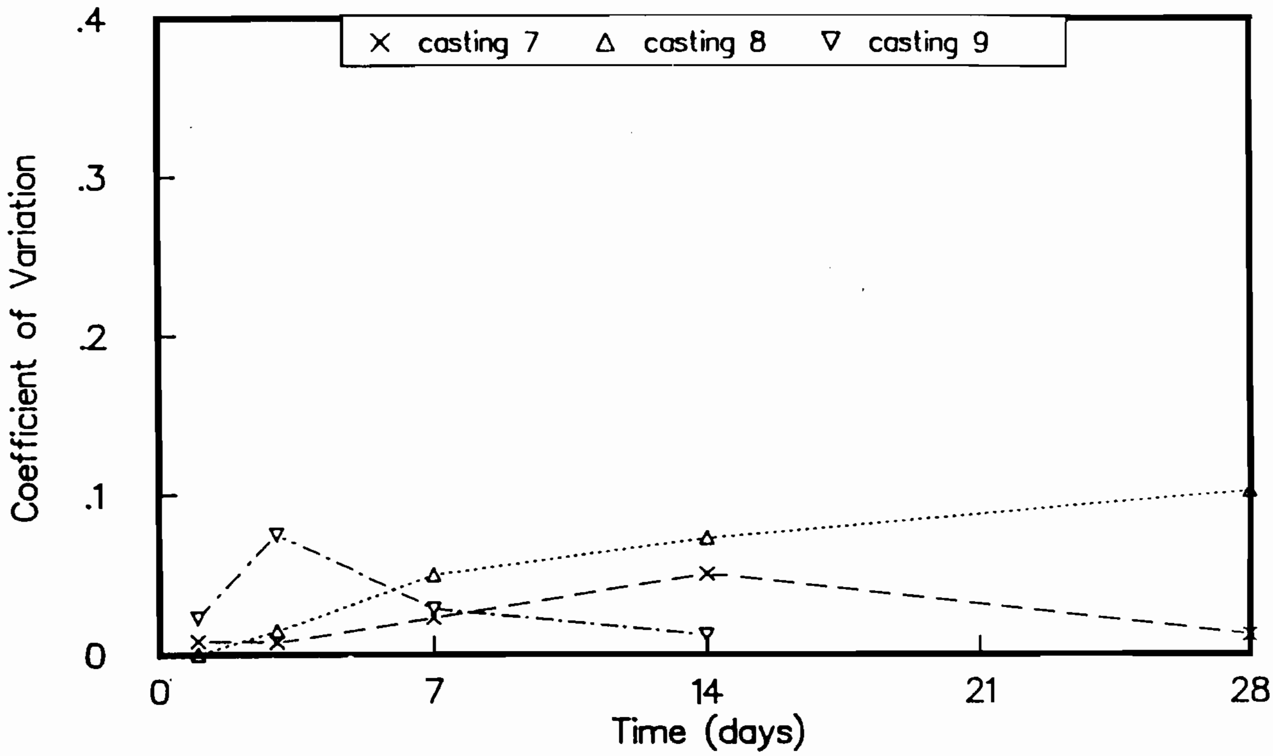


**Fig C-47. Coefficient of variation of cores drilled from slabs for castings 7, 8, and 9.**

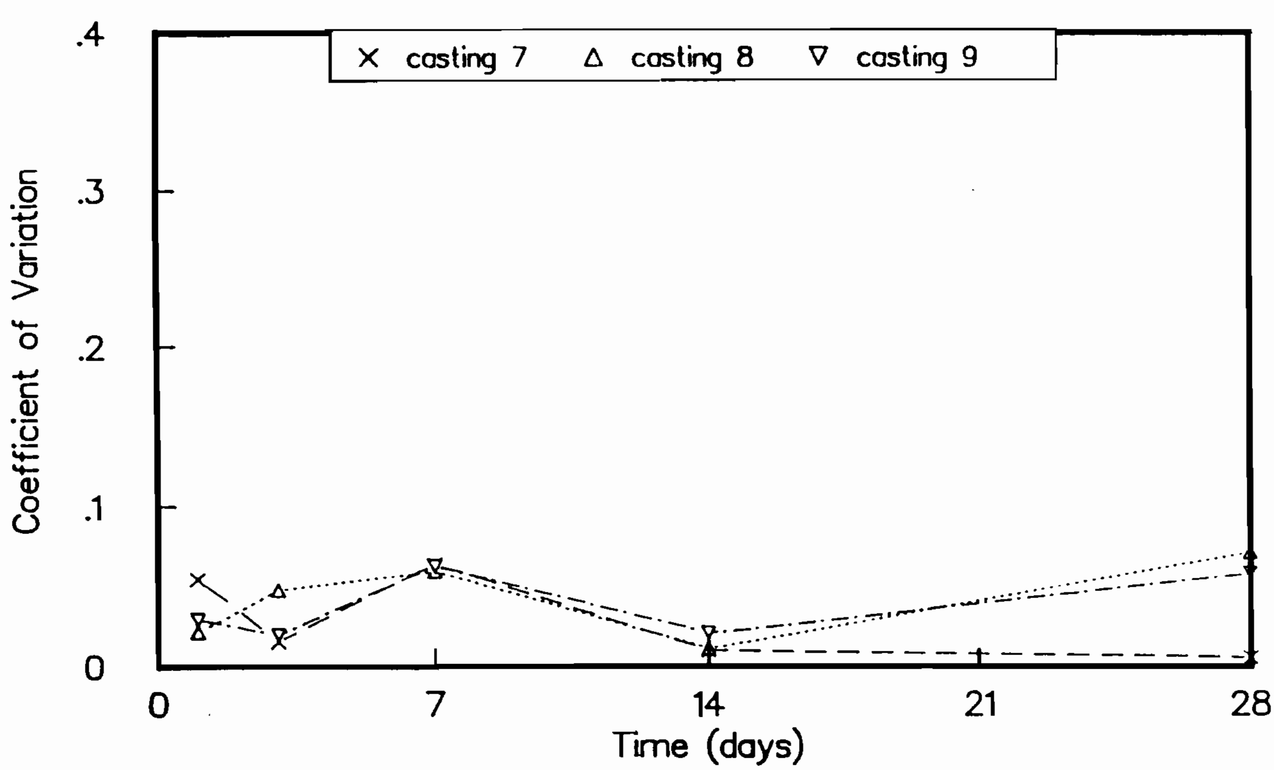


**Fig C-48. Coefficient of variation of flexural strengths of wetroom-cured beams for castings 7, 8, and 9.**

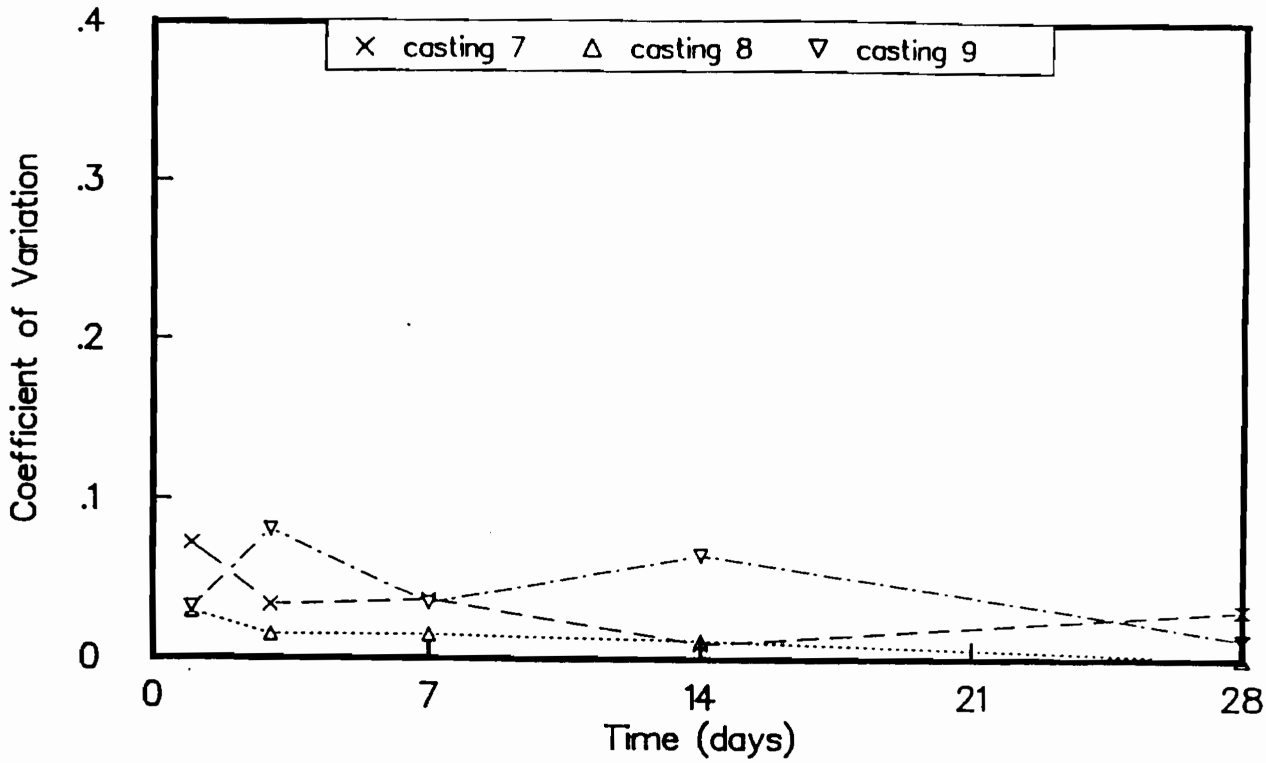


**Fig C-49. Coefficient of variation of flexural strength of outdoor-cured beams for castings 7, 8, and 9.**

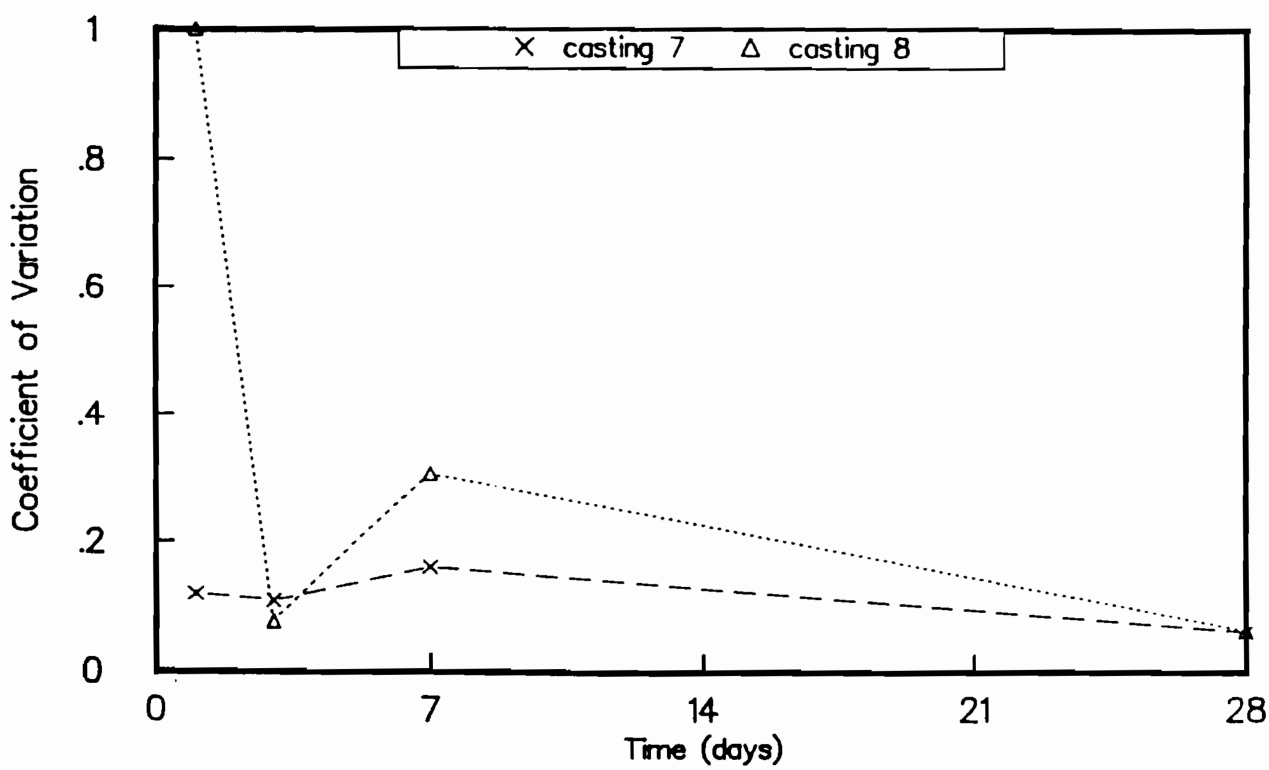


**Fig C-50. Coefficient of variation of "Lok-Test" pullout strengths for castings 7 and 8.**

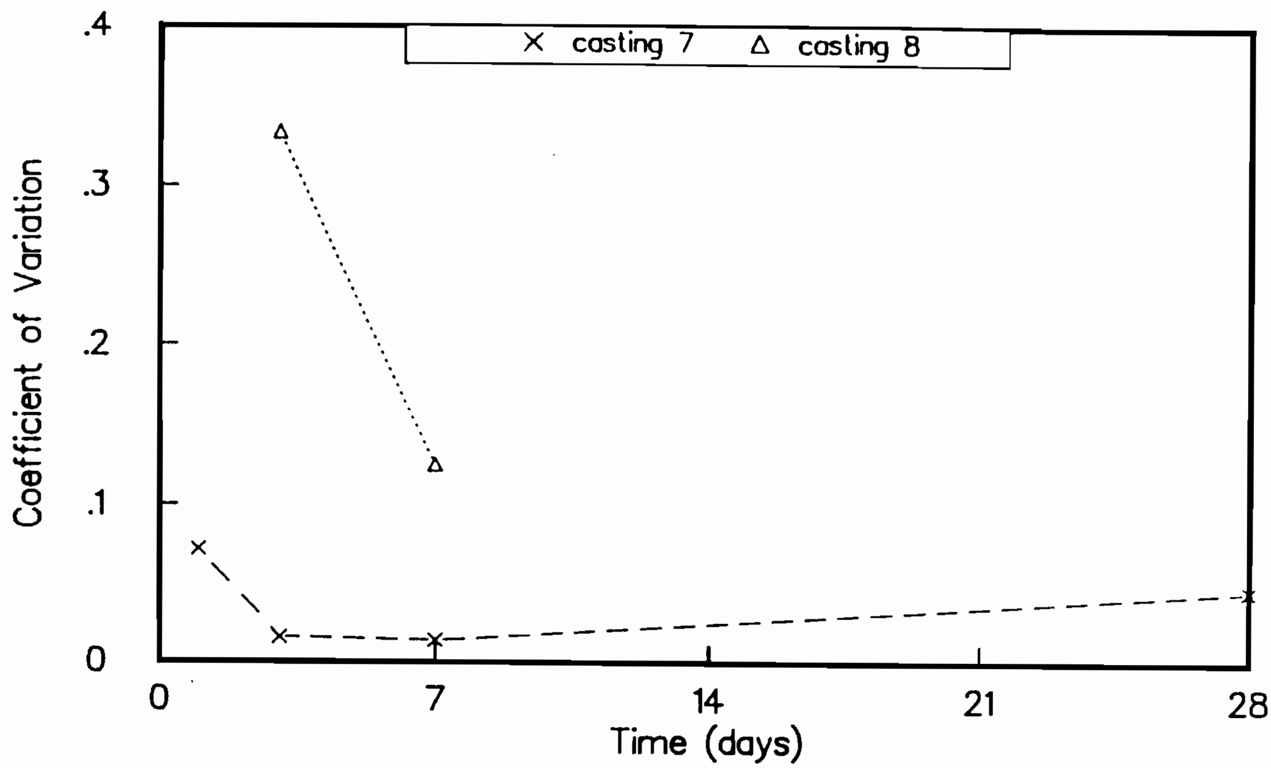


**Fig C-51. Coefficient of variation of Windsor probe penetration resistance strengths for castings 7 and 8.**

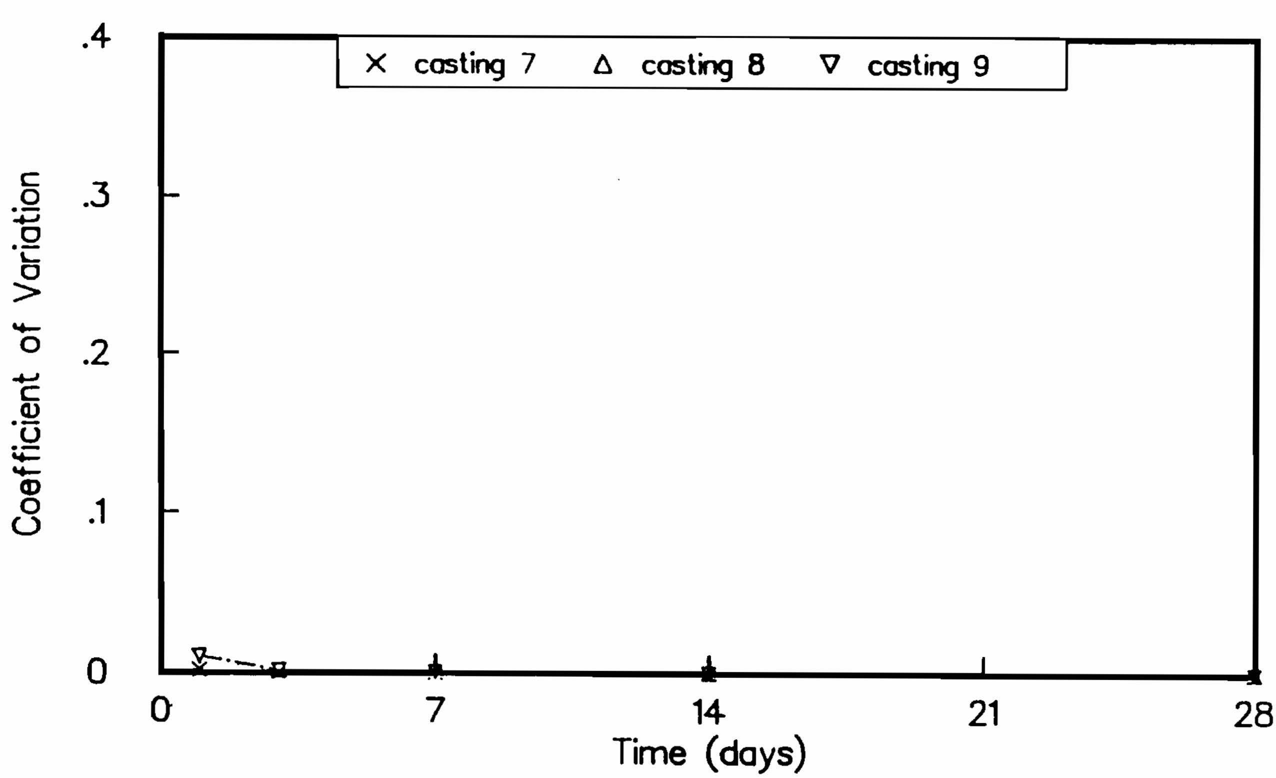


**Fig C-52. Coefficient of variation of maturity method strengths for castings 7, 8, and 9. Maturity curve based on wetroom-cured cylinders.**

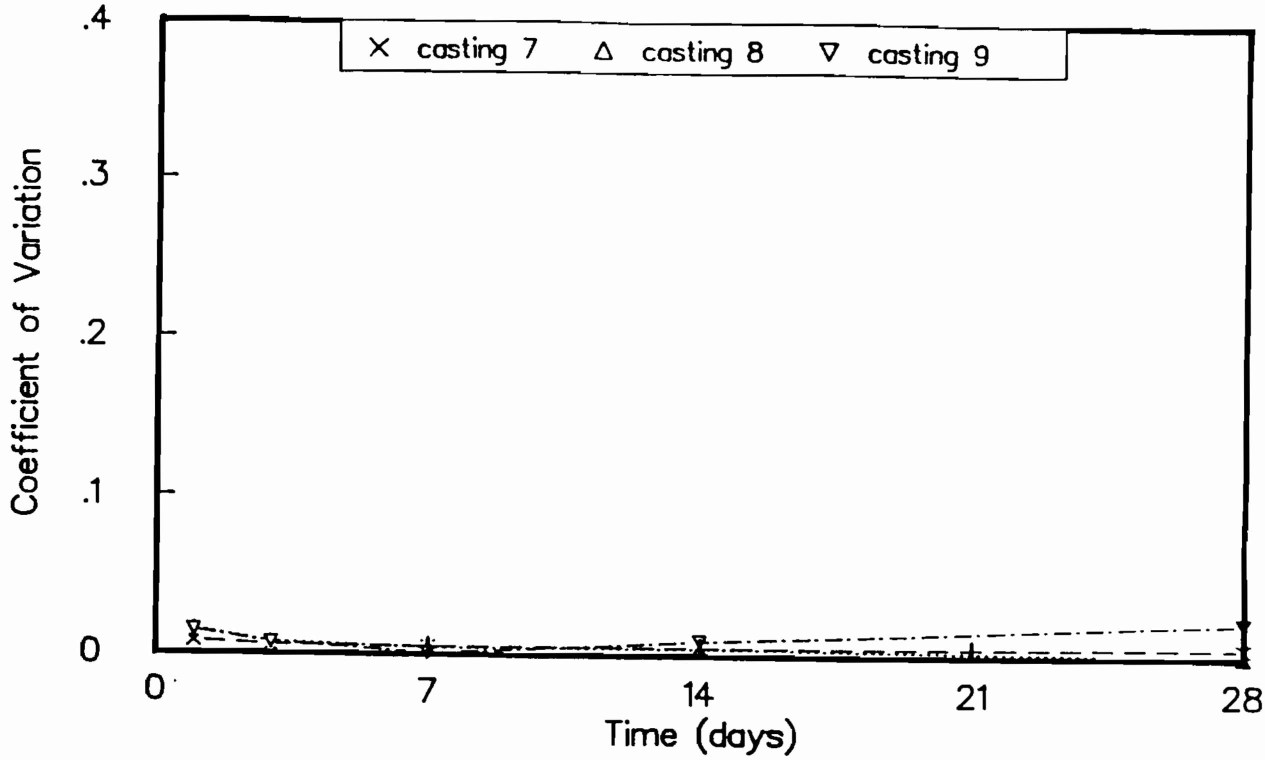


**Fig C-53. Coefficient of variation of maturity method strengths for castings 7, 8, and 9. Maturity curve based on outdoor-cured cylinders.**

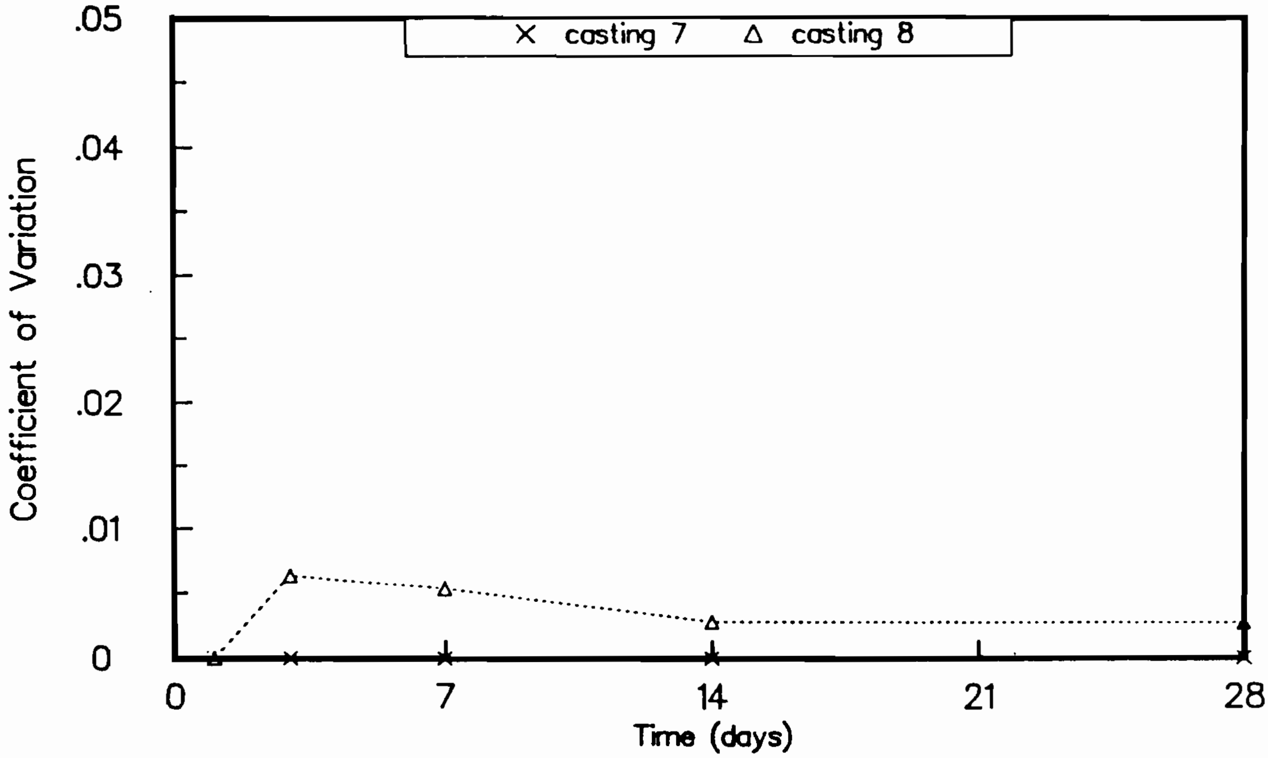


**Fig C-54. Coefficient of variation of maturity method strengths for castings 7 and 8. Maturity curve based on cores drilled from slabs.**

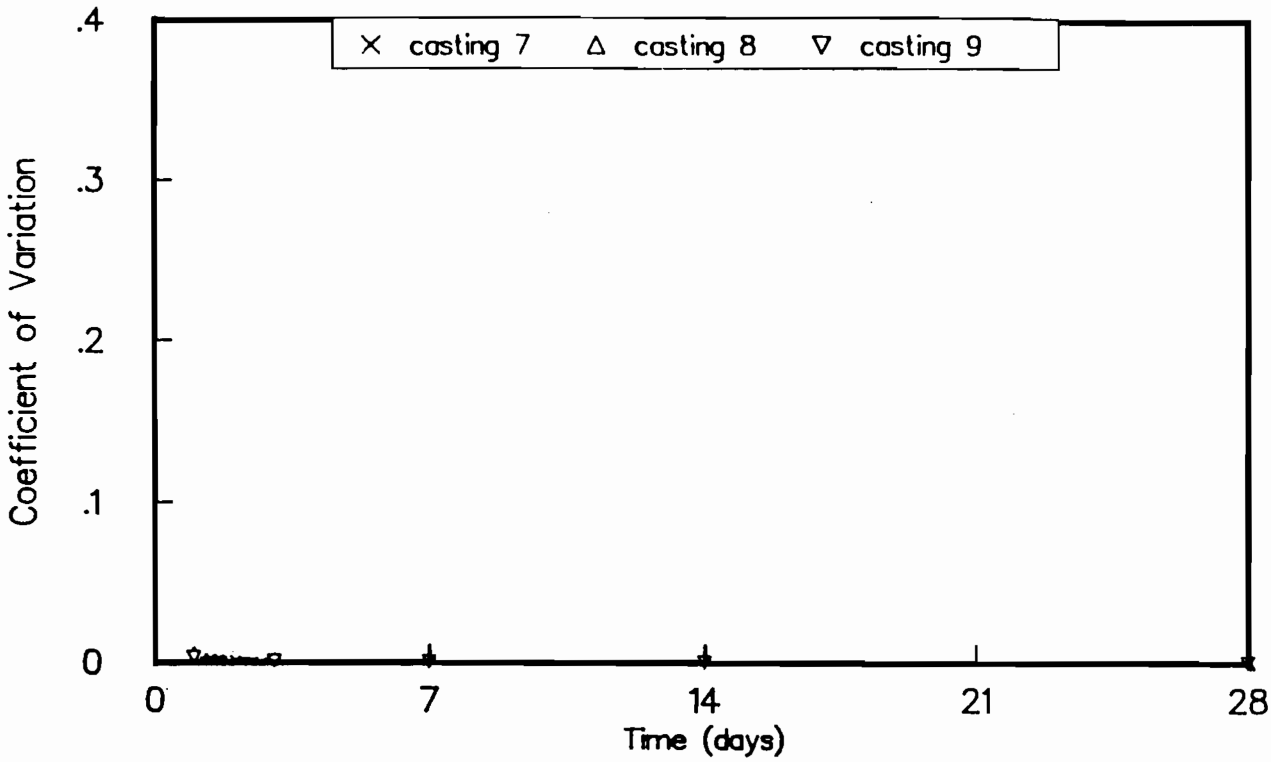


**Fig C-55. Coefficient of variation of maturity method strengths for casting 7. Maturity curve based on wetroom-cured beams.**

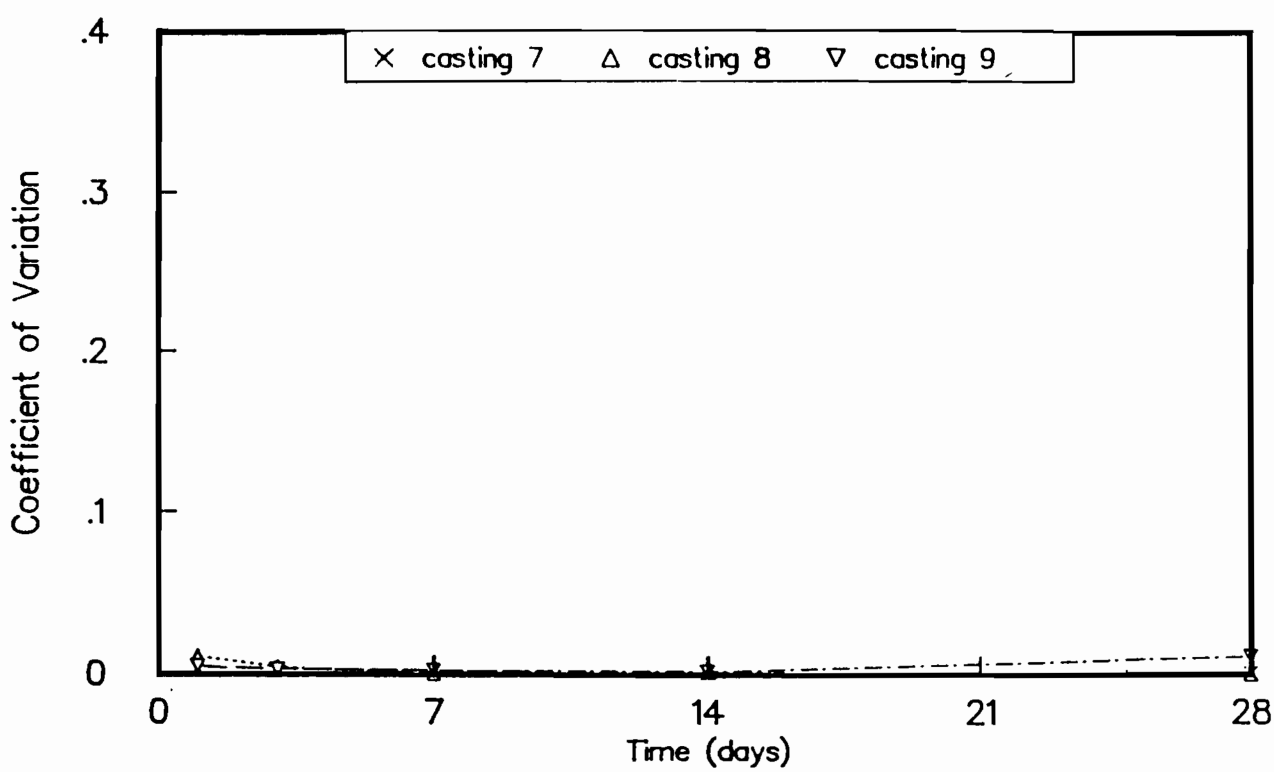


**Fig C-56. Coefficient of variation of maturity method strengths for castings 7, 8, and 9. Maturity curve based on outdoor-cured beams.**

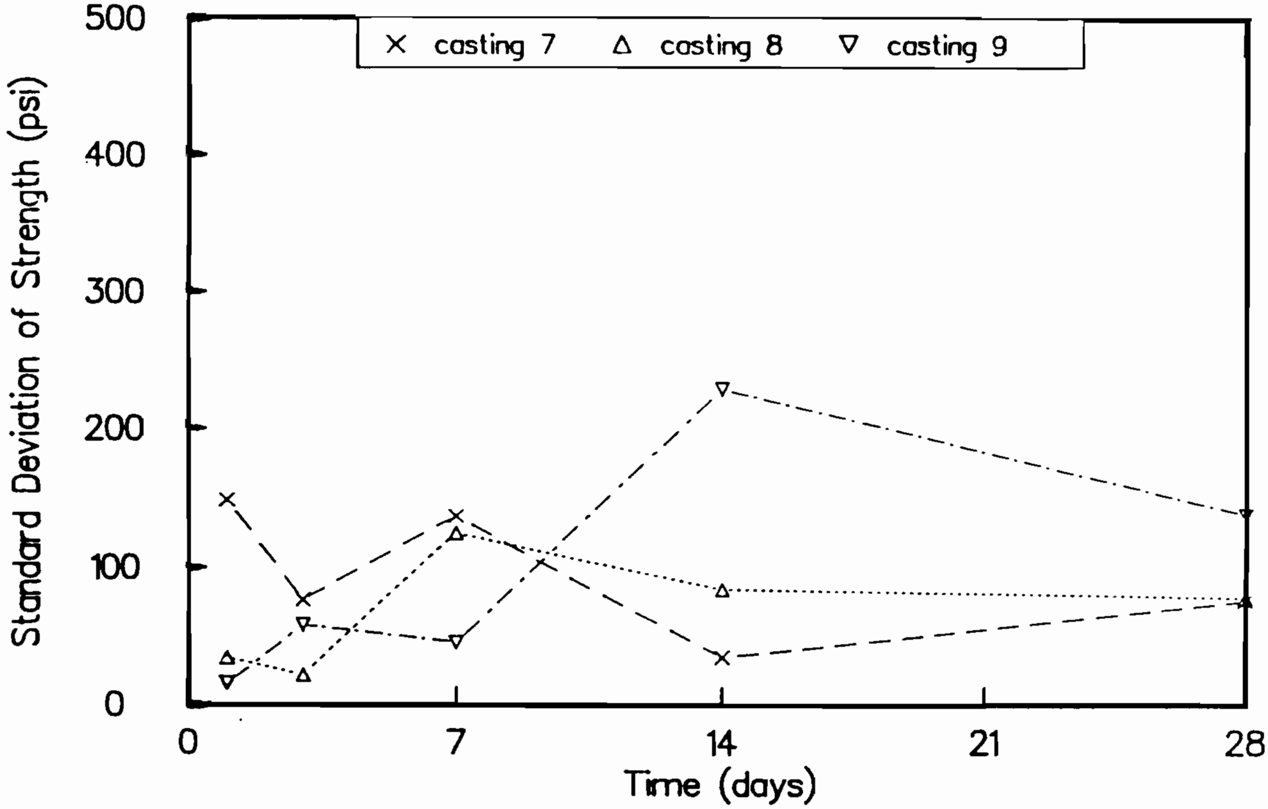


**Fig C-57. Standard deviation of wetroom-cured cylinder strengths for castings 7, 8, and 9.**

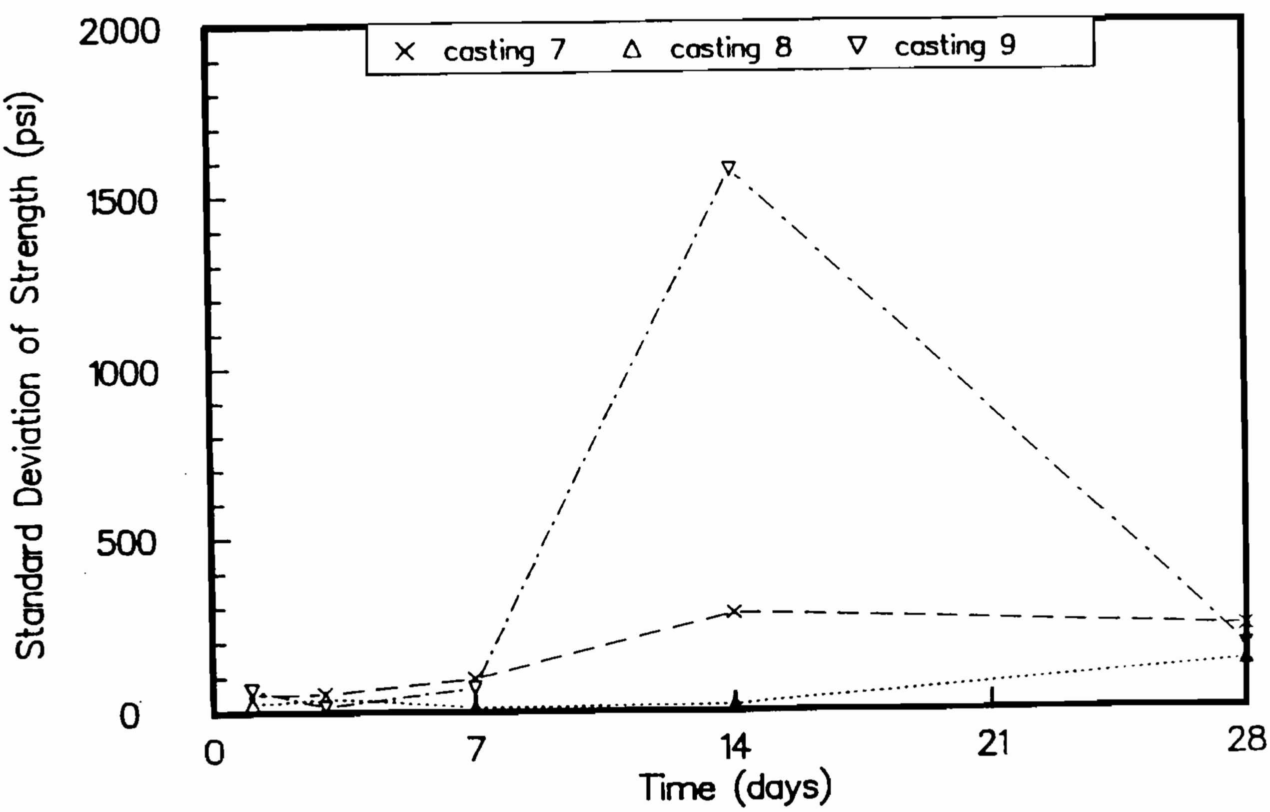


**Fig C-58. Standard deviation of outdoor-cured cylinder strengths for castings 7, 8, and 9.**

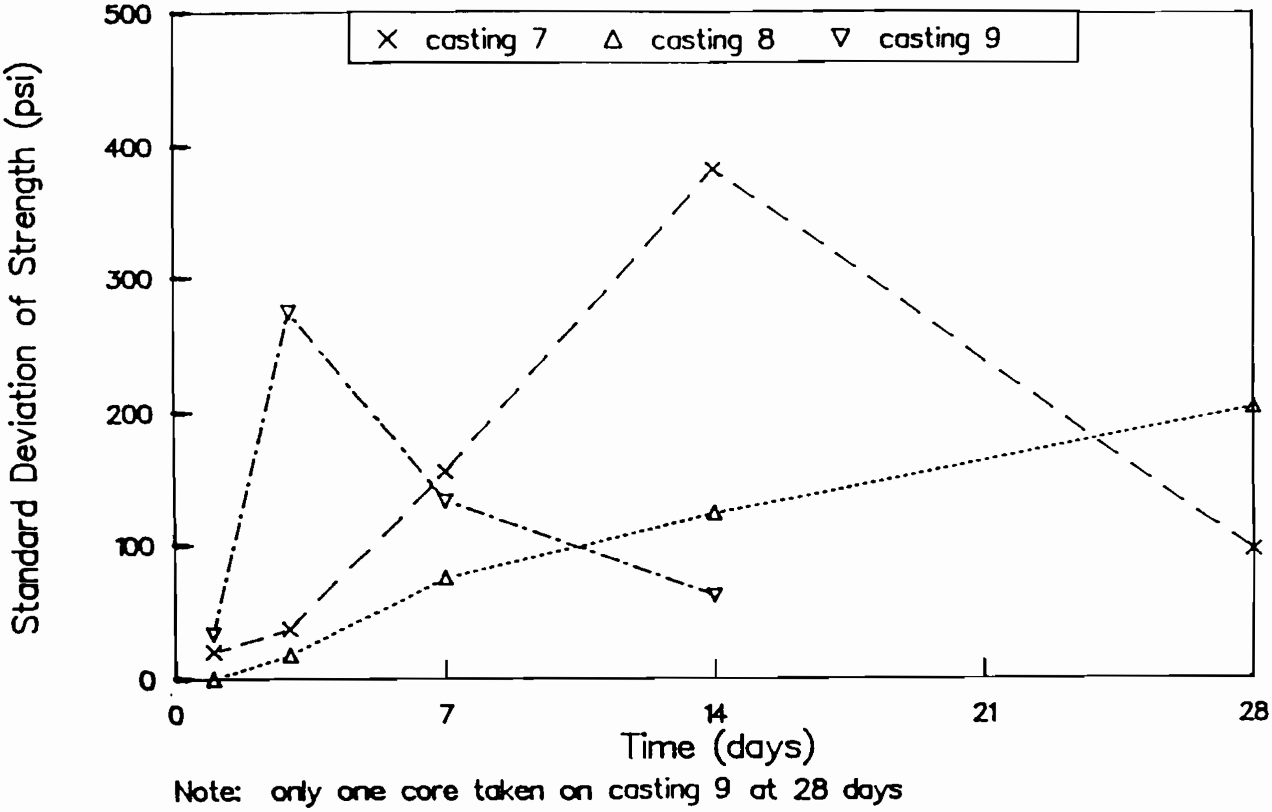


**Fig C-59. Standard deviation of cores drilled from slabs for castings 7, 8, and 9.**

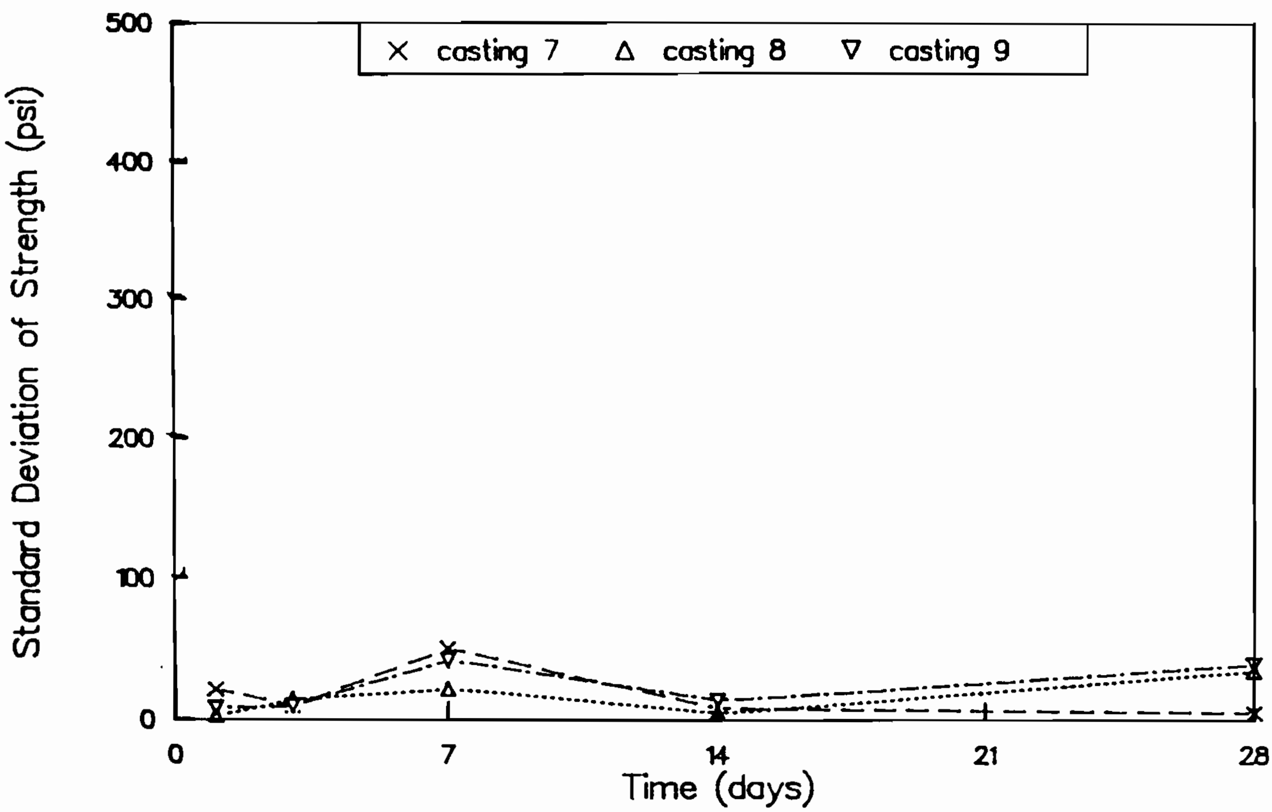


**Fig C-60. Standard deviation of flexural strength of wetroom-cured beams for castings 7, 8, and 9.**

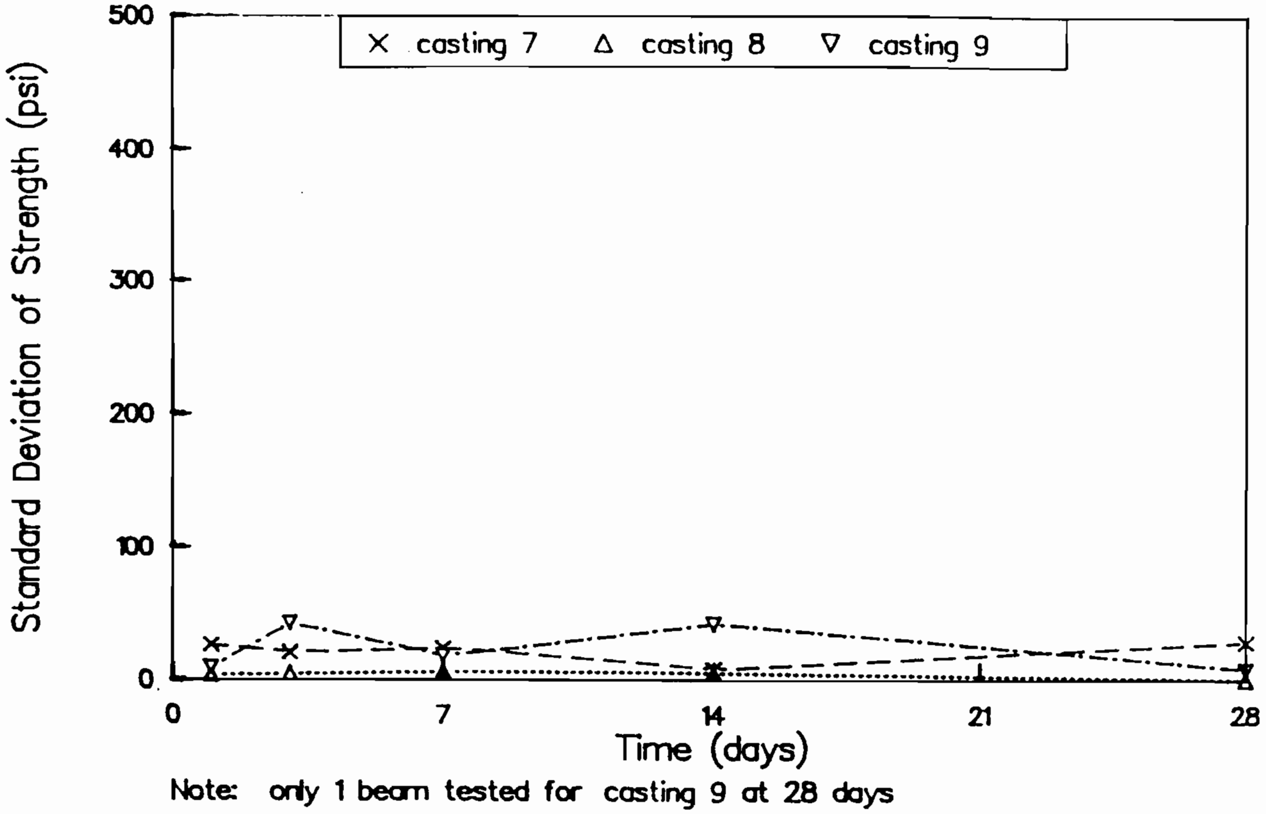


Fig C-61. Standard deviation of flexural strength of outdoor-cured beams for castings 7, 8, and 9.

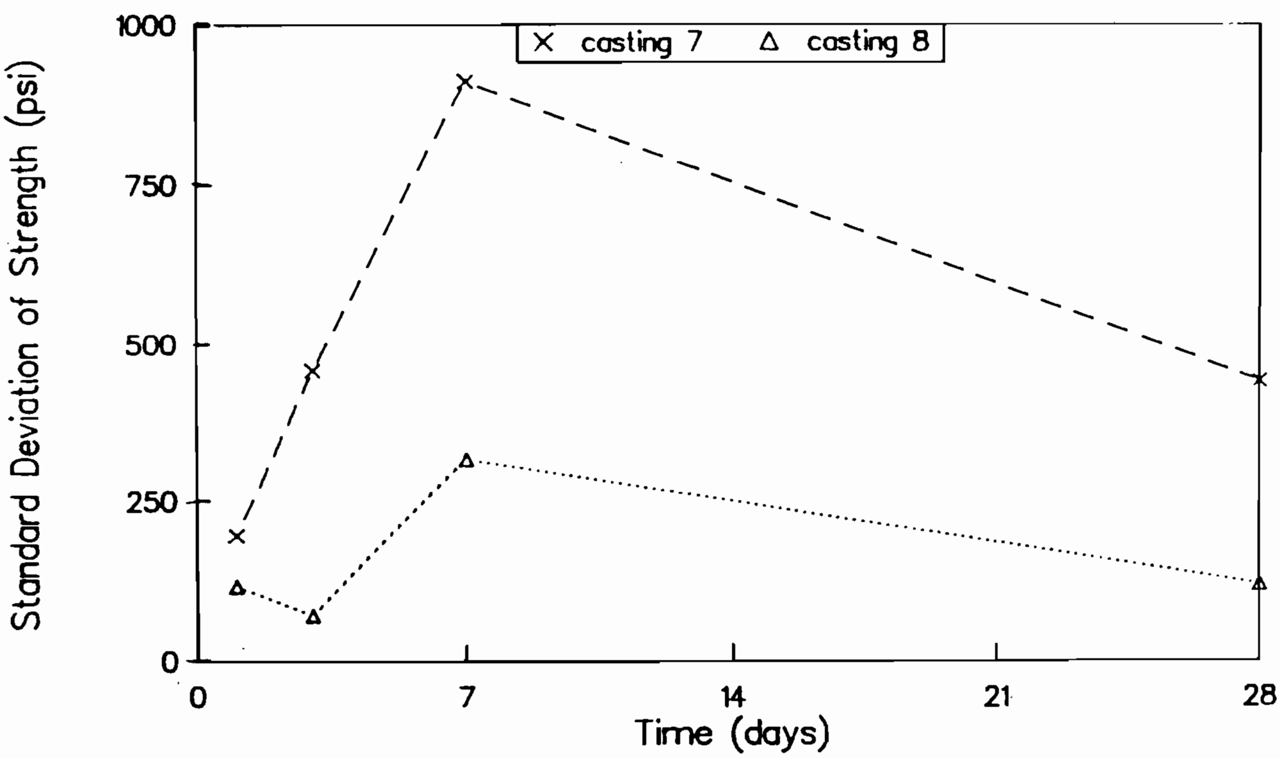


Fig C-62. Standard deviation of "Lok-Test" pullout strengths for castings 7 and 8.

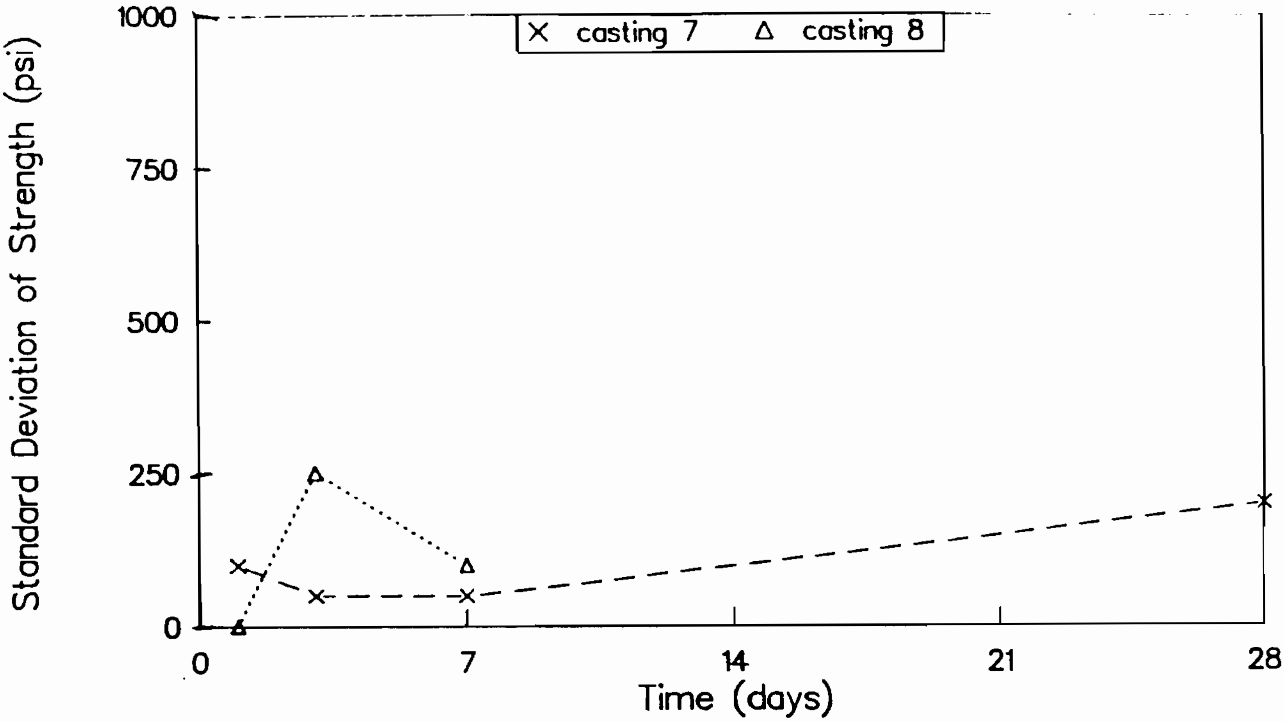


**Fig C-63. Standard deviation of Windsor probe penetration resistance strengths for castings 7 and 8.**

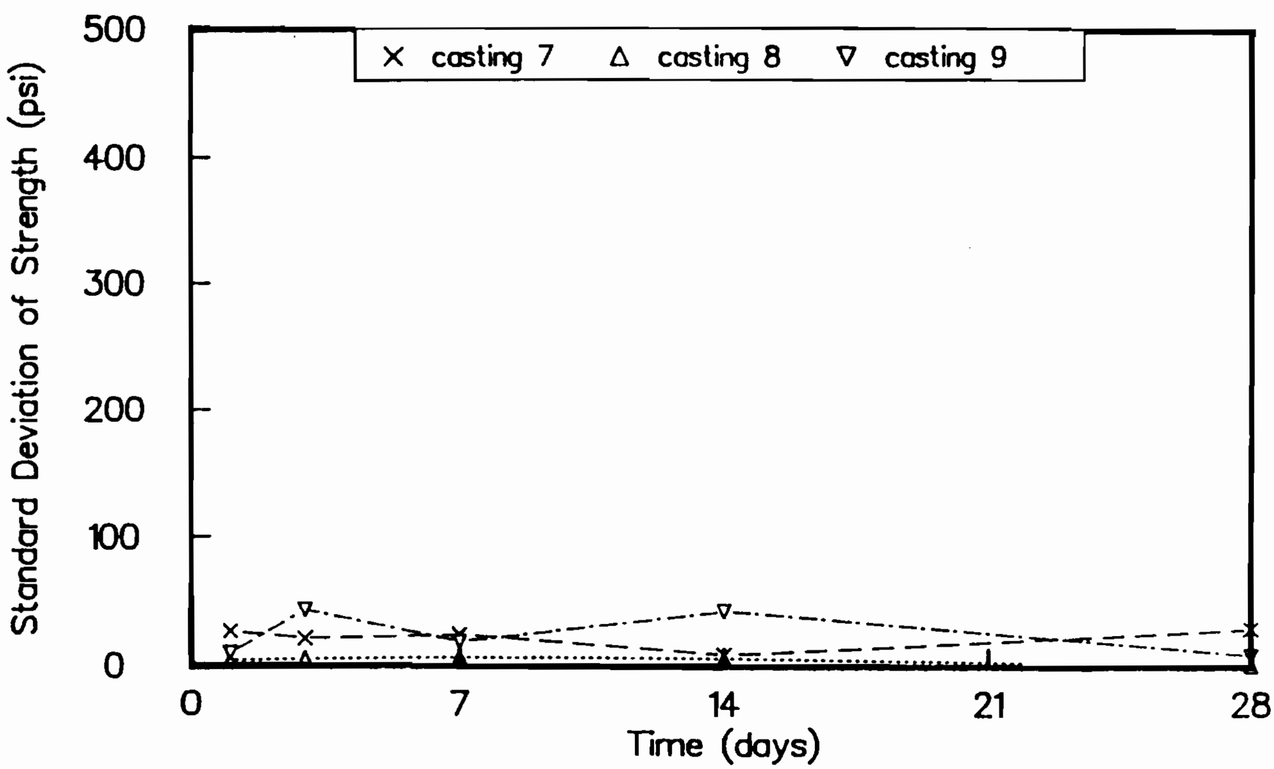


**Fig C-64. Standard deviation of maturity method strengths for castings 7, 8, and 9. Maturity curve based on wetroom-cured cylinders.**

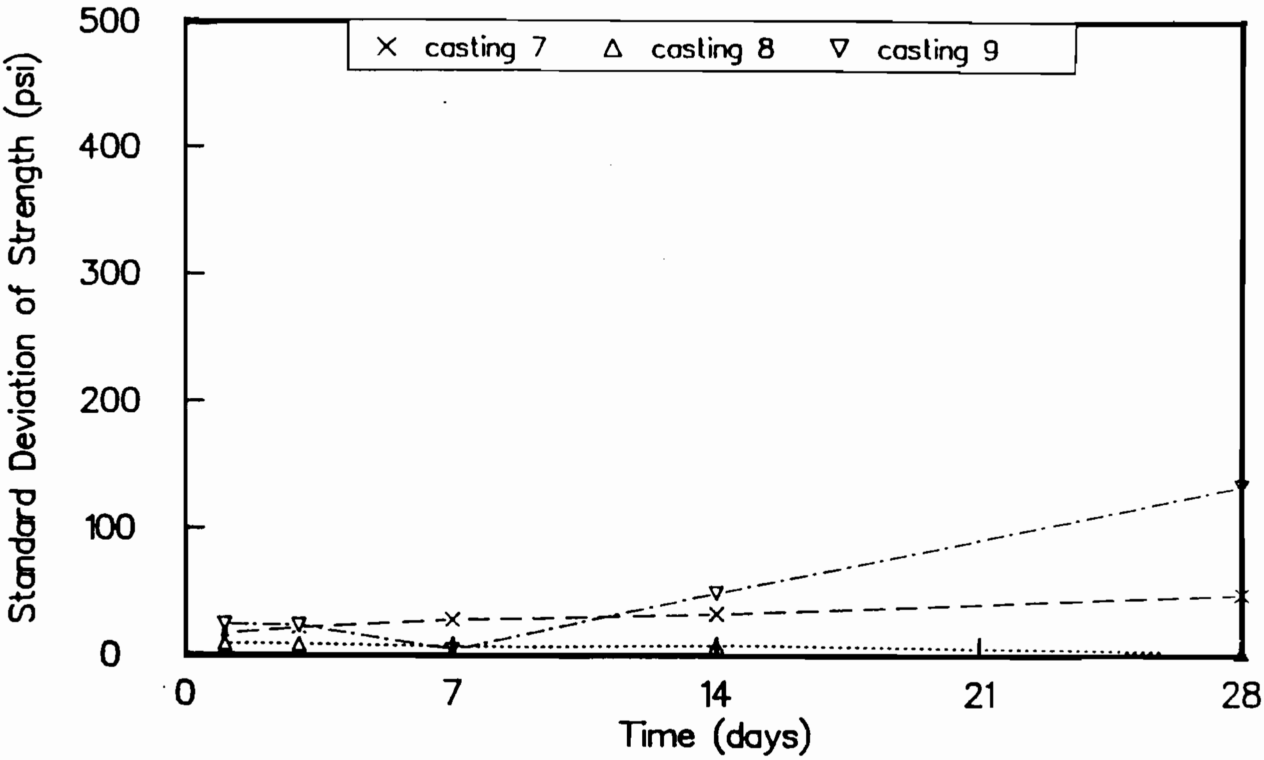


**Fig C-65. Standard deviation of maturity method strengths for castings 7, 8, and 9. Maturity curve based on outdoor-cured cylinders.**

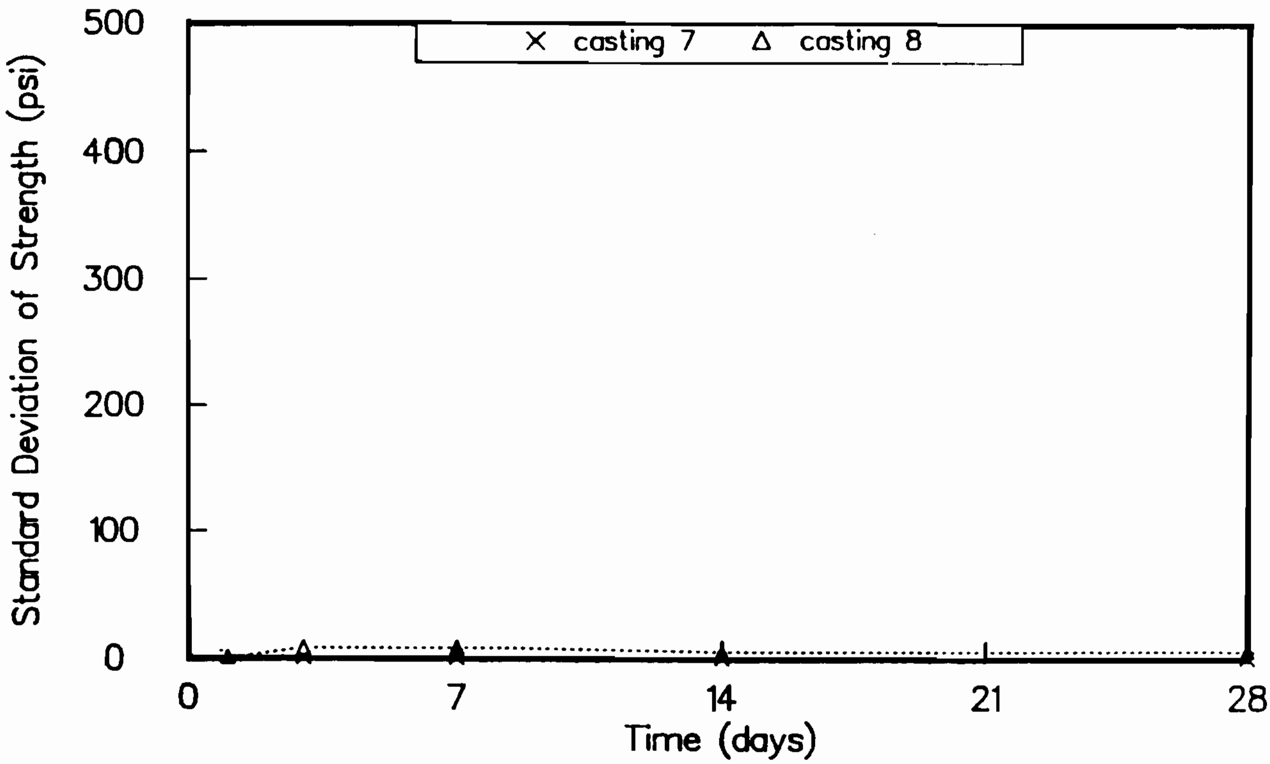


**Fig C-66. Standard deviation of maturity method strengths for castings 7 and 8. Maturity curve based on cores drilled from slabs.**

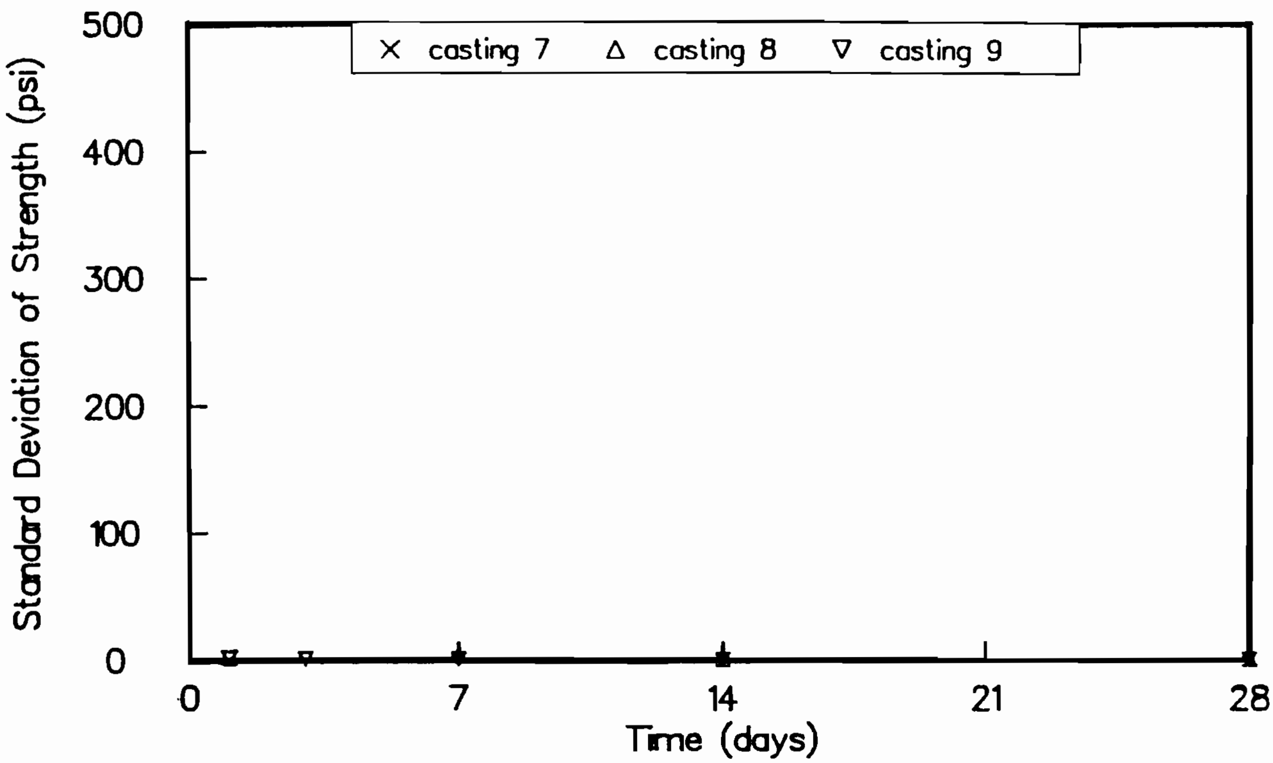


**Fig C-67. Standard deviation of maturity method strengths for castings 7, 8, and 9. Maturity curve based on wetroom-cured beams.**

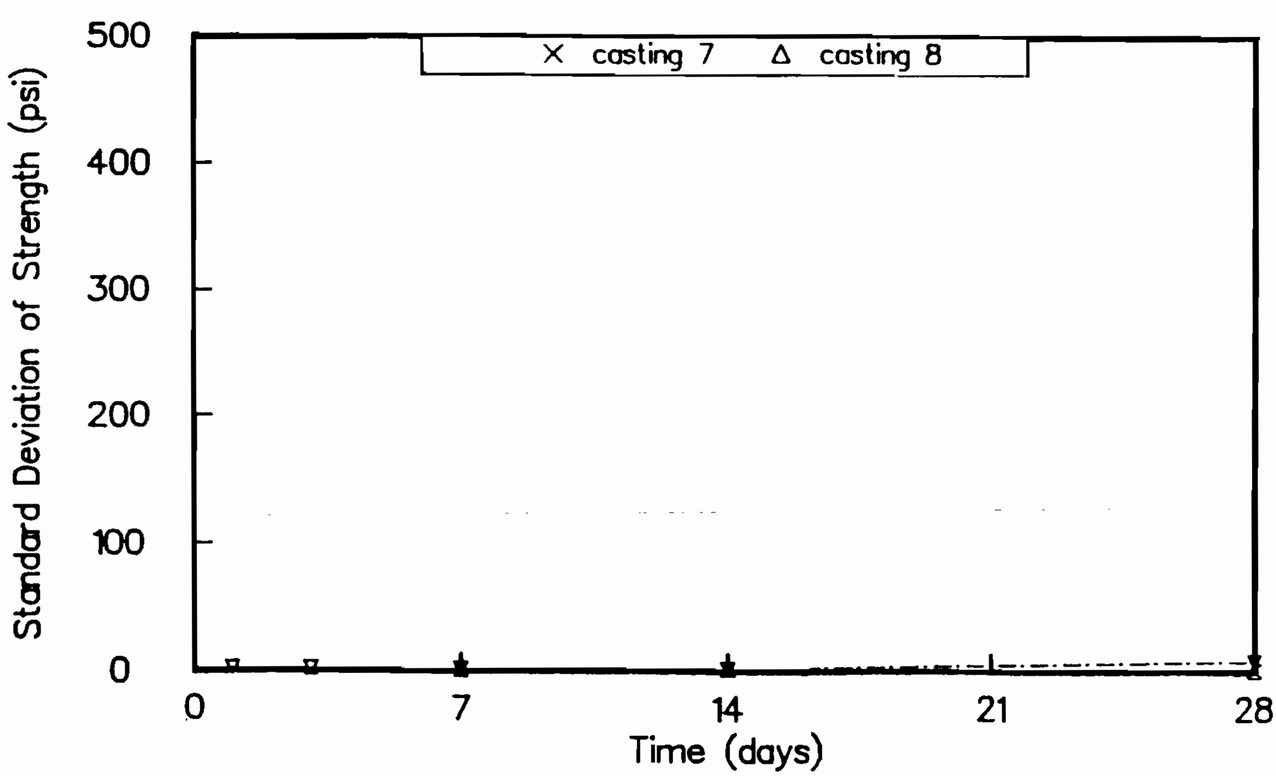


**Fig C-68. Standard deviation of maturity method strengths for castings 7 and 8. Maturity curve based on outdoor-cured beams.**

# APPENDIX D

## F-TEST AND BARTLETT'S TEST

According to Carino, Lew, and Volz,[20] Bartlett's Test must be performed before an F-Test can be performed. The purpose of Bartlett's Test is to establish that the variances of the regression lines of the groups being compared are not significantly different.

In Bartlett's test, a value of chi-square ($\chi^2$) is calculated and compared with tabulated values for the same number of degrees of freedom at a given confidence level. If the calculated value exceeds the tabulated value, the variances are not homogenous and the data are not suitable for analysis by the F-test. The tabulated values were taken from Appendix A of Snedecor and Cochran,[86] and the calculations follow.

**Casting 1: y = mx + b = 2,090 (Log Maturity) - 4,766**

| Actual Strength (1) | Maturity (2) | Log Mat (3) | Calc Strength (4) | Deviation (5) = (1) - (4) | Square of Deviation (6) = (5) x (5) |
|---|---|---|---|---|---|
| 1,000 | 636 | 2.80 | 1,093 | 93 | 8,690 |
| 2,290 | 2,106 | 3.32 | 2,180 | -110 | 12,090 |
| 3,050 | 5,136 | 3.71 | 2,989 | -61 | 3,695 |
| 3,580 | 9,981 | 4.00 | 3,592 | 12 | 151 |
| 4,170 | 20,256 | 4.31 | 4,235 | 65 | 4,186 |
| | | | | | 28,817 = $D^2$ |

**Casting 2: 1,868.00 = m** **-4,157.00 = b**

| Actual Strength | Maturity | Log Mat | Calc Strength | Deviation | Square of Deviation |
|---|---|---|---|---|---|
| 1,000 | 616 | 2.79 | 1,053.94 | 53..94 | 2,909.18 |
| 2,060 | 2,011 | 3.30 | 2,013.77 | -46.23 | 2,136.87 |
| 2,690 | 5,056 | 3.70 | 2,761.71 | 71.71 | 5,142.55 |
| 3,570 | 10,096 | 4.00 | 3,322.75 | -247.25 | 61,132.08 |
| 3,710 | 20,176 | 4.30 | 3,884.43 | 174.43 | 30,426.49 |
| | | | | | 101,747.17 = $D^2$ |
| | | | | | Log = 4.46 |

**Casting 3: 2,182.00 = m** **-4,954.00 = b**

| Actual Strength | Maturity | Log Mat | Calc Strength | Deviation | Square of Deviation |
|---|---|---|---|---|---|
| 1,280 | 702 | 2.85 | 1,256.71 | -23.29 | 542.54 |
| 2,500 | 2,100 | 3.32 | 2,295.08 | -204.92 | 41,991.18 |
| 2,770 | 4,965 | 3.70 | 3,110.50 | 340.50 | 115,937.40 |
| 3,670 | 10,035 | 4.00 | 3,777.31 | 107.31 | 11,515.63 |
| 4,650 | 20,145 | 4.30 | 4,437.69 | -212.31 | 45,074.27 |
| | | | | | 215,061.02 = $D^2$ |

| Sample Number | Sample Size | Sum of Sq's of Devi | DOF | Est Var | Log of Var | DOF x Log | 1/DOF |
|---|---|---|---|---|---|---|---|
| 1 . . . k | $n_i$ | $\sum_{1}^{n_i} (x - \bar{x}_i)^2$ | | | | | |
| 1 | 5 | 28,817 | 4.00 | | | | |
| 2 | 5 | 101,747 | | | | | |
| 3 | 5 | 215,061 | | 5,276 | 4.730 | 18.9 | 0.25 |
| | | | | 5 | 5 | 2 | 00 |
| $\Sigma$ | | 345,625 | 12 | | | | |

$$\bar{S}^2 = \frac{\sum\sum (x - \bar{x}_i)^2}{\sum (n_{i-1})} = \frac{345625}{12} = 28802$$

Now, using equation 14-2 of Snedecor:

$$\chi^2 = 2.3026 \{\log \bar{s}^2 \Sigma (n_i - 1) - \Sigma [ (n_i - 1) \log s_i^2 ] \}$$

Going to table A-7 of Snedecor, or equivalent, using
d. o. f. = k–1 = 2:

$\chi^2 = 3.55 < 5.99$ OK

Null hypothesis is correct at 5% level of significance. That is, there is a 5% chance that these values are *not* homogeneous.

We conclude that these values *are* homogeneous, with 95% certainty.

Bartlett's test — *PASSED*

Bartlett's test has determined that the data are not significantly different. The F-test can proceed as outlined by Carino, Lew, and Volz.[20]

STEP 1. For each group of data determine the sum of squares of deviations from the best-fit curve of that group. See calculations for Bartlett's test.

STEP 2. Add up the sums of the squares of the deviations of the K groups in Step 1. Call this total $D_o^2$. In this case there are three groups, so K = 3.

| *Group* | *Sum Sq's of Devs* |
|---|---|
| 1 | 28800 |
| 2 | 102000 |
| 3 | 215000 |
| | 346000 |
| | $= D_o^2$ |

STEP 3. Using the combined data for all K groups, determine the best-fit curve and determine the sum of the squares of the deviations of all the data from the curve. Call this sum $D^2$.

STEP 4. Calculate the approximate F-statistic using the formula

$$F_{n,d} = [(D^2 - D_o^2)(N - 2K)] / [2(K - 1)D_o^2]$$

where N is the total number of points in the combined data and K is the number of groups being compared.

N = 15 K = 3

$F_{n,d} = (711{,}250 - 346{,}000)(15 - 6) / 2(2)\ 346{,}000 = 2.37$

STEP 5. Compare the calculated value of $F_{n,d}$ with the table value at the level of significance the test is being made. The subscript n is the number of degrees of freedom associated with $(D^2 - D_o^2)$ and is equal to 2(K – 1); d is the number of degrees of freedom associated with $D_o^2$ and is equal to (N - 2K).

$n = 2(K - 1) = 2(3 - 1) = 4$

$d = N - 2K = 15 - 2(3) = 9$

Select 5% level of significance from Snedecor and Cochran,[86] Appendix A:

at 5% $F_{4,9} = 3.63$

$3.63 > 2.37$

STEP 6. When the calculated F-value is less than the tabulated value, one may conclude that, at the chosen level of significance, there is no difference among the K curves.

5% chance that there is a difference. These curves are practically one curve.

| Actual Strength (1) | Maturity (2) | Log Mat (3) | Calc Strength (4) | Deviation (5) | Square of Deviation (6) |
|---|---|---|---|---|---|
| 1,000 | 636 | 2.80 | 1,112.07 | 112.07 | 12,559.64 |
| 2,290 | 2,106 | 3.32 | 2,175.47 | -114.53 | 13,116.58 |
| 3,050 | 5,136 | 3.71 | 2,967.23 | -82.77 | 6,851.18 |
| 3,580 | 9,981 | 4.00 | 3,557.31 | -22.69 | 514.79 |
| 4,170 | 20,256 | 4.31 | 4,185.90 | 15.90 | 252.88 |
| 1,000 | 616 | 2.79 | 1,083.69 | 83.69 | 7,004.44 |
| 2,060 | 2,011 | 3.30 | 2,134.48 | 74.48 | 5,546.93 |
| 2,690 | 5,056 | 3.70 | 2,953.29 | 263.29 | 69,319.23 |
| 3,570 | 10,096 | 4.00 | 3,567.49 | -2.51 | 6.23 |
| 3,710 | 20,176 | 4.30 | 4,182.39 | 472.39 | 223,150.15 |
| 1,280 | 702 | 2.85 | 1,199.76 | -80.24 | 6,438.55 |
| 2,500 | 2,100 | 3.32 | 2,172.94 | -327.06 | 106,969.25 |
| 2,700 | 4,965 | 3.70 | 2,937.15 | 167.15 | 27,940.75 |
| 3,670 | 10,035 | 4.00 | 3,562.10 | -107.90 | 11,641.75 |
| 4,650 | 20,145 | 4.30 | 4,181.02 | -468.98 | 219,940.30 |
| | | | | | $711{,}250 = D^2$ |

**TABLE D-1. SUMMARY OF F-TESTS ON MATURITY CURVES FOR CASTINGS 4 THROUGH 9**

| Casting Number | N | K | Do Squared | D Squared | Calculated Fn,d | n | d | Tabulated Fn,d | Result |
|---|---|---|---|---|---|---|---|---|---|
| 4 | 15 | 3 | 794,773 | 886,930 | 0.261 | 4 | 9 | 3.63 | passed |
| 5 | 15 | 3 | 2,527,041 | 4,804,941 | 2.028 | 4 | 9 | 3.63 | passed |
| 6 | 15 | 3 | 3,093,232 | 4,054,965 | 0.700 | 4 | 9 | 3.63 | passed |
| 7 | 15 | 3 | 3,489,654 | 4,181,148 | 0.446 | 4 | 9 | 3.63 | passed |
| 8 | 14 | 3 | 103,422 | 283,940 | 3.491 | 4 | 8 | 3.84 | passed |
| 9 | 13 | 3 | 1,950,989 | 2,342,710 | 0.351 | 4 | 7 | 4.12 | passed |

Note: See explanation of procedures earlier in this appendix.