

Lewis H. Tuthill International Symposium

on Concrete and Concrete Construction

Grant T. Halvorsen
Editor



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American Concrete Institute, Detroit



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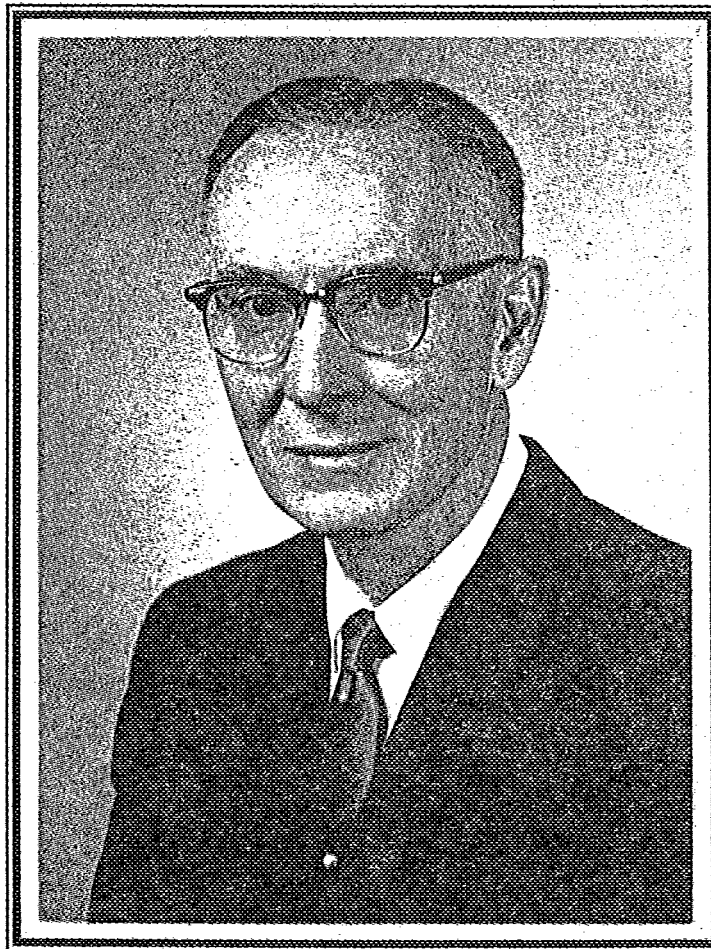
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Lewis H. Tuthill

PREFACE

In 1986, ACI Committee 224, Cracking, first approached the Technical Activities Committee (TAC) with a request to dedicate a convention session in honor of Committee 224 member Lewis H. Tuthill. This session was planned to recognize his contributions to the control of cracking through proper construction practices. The TAC members concurred with the merits of the plan, but realized that the original proposal was too modest to recognize properly Tut's broad contributions and decades of service to quality concrete, and to ACI.

In November, 1986, TAC gave final approval for the Lewis H. Tuthill International Symposium on Concrete and Concrete Construction, to be held at the ACI Fall Convention in Seattle, November 9-13, 1987. An ad hoc committee consisting of TAC members David Darwin and John M. Scanlon, ACI Past President Emery Farkas, and Committee 224 Chairman Grant T. Halvorsen, was appointed to plan and execute the details. Darwin and Halvorsen served as cochairmen of the symposium.

At the Seattle Convention, the Tuthill Symposium began with a banquet on Monday evening, and was followed by three half-day technical sessions, held Tuesday morning and afternoon, and Wednesday afternoon. Emery Farkas served as master of ceremonies at the banquet, while Halvorsen, Darwin, and Scanlon chaired the technical sessions.

The development of the symposium in only a year, and the printing of this ACI special publication in time for the 1987 Seattle Convention represent a tremendous investment in time and cooperation on the part of the authors, reviewers, and the ACI staff. In addition to those on the ad hoc committee, Terence C. Holland, chairman of ACI Committee 546, Repair, devoted a significant amount of time to the successful preparation of this volume.

With this symposium and volume of papers we honor Tut, and we promise to remember his lessons.

Grant T. Halvorsen, Editor

Lewis H. Tuthill International Symposium
on Concrete and Concrete Construction

Lewis H. Tuthill

Lewis Hamilton Tuthill was born in Philadelphia on November 24, 1899. His family moved to Oregon in 1909, and he graduated with a B.S. degree in Engineering from Oregon State College in 1920. Throughout his professional career, Tut has been involved with the technique and control of concrete construction, principally with regard to irrigation, water supply, and dams.

The early years of his career were spent in Oregon and California, where he worked for the Grants Pass and Merced Irrigation Districts, in private consulting, and then in concrete control work during the construction of the Exchequer, Melones, Pardee, and Calaveras dams, in addition to other projects in the San Francisco Bay area.

In 1931, Tut became the testing engineer for the Metropolitan Water District of Southern California and worked on the Colorado River Aqueduct. This project involved the acceptance of $6\frac{1}{2}$ million barrels of cement and $4\frac{1}{2}$ million cubic yards of concrete.

After completion of the Colorado River Aqueduct Project in 1939, he joined the U.S. Bureau of Reclamation in Denver, eventually becoming the Chief of the Concrete Laboratory. While at the Bureau of Reclamation, he participated in writing and editing the Bureau's first edition of the Concrete Manual. During World War II, he served for two years with the U.S. Maritime Commission in connection with the concrete ship program.

From 1956 until his retirement in 1969, he served as the Concrete Engineer for the California Department of Water Resources. During this time, he was responsible for matters affecting the quality of 7 million cubic yards of concrete in the two billion dollar California Water Project.

Since his retirement, Tut has actively served as a concrete engineering consultant. He is a registered Civil Engineer and Quality Engineer in California.

Lew Tuthill has been a member of the American Concrete Institute since 1926 and has participated in nearly every phase of Institute activity. He has contributed papers to the various ACI journals since the 1930's; a statement accompanying a 1942 paper on specifying coarse aggregates noted that "he has been too active as an ACI member to need further introduction."

A major part of his ACI activities has been in committee work. Over the years, he has served as chairman of the ACI committees on Concrete Floors, Winter Concreting Methods (now Cold Weather Concreting), Inspection, and Measuring, Mixing and

Placing Concrete. During his tenure as chairman of these committees, Tut served as the principal author for the original versions of the Recommended Practices for "Winter Concreting" and "Measuring, Mixing and Placing Concrete." The Fourth Edition of the Manual of Concrete Inspection was produced during his tenure as chairman of the Inspection Committee.

In addition to the committees he has chaired, Tut has served as a member of the ACI committees on Monographs, Nomenclature, Mass Concrete, Lightweight Concrete, Aggregates, Cracking, Hot Weather Concreting, Curing, Consolidation, and Construction Joint Practices. He was also a member of ASTM committees C-1 on Cement and C-9 on Concrete and Aggregates.

Lew Tuthill was the President of the American Concrete Institute in 1961, following service on the Technical Activities Committee (1953-58), the Board of Direction (1956-58), and as Vice President (1959-60). He then served on the Standards Board from 1960 to 1964. Since that time he has continued his devotion to ACI committee work and remains an active participant on eight committees.

Tut has received major honors and awards from ACI. He received the first ACI Construction Practice Award in 1945 for a paper on "Concrete Operations in the Concrete Ship Program" which was recognized to "involve a wealth of information applicable to high quality concrete construction for any purpose." In 1956, he and William A. Cordon were awarded the Wason Medal for the most meritorious paper, "Properties and Uses of Initially Retarded Concrete." In 1966, he was awarded the Henry C. Turner Medal for "his many important contributions to concrete technology and his untiring efforts towards improving the quality of concrete and of concrete construction practices." In 1967, Lewis H. Tuthill became the 48th Honorary Member of the American Concrete Institute and in 1970 he received the Institute's Distinguished Service Award.

Lew Tuthill married Anne Anderson in 1933. They have two children, John and Charles, who live in Denver and two grandchildren, John and Andy. Tut maintains an active personal professional life and claims that his major "hobby" is participating in ACI.

FOREWORD

By Lewis H. Tuthill

In this excellent group of papers the authors have ably shared their knowledge and experience with us. These papers serve as an excellent foundation for quality concrete construction. Those of us involved in concrete construction must take firm and confident control of all matters that will have an effect on the quality and service performance of the concrete work we do. We must do something positive and follow through to see that our intentions are met.

Quality has been mentioned in many high places but fully supported "follow through" has too often been lacking where the concrete is placed. There is talk about Quality, but what is done to make sure that it is achieved? Design for "a life of 50 years" is totally unrealistic for the aging infrastructure and much else. Hoover Dam is 50 years old, as are a number of the great water projects -- their required serviceability performance is as demanding now as it was at the start. Obviously we can not build the many elements of the infrastructure too well, and what little extra it may take to build in that extra assured quality is an investment of high return.

But, just spending that relatively minor premium will not assure the best possible results. Several additional steps must also be taken:

- First, specifications must state in full detail what the contractor / builder must do (furnish, use, perform) so the bid and contract price will include payment for what otherwise might be regarded as "extra's." It should all be part of the job without any special financial considerations.
- Second, a field engineering and inspection organization must be established and fully trained in the specifics and objectives of the job and specification requirements.
- Third, prebid and preconstruction meetings should be conducted to review the project requirements, and so that all concerned can meet each other.
- Fourth, Top-level Ownership and Management of the project must let it be known, by positive statement and actions, that the project engineering and inspection staff are supported fully in requiring performance in full accordance, and in all respects, of the specifications, and will entertain no questions or complaints without hearing from their staff at

the same time.

Many details of Quality Attainment in Concrete Construction will also be found in the following papers, principally in ACI publications:

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Design for Strength Variability: Testing and Effects on Cracking in RCC and Conventional Concretes

by E.K. Schrader

Synopsis: ACI and ASTM standards require an overdesign factor to be added to the design strength to account for variability in the mix. A high variability necessitates an average strength considerably higher than the design strength. There are two key problems with this approach:

(1) The higher strength requirement necessitates more cement which in turn results in more water. The result is more shrinkage, increased brittleness, and more crack potential. In massive structures, the increased cement also causes an increase in internally generated heat and greater thermal stresses or cracking.

(2) The amount of overdesign is based on results of cylinder strengths which may not be available until days, weeks, or months after the concrete has been placed. In dams (and especially in RCC structures), substantial volumes of concrete can already be in place before data is available that shows the strength should have been higher, or conversely, that it could have been lower (with a resulting savings in cost and lower internally developed stresses). A procedure is presented whereby variability can be based on a set of tests that are performed on the fresh mix as it is made. The variability is known in less than a day. This approach emphasizes quality control during mixing and placing so that the results are acceptable, rather than on obtaining later age test data that implies how good or bad the work was.

Keywords: coefficient of variation; compressive strength; concretes; cracking (fracturing); fresh concretes; mix proportioning; quality control; roller compacted concrete; shrinkage; strength; tests; thermal stresses; variability



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Ernest K. Schrader is a consulting engineer, Walla Walla, Washington. He has been involved with design, construction, research, and evaluation of concrete for 20 years and is one of the pioneers in RCC. Ernie has received numerous awards and is a member of several ACI committees including 224, "Cracking."

INTRODUCTION

Standards such as ASTM C-94 and ACI 318, and recognized authorities such as the United States Bureau of Reclamation (USBR) and the United States Army Corps of Engineers (COE) have adopted a statistical approach to establishing an appropriate overdress factor to account for field variability in concrete (1, 2, 3, 4 and 5).

Unfortunately, waiting for sufficient compressive strength data to accumulate so that the appropriate overdress factor can be calculated is often a problem. By the time data is available, the mix is already in place. The problem continues throughout construction. The overdress factor is adjusted up or down as variability increases or decreases, but the value used at any time is not necessarily correct for conditions at that time. The amount of overdress is based on what was already placed. Adjustments are not necessarily appropriate for future concrete either, nor can they be applied retroactively to adjust the mix already in place. This is especially important for structures with rapid rates of construction and where the design strength age is 90 days or more. Using an early age for control is the only reasonable approach, but there still is some delay until that data is available. Also, the correlation of variability and strength at an early age to what occurs at a later age is not always a constant and direct relationship. Especially with low strength, low cement content, and high fly ash mixes, the author has found difficulty in developing consistent early to later age relationships. This has occurred with both standard cure and accelerated cure techniques. Similarly, when "non-destructive" test methods have been used on structural concrete at early ages, the variability has been high.

These problems can be overcome by testing the fresh concrete for variability in properties that ultimately determine its strength. Primarily, these are water content, cement content, aggregate content and size distribution, compacted density, air content and unit weight of the air-free mortar. These



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properties can be rapidly tested after the concrete has been mixed, delivered, and deposited, but before it sets. A nuclear gage can be used to also determine the density and moisture content (and their variability) in the freshly compacted material. The procedure presented for doing these tests follows the basic concept of the Corps' approach to determining mixer performance as described in their Standard CRD-C55 (6).

The implications of an overdesign factor that requires a high average strength are very important issues. "Stronger" is not synonymous with "better." Efforts should be put into providing consistent concrete that permits lower average strength and cement contents - not in achieving higher strengths. Higher strengths that meet overdesign requirements are usually achieved by adding cement. This typically means a more brittle mix with more shrinkage stress or cracks - even if the water/cement ratio is not increased. In massive structures, the increased cement content also causes an increase in internally generated heat from hydration and greater thermal stresses or cracking.

CHARACTERISTIC STRENGTH

The required average strength, sometimes referred to as the "characteristic strength," f_{cr} , is the average strength level to be achieved at the design age. It is the minimum required average cylinder strength. One way to calculate it is:

$$f_{cr} = \frac{f'_c}{1-tv}$$

Where:

f'_c = Design strength

t = A constant obtained from standard statistics tables. It is based on the proportion of tests that may acceptably fall below the design strength, and on the number of samples used to establish v .

v = The coefficient of variation expressed as a decimal.

It is usually desirable to apply the formula on the last thirty test results, although more or less data can be used. A more detailed review of the procedure and its use can be found in References (1, 2, 6, 7, 8, 9, 10, 11).

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The characteristic strength is a higher value than the design strength. It is substantially higher when significant variability occurs in the concrete, and it is less when variability is low. The value of "t" is normally selected so that 90 percent of the test values will be above the design strength requirement for structures using prestress and ultimate strength design (most bridges, buildings, and common reinforced concrete construction). Another way to look at it is that 10 percent of the test values can be low if the average strength equals or exceeds the characteristic strength. The value of "t" is normally selected so that 80 percent of the test values will be above the design strength for dams, gravity structures, and designs which use the concept of working stress. In this case, 20 percent of the test values can be below the design strength. As discussed later, it seems appropriate for an even higher variability (lower overdesign) to be allowed in most massive structures such as large gravity walls and dams.

HOW COMMON IS THE USE OF OVERDESIGN FACTORS?

The standard procedure for overdesigning to compensate for variability is NOT in common practice, even though the concept is old, the statistical basis for it has long been established, and it is a part of current industry standards. The truth is that it is seldom used or enforced. In fact, most ready-mix operations do not even know the procedure exists. In an effort to quantify its use, the author found that the vast majority of ready-mix producers, construction inspectors, lab technicians, and engineers replied that they never used it, never needed it, and don't want it - but they don't want to be quoted or used in reference. This attitude needs to be overcome with cautious education. When told about it, people feel that the bureaucracy is imposing an unnecessary burden on them. They must be respectfully shown the need and benefit of the system to their industry.

A review of concrete dams, including those by major government agencies, shows that the overdesign concept typically has been addressed in one way or another at the time of initial mix designs. Later, during construction, field personnel usually kept track of strengths as they developed, kept the average value above the design value, and justified accepting low breaks as they occurred. Almost never has concrete been removed from a dam because cylinders broke below the design strength. On the other hand, almost never have low strengths been considered to be "excessively" low or "intolerable." Low breaks have typically been evaluated on a site-specific basis without using the

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statistical approach for overdesign which allows some low breaks.

Similarly, cement factors have typically been increased or decreased based on a judgemental review of cylinder breaks as they became available. The author has attempted to quantify what has been done since construction of Hoover and Grand Coulee dams, until and including current times. There has been no universally used procedure with set rules. Each project seems to have had one individual who took the responsibility of monitoring strengths and adjusting the cement factor. These individuals generally used 90-day strengths and about a two month moving average time period as a guide, but were also influenced by 28 and 7-day strengths as well as specific site conditions such as temperature, schedule, and construction needs.

The author's past approach to dam construction has been very similar to what others report. It has not usually followed the strict statistical approach now recommended by ACI and ASTM. A comparison to historical data generally shows that if these standards were used, a higher average strength and cement content would probably have resulted. Also, under the placing and temperature controls of the applicable construction contracts, the additional cement would have resulted in higher costs, more heat from hydration, and more cracking, whereas NOT enforcing the statistical approach resulted in little or no cracking at a lower cost - and satisfactory performance.

The author is now applying the statistical overdesign approach for some dams with the understanding that (1) results of tests on the FRESH mix are used to establish current variability; (2) the reasonableness of the resulting cement content and the overall procedure is evaluated as the work progresses; and (3) major efforts are put into reducing variability and providing a better overall quality concrete rather than into simply increasing cement to provide a higher average strength that allows a greater variability to occur.

At this point in the state-of-the-art, it appears that construction with mass concrete should follow the intent and general procedure of ACI and ASTM for overdesign factors, but that some modification is appropriate at least for gravity structures. The following compromise is suggested. This approach should be reviewed and endorsed or further modified by ACI Committee 207, "Mass Concrete."

* The overdiseign factor and required minimum average strength should be statistically calculated based on the last 30 test results of the FRESH mix or on early age strength data projected to in-place strengths expected at the design age.

* Not more than 25% of the results should be below the design strength.

* Not more than 4% of the results should be less than 75% of the design strength.

* Not more than 1% of the results should be less than 50% of the design strength.

* The reasons for obviously erroneous or damaged test data should be specifically noted and that data should not be included in the analysis.

The above recommendations are predicated on designs with structural factors of safety equal to or greater than those recommended by the USBR (12) and applied to the principal stress as follows:

- 3 for usual load conditions.
- 2 for unusual load conditions.
- 1 for extreme conditions.

Projecting from early test results to the expected ultimate in-situ strength at the design age should be done on a site-specific basis. A simple semi-logarithmic plot of early age data (log time vs strength) is a useful tool to help project strengths at later ages based on early data. If test data showing the long-term strength development for the particular mix design, cement, aggregate, and ages involved is not available, careful extrapolation based on the most similar available data should be used. Where no similar data is available, a reasonably conservative approach should be taken while also considering factors such as the conditions of cure and maturity within the mass, quality and frequency of inspections, probable actual age at the time of loading, and the detrimental side effects of unnecessary excess cement.

EARLY ESTIMATION OF LONG-TERM VARIABILITY

The step-by-step procedure given in Appendix 1 can be used to help establish the probable variability in long-term strength without needing to wait for the results of compressive cylinders that arrive too late to be effectively used. The specific procedure and sample sizes given in the appendix are suited to large aggregate roller compacted concrete (RCC), but the

approach is also applicable to conventional slump mass concrete by using vibration or rodding for consolidation instead of pneumatic tamping. The procedure is also applicable to smaller aggregate structural and paving mixes by simply reducing the sample sizes.

Bulk samples of the mix are taken from the placement after being deposited but before final compaction. For conventional concrete, the sample is taken after vibration. This approach is used instead of sampling at the mixer because it includes the beneficial effect of additional blending during transport and placing or, conversely, the harmful effects of segregation and drying if it occurs during transport and placing. Ultimately, it is the variability of concrete within the structure that is of concern - not its condition at the mixer. If there is reason to study the effect of transporting and placing, a comparison can be made using supplemental tests of samples taken from the mixer.

Sampling is done three times during a shift of placement. Each sample is immediately evaluated for:

- (1) Cement content.
- (2) Moisture content.
- (3) Unit weight of the air-free mortar.
- (4) Unit weight of the total mix.
- (5) Coarse aggregate content.

(In future work, it is suggested that the air content of the full mix is also determined.)

A "variability index" is calculated for each of the test items to compare variability of the in-place fresh concrete during the shift. The original CRD-C55 method obtained its samples from three different areas within a single batch. This tested only the mixer and "within batch" variations. Table 1 shows the maximum allowed variability for typical Corps of Engineers projects. If variabilities were higher, additional mixing time or modifications to the mix process were mandated. The table also shows tolerable variability index values that have been used for the new approach where samples are taken from the placement and include both "batch-to-batch" and "within-batch" variations.

The coefficient of variation can also be calculated for each of the test items. Preferably,

this uses the last 30 individual test results for each item. Each test item influences the ultimate strength and its variability within the structure. Arguments can be made that some of the items tested are more or less influential than others, but there is not enough data to establish if this is the case or to what degree it occurs.

Table 2 shows the variability of each test item for fresh mixes on various RCC projects. It also shows the coefficient of variation for all 6x12-inch (152x305 mm) cylinders at each age of testing. A very important and reasonably consistent relationship exists between variability in the constituents of the fresh mix and the variability in strength of test cylinders made for the mix. The average coefficient of variation in cylinder strength is about 2.7 times the average coefficient of variation in the fresh mix properties. By applying this factor to the variability of test results on the fresh mix, an early overdesign factor appropriate for what will be the eventual ultimate cylinder strength can be obtained.

A factor for conventionally placed concrete is not available at this time due to insufficient data.

The "2.7" factor might better be based on the relationship of variability in the fresh mix to variability in cores rather than cylinders, but again there is insufficient data at this time to make that evaluation. Using the relationship as it applies to cylinders may not be correct, but it is consistent with the current recommendations of ACI and ASTM which assume that the variability of cores will be similar to cylinders.

The very important issue of core vs cylinder strength has been reported to various degrees for mass and structural concrete using various mixes (4, 13, 14, 15, 16). Overall, it is reasonably well accepted that the long-term strength of 6 x 12-inch (152x305 mm) cylinders made with concrete having a maximum aggregate size of up to 2 inches (50 mm) will achieve approximately the same or slightly less strength as the same size cores taken from concrete in the structure if it has been properly placed and cured. There are exceptions. The exact relationship is dependent on peculiarities of each mix and placement. Generally, higher strength mixes have slightly lower core-to-cylinder strength ratios. Also, a knowledgeable technician can influence results by, for example, taking vertical cores from the top of a tall column placement (for lower strengths) or horizontal



cores at the base of the placement (for higher strengths).

For conventional large aggregate mass mixtures, the plus 1-1/2 or 2-inch (38 or 50 mm) aggregate is removed from material used to make the cylinders. The long-term core-to-cylinder strength relationship is generally the same as for the smaller aggregate mixes. However, there are more possibilities of situations with a discrepancy between cores and cylinders.

The above discussion is for conventionally placed slumpable concrete. Very important data now becoming available shows that the relationship of cores to cylinders may be very different for RCC mixes. Of special concern is data that implies that, at least in some field placements, high cementitious content RCC mass mixes which result in high cylinder strengths DO NOT result in similarly high in-situ concrete strengths in the structure. This is obviously a critical factor to consider when selecting a dam or other gravity section and the resulting stress levels which it will be required to carry. Table 3 shows available comparisons.

EFFECTS OF OVERDESIGN FACTORS ON CRACKING

Higher strengths achieved by adding more cement will almost always result in higher internally developed cracking stresses. The three main reasons for this are (1) increased temperature at the time of hardening (2) increased drying shrinkage, and (3) a more brittle concrete with a higher modulus of elasticity. ACI Committee 207 is developing a report that addresses these issues for structural as well as massive concrete sections. The problem is NOT limited to dams as generally believed. However, because dams are essentially unreinforced and engineers have traditionally been more concerned with cracks in dams than in beams, slabs, or columns, dams are a classical example.

One of the purposes of this paper is to bring awareness to the fact that excessive overdesign used to account for variability has harmful side effects that cannot be ignored. Anyone insisting on overdesign should not go "overboard" with it and should be aware of its consequences. By the same token, sufficient overdesign is needed to account for variability. As stated earlier, it often is better to put efforts and monies into reducing variability and overdesign requirements than it is into simply adding more cement and increase the allowed variability.

There are an infinite number of examples that can be given, and each situation must be individually evaluated. For one example, consider an unreinforced lean concrete mix for a gravity wall requiring a design strength at one year of 2,000 psi (13.8 MPa). A reasonably good coefficient of variation (v) of 10 would require an average or "characteristic" strength of 2180 psi (15.0 MPa). If little attention were given to variability and it had a " v " of 30, the required strength would be 2675 psi (18.4 MPa). Achieving this higher strength at a similar water-to-cement ratio would require about 75 more pounds (34 Kg) of cement and 50 more pounds (23 Kg) of water. For a typical mass mix, the potential drying shrinkage would increase about 100 millionths, the peak temperature from hydration would increase about 15 F degrees, and the modulus of elasticity would increase slightly - say on the order of about 200,000 psi (1.38 GPa).

Using a typical coefficient of thermal expansion of 4.5 millionths per F degree (8.1 millionths per C degrees) and an original modulus of elasticity value for the low strength concrete of 3.2 million psi (22.1 GPa), the higher strength mix will develop about 570 psi (3.9 MPa) MORE internal tensile cracking stress than the original mix. Using an approximate tensile strength equal to 10 percent of the compressive strength, the higher strength mix used to account for more variability will develop about 50 psi (0.3 MPa) more tensile strength but 570 psi (3.9 MPa) more internally developed stress. This example is simplified and does not take into account a probable lower creep rate which would add even more to the long-term tensile stress. It also looks at only one method of adjusting the mix.

CONCLUSIONS

The variability of concrete cylinder strengths can be predicted with reasonable effectiveness based on variability in the following properities or constituents of the fresh mix: Cement content, moisture content, unit weight of the air-free mortar, unit weight of the total mix coarse aggregate content. This information can be used to establish the appropriate characteristic strength or overdesign needed to account for variability.

Roller compacted concrete for mass applications with strengths in excess of about 2500 psi (17.2 MPa) may develop the required strength levels in cylinders, but the mix may not develop similar strengths in the structure.



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It is appropriate to consider special allowances for redundancy and size effects when determining the overdesign factor in large gravity structures.

Non-judicious use of overdesign factors for variability can result in higher levels of tensile stresses and cracking.

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(2) Extensive data made available through the U. S. Army Corps of Engineers; the U. S. Bureau of Reclamation; Guttridge, Haskins and Davey Engineers; O'Brien & Gere Engineers; Douglas County; and Morrison Knudsen Engineers. Because of their cooperative spirit and professional willingness to share information, it was possible to present the data and comparisons in Table 2. This is the first time that the RCC industry has developed a strong basis for probable strength variability.

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TABLE 1--MAXIMUM ALLOWED "VARIABILITY INDEX" VALUES

	SAMPLING AT MIXER	SAMPLING AT PLACEMENT
Cement Content	82.5	70
Moisture Content	91.5	75
Unit Weight of the Air-Free Mortar	98.5	85
Coarse Aggregate Content	90.5	80

Variability Index = (Smallest Value/Largest Value) x 100

TABLE 2--COEFFICIENTS OF VARIATION FOR MASS CONCRETE PROJECTS USING ROLLER COMPACTED CONCRETE

	WILLOW CREEK	COPPERFIELD	GALESVILLE	MONKSVILLE	UPPER STILLWATER*	MIDDLE FORK
Cement + Fly Ash (lb/cu yd)	80+32	135+50	89+36	105+00	134+291	112+00
Cement + Fly Ash (Kg/cu m)	47+19	80+30	53+51	62+00	80+173	66+00
FRESH MIX TESTS						
Unit Wt Air Free Mortar	2	-	2	1	-	-
Unit Wt Full Mix	1	-	1	1	-	-
Coarse Aggregate	8	-	8	14	-	-
Cement Content	21	-	16	13	-	-
Moisture Content	23	-	13	12	-	-
Average Coeff of Variation	11.0	-	8.0	8.2	-	-
CYLINDERS						
3 day	26	-	-	22	-	-
7 day	29	-	28	28	19	23
14 day	25	-	28	21	-	27
28 day	28	30	24	24	21	33
56 day	-	-	20	21	-	-
90 day	29	25	20	21	19	16
			(120 day)			
180 day	27	-	-	26	-	-
365 day	33	-	13	-	-	-
Average Coeff of Variation	28	28	22	23	20	25
RATIOS						
CofV (cylinder)/CofV (fresh mix)	2.55	-	2.75	2.80	-	-

*Based on data available as of April 1987 for 1985 and 1986 placements.

TABLE 3--COMPARISON OF CORES TO CYLINDER STRENGTHS (U.S. CUSTOMARY UNITS)

PROJECT(1)	MSA (IN)(2)	CEMENT (LB)	POZZOLAN (LB)	APPX AGE	APPX SIZE (IN)	CORES (3)		AVG LABORATORY CYLINDER STRENGTH - PSI (3)(4)				
						AVG STRENGTH (PSI)		28 DAY	56 DAY	90 DAY	180 DAY	365 DAY
Willow	1 1/2	315	135	75 days	8 x 12	394 S		3408		4469		2623
Willow	3	80	32	1 yr	8 x 12	2300						2210 (*)
Willow	3	80	32	1 yr	8 x 12	2300						2623
Willow (5)	3	80	32	1 yr	8 x 12	2150						3779
Willow	3	175	0	1 yr	8 x 12	2150						2931 (*)
Willow	3	175	0	1 yr	8 x 12	2080						3779
Willow	3	175	0	1 yr	8 x 12	2080						4146
Willow	3	175	80	1 yr	8 x 12	3220						3526 (*)
Willow	3	175	80	1 yr	8 x 12	3220						4146
Willow	3	175	80	1 yr	8 x 12	2260						
Copperfield	2	185	0	28 days	6 x 12	946		667				
Copperfield	2	185	0	56 days	6 x 12	1400		667		1040		
Copperfield	2	185	0	170 days	6 x 12	1682				1040		
Copperfield	2	135	50	170 days	6 x 12	1262		590		871		
Middle Fork	3	112	0	42 days	6 x 12	2016		1447		2067		
Galesville	3	89	86	1 yr	6 x 12	2170						1561
Galesville	3	89	86	1 yr	6 x 12	2020						1561
Galesville	3	115	110	1 yr	6 x 12	2000		2130 (x)	3510 (x)			1881
U. Stillwater (1)	1	182	210	28 days	6 x 12	1660						
U. Stillwater (1)	1	182	210	90 days	6 x 12	2400						
U. Stillwater (1)	1	182	210	180 days	6 x 12	2750						
U. Stillwater (1)	1	182	210	1 yr	6 x 12	2910						
U. Stillwater (1)	1	121	269	28 days	6 x 12	1960		1820 (x)		2150 (x)		5220 (x)
U. Stillwater (1)	1	121	269	90 days	6 x 12	2690						
U. Stillwater (1)	1	121	269	180 days	6 x 12	2920						
U. Stillwater (1)	1	121	269	1 yr	6 x 12	3080		1640 (x)				4780 (x)
U. Stillwater (1)	1	129	286	28 days	6 x 12	2690						
U. Stillwater (1)	1	129	286	90 days	6 x 12	3080						
U. Stillwater (1)	1	129	286	180 days	6 x 12	3330						
U. Stillwater (1)	1	129	286	1 yr	6 x 12	3640						
U. Stillwater (1)	1	127	277	34 days	6 x 12	3070		2870 (x) (34 days)				4960 (x)
TVA TF-2(1)	1	127	277	90 days	6 x 12	3640				3890 (x)		
TVA TF-2(1)	1	75	300	90 days	6 x 12	3620				2580 (x)		
TVA AB INT(1)	1	135	159	90 days	6 x 12	3680				3750 (x)		
TVA AB -35(1)	1	172	69	90 days	6 x 12	3640				2980 (x)		
TVA AB -46(1)	1	135	69	90 days	6 x 12	3640				2890 (84 days)		
MES-1(1)	4	235	0	71 days	6 x 12	3150			5250 (65 days)			
MES-2(1)	4	235	0	73 days	6 x 12	3720			4280 (71 days)			
MES-3(1)	3	235	0	66 days	6 x 12	3980						

(1) Projects that were demonstrations, test fills, field trials, or had relatively low volumes (less than about 35,000 cubic yards) are noted (1). Data for other projects (not noted (1)) represent the average for an entire dam or other large mass.

(2) Maximum size aggregate in the full mass mix.

(3) Where cubes were used the strength has been converted to the approximate equivalent core or cylinder value by applying a reduction factor of 20%.

(4) Except as noted by the following indicators, all cylinders are 6 x 12 inch with the plus 1/2 or 2 inch material removed by wet screening and compaction was by pneumatic tamping: (*) 9 x 18 inch cylinders; x = modified veebe compaction.

(5) These cores included lift joints. All other cores generally did not include joints.

TABLE 3(Continued)--COMPARISON OF CORES TO CYLINDER STRENGTHS (METRIC)

PROJECT(1)	MSA(MM)(2)	CEMENT(KG)	POZZOLAN(KG)	APPX AGE	APPX SIZE(MM)	AVG STRENGTH(MPa)	AVG LABORATORY CYLINDER STRENGTH -MPa(3)(4)				
							28 DAY	56 DAY	90 DAY	180 DAY	365 DAY
Willow	40	143	61	75 days	200 x 305	27.2	23.5		30.8		
Willow	75	36	15	1 yr	200 x 305	15.9					18.1
Willow	75	36	15	1 yr	200 x 305	15.9					15.2(*)
Willow(5)	75	36	15	1 yr	200 x 305	15.2					18.1
Willow	75	79	0	1 yr	200 x 305	14.8					26.1
Willow(5)	75	79	0	1 yr	200 x 305	14.8					20.2(*)
Willow	75	79	36	1 yr	200 x 305	22.2					26.1
Willow	75	79	36	1 yr	200 x 305	22.2					28.6
Willow(5)	75	79	36	1 yr	200 x 305	15.6					24.3(*)
Copperfield	50	110	0	28 days	150 x 305	6.5	4.6				
Copperfield	50	110	0	56 days	150 x 305	9.7	4.6				
Copperfield	50	110	0	170 days	150 x 305	11.6			7.2		
Middle Fork	75	80	30	42 days	150 x 305	8.7	4.0		6.0		
Galesville	75	51	0	1 yr	150 x 305	13.9	10.0		14.3		
Galesville(5)	75	40	39	1 yr	150 x 305	15.0					
U.Stillwater(1)	40	52	39	1 yr	150 x 305	13.9					10.8
U.Stillwater(1)	40	83	95	28 days	150 x 305	11.4					10.8
U.Stillwater(1)	40	83	95	90 days	150 x 305	16.5	14.7(x)	24.2(x)			13.0
U.Stillwater(1)	40	83	95	180 days	150 x 305	19.0					
U.Stillwater(1)	40	55	122	1 yr	150 x 305	20.1				32.5(x)	
U.Stillwater(1)	40	55	122	28 days	150 x 305	13.5	8.4(x)				36.0(x)
U.Stillwater(1)	40	55	122	90 days	150 x 305	18.5					
U.Stillwater(1)	40	55	122	180 days	150 x 305	20.1					
U.Stillwater(1)	40	59	130	1 yr	150 x 305	21.2			14.8(x)		
U.Stillwater(1)	40	59	130	28 days	150 x 305	18.5					
U.Stillwater(1)	40	59	130	90 days	150 x 305	21.2	11.3(x)			22.3(x)	
U.Stillwater(1)	40	59	130	180 days	150 x 305	23.0			19.1(x)		
TVA TF-2(1)	40	58	126	1 yr	150 x 305	25.1				27.7(x)	
TVA TF-2(1)	40	58	126	34 days	150 x 305	21.2	19.8(x)(34 days)				34.2(x)
TVAB INT(1)	40	34	136	90 days	150 x 305	25.1					
TVAB .35(1)	40	34	136	90 days	150 x 305	18.1					
TVAB .46(1)	40	60	72	90 days	150 x 305	25.4					
WES-1(1)	115	78	31	90 days	150 x 305	25.1					
WES-2(1)	115	107	0	71 days	150 x 305	21.7					
WES-3(1)	75	107	0	73 days	150 x 305	25.6					
				66 days	150 x 305	27.4					
								36.2 (65 days)			
								29.5 (71 days)			

- (1) Projects that were demonstrations, test fills, field trials, or had relatively low volumes (less than about 26,700 cubic meters) are noted (1).
- (2) Data for other projects (not noted (1)) represent the average for an entire dam or other large mass.
- (3) Maximum size aggregate in the full mass mix.
- (4) Where cubes were used the strength has been converted to the approximate equivalent core of cylinder value by applying a reduction factor of 20%.
- (5) Except as noted by the following indicators, all cylinders are 150 x 305 mm with the plus 40 or 50 mm material removed by wet screening and compaction.
- (*) = 230 x 455 mm cylinders; (x) = modified vebe compaction.
- (5) These cores included lift joints. All other cores generally did not include joints.

APPENDIX

MIXER AND PLACING PERFORMANCE TEST PROCEDURE
FOR ROLLER COMPACTED CONCRETE*

1. SCOPE

1.1 The procedures described here are used to evaluate how thoroughly roller compacted concrete (RCC) is mixed after it has been placed during construction, but prior to compaction. They determine water content, cement content, and coarse aggregate content for the full mix, and unit weight of the air-free mortar. By comparison of results from three samples taken throughout a shift of placing, the ability of the mixing and placing procedure to provide consistent material is determined. The Engineer may authorize deletion of the cement test portion of the procedure or use of an alternate method.

2. GENERAL

2.1 Mixer performance tests are used to establish minimum allowed mix times for conventional concrete or to determine the variability of a mix after a specified duration of mixing. Normally, the procedure consists of mixing a specified batch size for the designated length of time, obtaining samples of material from different locations in the mixer, and comparing results of tests on each sample to see how much they vary.

2.2 The procedure usually follows Corps of Engineers Test Method CRD-C 55 or the method allowed in ASTM C-94, Annex A1. A brief description of each test follows:

CRD-C 55 is based on three samples taken from a single batch. It checks for water content, cement content, unit weight of the air-free mortar, and coarse aggregate content. It requires about 1 week before results from all of the tests are analyzed and data by the chemical method of cement determination on hardened mortar is available. Most of the testing is performed on the mortar portion of the mix after after sieving from the full mix.

ASTM C-94 is based on two samples taken from a single batch. It tests for slump, air content, coarse aggregate content, unit weight of the 1-1/2 inch minus portion of the mix, compressive strength at 7 days, and unit weight of the air-free mortar. It requires

1 week before all results are available, and it does not use the full mix in all of the test procedures.

*NOTE: Metric conversions are not provided because a separate metric version of the procedure with similar (but not identical) sample sizes has been developed that uses convenient metric sizes. It is available upon request.

2.3 Each of these procedures (ASTM & CRD) were developed for conventional concrete mixes, and each checks only the effectiveness of the mixer itself. Blending of roller compacted concrete materials can be significantly affected during transporting, rehandling, and spreading operations. Generally this can be expected to provide additional beneficial mixing, although with improper practices, it could adversely cause segregation. This additional mixing can be used to advantage by taking samples from the placing area at different times during a shift rather than taking them from a single batch in the mixer. Using this technique, different batches of concrete are intermixed and the composite effect of "batch-to-batch" variability as well as "within batch" variability is taken into account. Accordingly, the maximum permissible variability of the tested samples is increased from typical values used when only within batch testing is done.

2.4 An additional significant advantage of sampling from the placement instead of taking samples from one batch at the mixer is that fresh samples are used in the tests. In CRD-C 55, three samples are obtained at the same time for testing. In practice, the third sample may be hours old before it is subjected to the tests, and results can be adversely affected.

2.5 Another major advantage of obtaining samples from the placement is its compatibility with continuous mixers.

2.6 The procedure described here also uses a rapid chemical analysis for determining the cement content, and results of the entire test procedure are normally available by the start of the day following the tests.

3. SAMPLING

3.1 After an agreed initial and conservative mix time has been established, a shift of placement shall be permitted at that mix time while testing is performed. If no guidance is provided in the contract

documents for an initial mix time, it shall be 1 minute for mixers with a capacity of 3 cubic yards. For larger mixers, the time shall be increased by 15 seconds for each additional cubic yard or fraction thereof. The mix time shall begin after all ingredients have entered the mixer and shall stop after the discharge cycle begins. For continuous mix plants, the mix time applies to equivalent discharge rates in cubic yards per minute.

3.2 During the shift of placement, samples shall be obtained during the first, second, and third portions of the shift. However, the first sample shall not be taken until after the plant has been in operation for at least the first 5 percent of the shift. The second sample shall not be obtained until after testing on the first sample has progressed to the point that it will be essentially completed and so that full attention can be given to the second sample as soon as it arrives at the laboratory. Similarly, the third sample shall not be obtained until testing on the second sample is essentially complete. All testing (except waiting for final drying where applicable) shall be completed within 2 hours after obtaining each sample.

3.3 The samples shall be obtained at random under the direction of the laboratory chief. They shall be excavated from the approximate center of a zone of RCC that has been spread for compaction by production methods, but prior to compaction. The sample size shall be approximately 5 cubic feet or 650 pounds. It shall include essentially all material through the full depth of the spread RCC layer. A recommended method of obtaining the sample is with a 1-ton 4x4 pickup truck and front-end loader assisted by two laborers with shovels. The sample shall be covered with a tarp and protected from wind, rain, sun, etc. while being transported. The sample shall be kept in, and all testing shall be done in, a protected environment out of the wind and sun and within a temperature range of 40 to 80 F degrees.

4. WATER CONTENT

4.1 Obtain approximately 1/2 cubic foot or 75 pounds of the full RCC mix and determine the sample weight to the nearest 0.1 pound. This sample should be taken and weighed as soon as possible after the bulk material is delivered to the laboratory. After weighing, spread the RCC material loosely in a series of pans that will fit into an oven. Be careful not to lose any of the sample. Preferably the sample should be initially weighed in the drying pans.

4.2 Dry the sample to constant weight, keeping it loosely separated to facilitate drying. Initial drying for 1 to 6 hours shall be accomplished by simple exposure to the dry atmosphere of an enclosed room, the sun, a breeze from an electric fan, etc. After initial drying, separate the material with a trowel or hoe to assure that it is still loosely separated and not hydrating into a solid mass. Then dry the sample to constant weight in an oven at 230 plus or minus 9 F degrees. Drying for 12 hours in the oven after the initial drying period is usually sufficient to obtain constant moisture.

4.3 Compute the total moisture content as follows:

$$P = \frac{W - D}{D} \times 100$$

Where:

P = The total moisture content in the mix (%)

W = The weight of the sample prior to drying (lbs)

D = The weight of the sample after drying (lbs)

5. UNIT WEIGHT OF AIR-FREE MORTAR

5.1 Sieve approximately 1 cubic foot or 150 pounds of the full RCC mix sample over a 2-inch screen. The sieving action shall be vigorous enough so that lumps of pasty material, mortar, and smaller aggregate fall through the screen. This can be accomplished by raking the sample back and forth over the screen as it is suspended above a platform or wheelbarrow. The retained coarse aggregate particles should not be washed and will have a slight coating of cement paste with fines. Discard the retained coarse aggregate.

5.2 Compact the material passing the 2-inch screen into a minimum 1/4 cubic foot pressure-type air meter and determine the compacted weight of the sample. Compaction should be accomplished in three lifts, compacting each lift with a pneumatic tamper prior to placing the next lift. The pneumatic tamper shall be equivalent to an Ingersoll-Rand Model SPG-30 with a 5-3/4 inch lock-type butt. The butt shall have a flat (not elliptical) striking surface, but the outside edge may be rounded. As the last lift is compacted in the mold, a helper should simultaneously add material so that the final compacted surface is essentially level with the top of the container. (A

collar that fits over the container is recommended to allow initial overfilling of the loose material and to protect the edge of the container from damage during compaction.)

5.3 Determine the air content of the compacted sample using the ASTM C-138 (Gravimetric Method) or ASTM C-231 (Pressure Method).

5.4 Wash the entire air-content test sample over a No. 4 sieve so that all material including paste and fines adhering to the plus No. 4 material is removed. Waste the minus No. 4 material. Obtain the saturated surface dry weight of the aggregate from the air content test sample that is retained on the No. 4 sieve. Towels and a fan will expedite drying the aggregate back to a saturated surface-dry condition.

5.5 Calculate the unit weight of air-free mortar as follows:

$$M = \frac{b - c}{V - \frac{V \times A}{100} - \frac{c}{G \times W}}$$

Where:

M = Unit weight of air-free mortar (lb/cu ft)

b = Weight of the concrete sample compacted into the air content test container (lb)

c = Saturated surface-dry weight of aggregate from the air content test container that is retained on the No. 4 sieve (lb)

V = Volume of the air content test container (cu ft)

A = Air content of the test sample (%)

G = Specific gravity of the coarse aggregate

W = Unit weight of water (62.3 lb/cu ft)

6. PERCENTAGE OF COARSE AGGREGATE

6.1 Obtain approximately 1-1/2 cubic feet or 200 pounds of the full mix RCC sample and accurately determine its weight. Wash the sample to remove all material passing the No. 4 sieve, including all coatings on the coarse aggregate particles.



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facilitate the washing operation, the weighted sample should be washed in smaller increments of approximately 20 to 30 pounds each, and a series of nested screens should be used above the No. 4 screen to avoid overloading. Discard everything passing the No. 4 sieve. Save everything retained on the No. 4 sieve.

6.2 Dry all washed material retained on the No. 4 sieve to a saturated surface-dry condition and obtain the weight. Towels and an electric fan will expedite drying the aggregate back to a saturated surface-dry condition.

6.3 Compute the percentage of coarse aggregate (as percentage of the total mix by weight) as follows:

$$C = \frac{w}{W} \times 100$$

Where:

C = Percentage of the full mix that is coarse aggregate (%)

W = Weight of the full mix sample that is taken for sieving (lbs)

w = Weight of saturated surface-dry aggregate retained on the No. 4 sieve (lbs)

7. CEMENT CONTENT

7.1 Cement contents of each sample shall be determined for the full mix using the calcium analyzer and suspension tank (washing machine) of the concrete quality monitor (CQM). The CQM as used for conventional concrete is described in Publication TR M-293 titled "Corps of Engineers Concrete Quality Monitor: Operations Guide," dated May 1981. It is available from the U. S. Army Corps of Engineers Engineer Research Laboratory, Champaign, Illinois. Modifications for use with RCC include adding Calgon to the wash water, using a larger sample, and providing more sieves to facilitate the washing operation. Extra care must be taken and extra effort is normally required when using roller compacted concrete as compared to conventional concrete because of the typically lower cement factors and higher fines content.

7.2 Fill the suspension tank with tap water to the 10-gallon mark on the side of the tank.

7.3 Provide a nest of the following stainless steel sieves above the suspension tank: 3/4 inch, No. 4, No. 30, No. 50, No. 100. (The sieves may be modified to best suit the gradation of the RCC and to facilitate washing, but the bottom sieve must be the No. 100).

7.4 Turn on the recirculating pump for the suspension tank and add 1/2 pound of Calgon Water Softener to the water. Be careful to keep the recirculating hose above the screens. Do NOT lose any of the recirculated water. If the hose slips from above the screens and water is lost or pumped out of the system at any time, the test procedure shall be restarted.

7.5 Obtain approximately 8 pounds of the full RCC mix and place it into one or more polyethylene open-top tubs. Special attention must be given to obtaining representative samples, especially if they contain large aggregate in excess of 1-1/2 inches. Accurately determine the total sample weight.

7.6 Transfer the sample to the nest of sieves over the suspension tank, taking care not to overload the sieves or restrict the flow of water through them. Depending on the gradation of the mix and the amount of fines, it may be advantageous to wash about 4 pounds of the sample at a time. Carefully wash the aggregate, using water from the recirculating pump hose. It may be necessary to individually hand rub or scrub the aggregate particles with a fine wire bristle brush to remove all adhering cement and fines. After material retained on the top sieve has been washed, the material and sieve may be removed so that washing can proceed for the material retained on the next sieve etc. Special care shall be taken to thoroughly rinse the finer aggregates that cannot be individually scrubbed. Rubbing them over the screen will help.

7.7 Obtain a sample of the recirculated water from the suspension tank. With the recirculating pump operating, insert a stirring spoon and vigorously stir the 10 gallon solution. Immediately extract a sample of the solution with a 30 ml syringe pipet. Place the suspended material in a 500 ml Erlenmeyer flask. Refill the syringe pipet with 5 percent nitric acid and then discharging it into the contents of the Erlenmeyer flask. While discharging the acid solution from the syringe pipet, shake it occasionally to ensure that all cement that settled out when the sample was taken has dissolved and is flushed out

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with the acid solution. Use a volumetric flask to add 250 ml of top water to the Erlenmeyer flask.

7.8 Put a magnetic stirring bar in the Erlenmeyer flask and place it on a magnetic stirrer. Turn on the stirring motor and stir for at least 3 minutes before taking the first sample from the flask for analysis. Continue stirring until the last sample has been taken from the flask.

7.9 Turn the calcium analyzer power switch to "on."

7.10 (This step is required each time the calcium analyzer cuvette is filled with new potassium hydroxide solution. A single cuvette filling is sufficient for 15 to 20 readings.) Fill the cuvette to the indicated mark with 1.0 N (Normal) potassium hydroxide and 100 *ML* (Eppendorf) of reconstituted calcein reagent. Put the cuvette in the analyzer, add 100 (Eppendorf) of calcium standard solution and push the titration button to condition the cuvette for analysis.

7.11 Begin the analysis by placing the m Eq/mg% toggle switch on the mg% position and adding 100 (Eppendorf) of the calcium standard to the cuvette. Press the titration button. Record the result and repeat the test by adding another 100 *ML* sample. Continue repeating until consecutive results are less than 1.5 percent apart. Push the calibration button and run an additional 100 *ML* sample of the calcium standard to ensure that the calcium standard readout value is 10 plus or minus 0.1 mg percent.

7.12 Determine the strength of the cement solution in the Erlenmeyer flask by analyzing a 20 *ML* sample in the calcium analyzer. Repeat by adding more 20 *ML* samples from the Erlenmeyer flask until the values are less than 1.5 percent apart. If the calcium meter readings are less than 5, the sample size and for calibration may be increased to a 100 sample.

7.13 Determine the cement content by entering the calibration graph with the stabilized calcium analyzer reading. If the sample size (approximately 8 lbs) is different from the size on which the calibration chart is based, the readings should be mathematically adjusted by proportioning to correct for the difference.

7.14 (This procedure is necessary prior to running any production tests.) Establish a

calibration graph indicating the cement content in pounds per cubic yard as a function of the calcium analyzer reading (sample attached). This is accomplished by making small batches of thoroughly mixed RCC in the laboratory using accurately weighed ingredients ingredients and variable cement contents. For each mix, the calcium analyzer reading corresponding to the known cement factor is determined. The points are plotted and a straight line graph is drawn.

7.14.1 The batch size should be not less than 2-1/2 cubic feet mixed in a mixer of at least 3 cubic feet capacity. Dampen the mixer drum prior to mixing, preblend the weighed materials (except water) by shovel on a hard floor prior to adding them to the mixer. Add the dry materials intermittently with the water while the drum is rotating. Use the last of the water to rinse down the inside edge of the mixer drum while it is rotating. After all ingredients have been added, mix for 3 minutes at the flattest drum angle that does not spill material out of the mixer. Rest for 3 minutes and scrape the inside surfaces of the mixer drum and blades. Mix again for 2 minutes and discharge onto a damp hard floor. Scrape all materials from within the mixer drum and remix the material on the floor by shovel. Obtain a representative sample and run the calcium analyzer test on the mix. It is desirable to use exactly the same sample size (8 pounds) for each calibration test. If different sample sizes are used, the results must be mathematically adjusted to normalize them to the same initial sample size on which the calibration chart is based.

7.14.2 As a minimum, four mixes shall be used to establish the calibration curve for each cement factor and for each set of mix proportions used in production. For each mix, all batch quantities except cement shall remain constant. One mix shall be at a cement factor of zero (none) pounds per cubic yard. One mix shall be at the designated cement factor. One mix shall be at approximately 15 percent less than the designated cement factor. One mix shall be at approximately 15 percent more than the designated cement factor.

8. REPORT

8.1 A report including the variability index for each test shall be prepared following the format shown on the attached sample sheet.

SP 104-2

Effects of Construction Practice on Concrete-Steel Bond

by D. Darwin

Synopsis: Construction methods and concrete properties affect the bond strength between concrete and reinforcing steel. Research on the effects of concrete slump, consolidation practice, bar position, concrete cover, and bar spacing is summarized. The research shows that the use of high slump concrete reduces bond strength. An increased density of vibration increases bond strength. Vibration is especially important for high slump concretes, both with and without superplasticizers. Revibration is of greatest benefit to top-cast bars in high slump concrete. Revibration may damage bond in initially well consolidated, low slump concrete. Top-cast bars show reduced bond strength with as little as 6 in. [150 mm] of concrete below the bars. Bond strength decreases with increased concrete depth below bars, decreased cover, and decreased bar spacing. Monolithic cover provides a higher bond strength than laminar cover of the same total thickness.

Keywords: bond (concrete to reinforcement); bridge decks; concrete construction; consolidation; cover; moving loads; plasticizers; pullout tests; reinforced concrete; reinforcing steels; repairs; resurfacing; structural engineering; vibration; water-reducing agents; workability

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INTRODUCTION

Construction methods and concrete properties play an important role in determining the bond strength between concrete and reinforcing steel. Changes caused by construction-related variables can exceed the modification factors that are used in development length design (ACI 318-83).

The key variables are concrete slump, consolidation practice, bar position, concrete cover, and bar spacing. In most cases, the effects of these variables are interrelated. For instance, the negative impact on bond strength of high slump concrete is magnified with a lack of consolidation, increased depth of concrete below a bar, and decreased cover.

This paper presents a survey of research on the effects of the key construction-related variables on concrete-steel bond strength. Most of the work pertains to horizontal deformed bars. Recent work concerning the use of superplasticized concrete and the use of revibration is of special interest. The role of transverse steel, an important design consideration, is not covered.

Lewis H. Tuthill has made "quality concrete construction" the major theme of his professional life. The research presented in this paper strongly emphasizes the importance of construction practice in determining concrete-steel bond strength.

CONCRETE SLUMP

The workability of concrete, generally measured by slump, affects the bond strength between concrete and reinforcing steel. After concrete is cast, it continues to settle and bleed. Settlement leaves a void below rigidly held bars. Bleed water collects below bars, whether rigidly held in place or not. The higher the concrete slump, the greater the tendency to settle and bleed.

Properly consolidated, low slump concrete provides the best bond with reinforcing steel. High slump concrete, used primarily when it is desirable to use little or no consolidation effort, results in decreased bond. Top-cast bars (bars near the upper surface of a concrete placement) appear to be especially sensitive to slump. Top-cast bars may or may not be "top reinforcement," defined as "horizontal reinforcement so placed that more than 12 in. of fresh concrete is cast in the member below the reinforcement" (ACI 318-83).

Menzel (1952) studied the effect of slump on bond strength for top-cast bars. Using specimens shown in Fig. 1, he observed a marked reduction in bond strength as the height of top-bars was increased from $2\frac{1}{8}$ in. to $3\frac{3}{8}$ in. (54 to 841 mm) when using 5 to 6-in. (127-152 mm) slump, hand-rodded concrete. The rate of decrease in bond strength with increasing height of the top-bars was greatly reduced by decreasing the concrete slump to a range of 2 to 3 in. (51 to 76 mm).

Zekany, Neumann, Jirsa, and Breen (1981) studied the effect of concrete slump on top-cast and bottom-cast splices. They found that the bond strength of both top-cast and bottom-cast bars decreased with increasing slump. The effect was most pronounced for the top-cast bars. Luke, Hammad, Jirsa, and Breen (1981) studied the influence of casting position on development and splice length using a 72-in. (1.83-m) deep wall specimen (Fig. 2). As shown in Fig. 3, the bond of top-cast reinforcement was reduced significantly with $8\frac{1}{2}$ -in. (216-mm) slump concrete, as compared to 3-in. (76 mm) slump concrete. The bond strength of these bars decreased 40 to 50 percent due to the increase in slump alone.

Donahey and Darwin (1983, 1985) studied the bond strength of top-cast bars in bridge decks. The bars had different amounts and types of cover. These included two monolithic covers, $\frac{3}{4}$ and 3-in. (19 and 76-mm) thick, and a laminar cover consisting of a $\frac{3}{4}$ -in. (19-mm) monolithic cover topped with a $2\frac{1}{4}$ -in. (57-mm) high density concrete overlay. Eight in. (203 mm) of concrete was used below the reinforcement. As shown in Fig. 4, increasing slump resulted in decreased bond strength.

High slump concrete can be obtained in a number of ways. It can be obtained by the addition of water, in which case the strength of the concrete is reduced. It can be obtained by the addition of water and cement, in which case the strength of the concrete can be maintained approximately constant. Or, it can be obtained by the addition of a high-range water-reducer or superplasticizer, in which case the strength is usually increased.

A number of recent investigations have specifically considered the bond of reinforcement to superplasticized concrete. Brettmann, Darwin, and Donahey (1984, 1986) used modified cantilever beam specimens (Fig. 5) and concretes varying in slump from $1\frac{3}{4}$ to 9 in. (44 to 229 mm). Three specimen depths were used. The key variables were degree of consolidation, concrete slump with and without superplasticizer, concrete temperature, and bar position. Based on their work, Brettmann et al. (1986) concluded that, if cast at a relatively high temperature (resulting in a short setting time), properly vibrated (ACI 309-72) high-slump superplasticized concrete and its low slump nonsuperplasticized base concrete provide approximately the same bond strength. The equal bond strength is due largely to the increased concrete strength obtained with the high-range water-reducer. However, for the same concrete strength, high slump concrete made with a high-range water-reducer has a lower bond strength than low slump concrete. Brettmann et al. also observed that if the high slump, superplasticized concrete is cast at a low temperature (resulting in a longer setting time), or if high slump, high cement factor, nonsuperplasticized concrete is used, bond strength decreases regardless of concrete strength. The results of Brettmann et al. for both superplasticized and nonsuperplasticized concrete, normalized for concrete strength, are illustrated in Fig. 6. In this case, the bond strengths are normalized to a concrete strength, f'_c , of 4000 psi by multiplying the experimental results by $\sqrt{4000/f'_c}$.

Musser, Carrasquillo, Jirsa, and Klingner (1985) and Zilveti, Sooi, Klingner, Carrasquillo, and Jirsa (1985) also studied the effect of high slump, superplasticized concrete on bond strength. Specimens were consolidated using an internal vibrator in both investigations. Musser et al. considered the anchorage of deformed bars in wall specimens. Bond was tested by a straight pull-out procedure (Fig. 7). Zilveti et al. used modified cantilever beam specimens similar to those used by Brettmann et al. (1984, 1986) (Fig. 5). Of the two test methods, modified cantilever beam tests provide a more realistic comparison to bond in structural members than do pull-out tests, since the test places both the concrete and the steel in tension. Pull-out tests of the type used by Musser et al. place the concrete in compression around the test bar. Musser et al. found little effect, with or without a correction for concrete strength. Zilveti et al. concluded that the addition of a superplasticizer simply to improve workability has no effect on bond strength. Additional analysis of Zilveti's results, however,

reveals that when corrected for concrete strength, the bond strength obtained with high-slump, superplasticized concrete is lower than the bond strength obtained with low slump concrete. This agrees with Brettmann's results. Also like Brettmann et al., Zilveti's tests show that high slump is more detrimental to bond strength when the temperature of the concrete is initially low. The lower temperature provides a longer period during which the concrete remains plastic and, thus, a longer period during which settlement and bleeding occur.

CONSOLIDATION

It is generally agreed that adequate consolidation is a key factor in quality concrete construction. The role of consolidation is usually described in terms of removing entrapped air. In terms of bond, adequate consolidation, usually in the form of high frequency internal vibration, plays the additional role of reducing the effects of settlement and bleeding, which result in the accumulation of bleed water and low density, weak concrete just below horizontal reinforcement. By disturbing the concrete, vibration helps restore local uniformity. Both the removal of entrapped air and the restoration of local uniformity appear to play an important role in improving bond strength.

In the following sections, the effects of initial vibration, delayed vibration, and revibration are discussed.

Vibration

Davis, Brown, and Kelly (1938) studied the effects of delayed vibration and sustained jiggling on bond strength. Using 4-in. (102-mm) slump concrete, vibrations were applied 0 to 9 hours after the placement to specimens consisting of deformed bars held vertically in cylindrical specimens. Delayed vibration up to 9 hours improved bond strength compared to initial vibration. Increases in bond strength of up to 62 percent were recorded. This improved bond strength has been used as evidence of the positive effect of revibration (Tuthill 1938, 1977). However, since the specimens were not initially vibrated, the improved bond strength must be attributed to delayed vibration, not revibration.

Using similar specimens for sustained jiggling tests, Davis et al. obtained increases in bond strength as the period of jiggling increased from one-half hour to two hours, after which there was no significant change up to a maximum of 6 hours.

Robin, Olsen, and Kinnane (1942) compared bond strengths of horizontally cast bars consolidated with external vibration and

hand rodding. External vibration of 3 to 4-in. (76 to 102-mm) slump concrete produced lower strengths than hand rodding for bars $1\frac{1}{2}$ or 3 in. (38 or 76 mm) from the bottom of the forms, but higher bond strengths than hand rodding for bars 6 or $7\frac{1}{2}$ in. (152 or 191 mm) from the bottom of the forms.

Using 2-in. (51-mm) slump concrete and both top-cast and bottom-cast deformed bars (Fig. 1), Menzel (1952) found that internal vibration significantly increased bond strength compared to hand rodding (Fig. 8). The relative improvement in bond strength with vibration increased as the distance of the bar above the base of the specimen increased.

In the study of top-cast bars in bridge decks described earlier, Donahey and Darwin (1983, 1985) considered the effects of the density of vibration on bond strength. They compared procedures using "high" density vibration (in which vibrator radii of influence overlapped) and "low" density vibration (in which the radii of influence did not overlap). They found that high density internal vibration improves bond strength.

Using the specimens shown in Fig. 5, Brettmann et al. (1984, 1986) observed that vibration is especially important when high slump concrete is used, whether the high slump is produced with the addition of water and cement, or with the addition of a superplasticizer. Brettmann used internal vibration and followed standard practices for vibration (ACI 309-72). As shown in Fig. 9, except in two cases, the bond strength of nonvibrated specimens was lower than the bond strength of vibrated specimens.

For high slump concrete obtained without a superplasticizer (see Group 2 in Fig. 9), bond strengths averaged 14 percent lower for nonvibrated specimens than for vibrated specimens. For bottom-cast bars, the average decrease was only 6 percent for nonvibrated specimens, largely due to the consolidation provided by the concrete above the bars. For top-cast bars, bond strength decreased 23 percent when the concrete was not vibrated.

For high slump, superplasticized concrete, nonvibrated specimens also exhibited a lower bond strength. There were two exceptions involving top-cast bars in high temperature [84°F (29°C)] concrete which received additional consolidation due to finishing operations. For the high temperature concrete (see Group 1 in Fig. 9), nonvibrated bottom-cast bars had 25 percent lower bond strength than vibrated bars. For lower temperature [54°F (12°C)] concrete (Group 3), the lack of vibration caused a decrease in bond strength that ranged from 8 percent for bottom-cast bars to 41 percent for top-cast bars in deep specimens.

Construction-Related Vibrations

Both field and laboratory studies have considered the effects of external construction-related vibrations. The concern is that these vibrations, which occur while the concrete is setting up, will damage the bond between the reinforcement and the concrete.

In 1974, the Massachusetts Department of Public Works attempted to study bond strength in bridge decks that were widened while traffic was maintained on the decks. They concluded that traffic-induced vibrations caused no reduction of bond strength. Unfortunately, the control specimens were made and cured in the laboratory and differed in strength and geometry from the specimens subjected to traffic. Hence, these results are of little use in determining the effects of traffic-induced vibrations on bond strength.

Furr and Fouad (1981) also studied the effects of maintaining traffic on existing lanes of a bridge while the bridge was being widened. In both field and laboratory investigations, they found that it was possible to have some reduction in bond strength. However, this reduction was limited to areas close to longitudinal construction joints, where relative movement between the concrete and the steel was greatest.

Harsh and Darwin (1984, 1986) studied the effects of simulated traffic-induced vibrations on bond strength in bridge deck repairs. They found that if low-slump concrete is used, bond strength actually increases. However, for slumps in excess of 4 in. (102 mm), traffic-induced vibrations result in a reduction in bond strength, in some cases over 10 percent. They concluded that with stiff concrete, vibrations improve consolidation, whereas with fluid concrete, vibrations cause additional settlement and bleeding. Overall, traffic-induced vibrations do not appear to be detrimental to bond strength in bridge deck repairs if low-slump concrete is used.

Revibration

When properly executed, internal vibration improves the quality of bond between concrete and steel. Much less is known about the effects of revibration, the process in which a vibrator is reapplied to concrete at some time after initial vibration.

Vollick (1958) found that internal revibration increases concrete compressive strength from 6.9 to 18.7 percent, depending on the concrete mix. He did not study the effect of revibration on bond strength. Larnach (1952) studied the effects of external

revibration on both bond and compressive strength. He found that external revibration reduced the bond strength of horizontally cast smooth bars from 6 to 33 percent, depending on the time after initial consolidation. He obtained reductions in compressive strength of approximately 15 percent.

Menzel (1952) studied the effects of internal revibration on the bond strength of deformed bars with 2 to 3-in. (51 to 76-mm) slump concrete. He found that revibration after 1 hour had no adverse effect on bottom-cast bars, but reduced the bond strength of top-cast bars by 28 to 60 percent.

Altowaiji, Darwin, and Donahey (1984, 1986) studied the effects of internal revibration on bond strength for concretes with slumps between $2\frac{3}{4}$ and $7\frac{1}{2}$ in. (70 and 191 mm). They tested both bottom and top-cast bars using specimens similar to those used by Brettmann et al. (1984, 1986) (Fig. 5). Forty-five and 90 minute intervals were used between initial vibration and revibration. Altowaiji et al. found that revibration improves the bond strength of top-cast bars and bars placed in high slump concrete (Fig. 10, 11). However, revibration can significantly reduce the bond strength of bars cast in well consolidated, low slump concrete (Fig. 11). Revibration is almost universally detrimental to the bond strength of bottom-cast bars.

The bars benefiting most from revibration, top-cast bars placed in high slump concrete, are the bars that are affected most by settlement and bleeding. Revibration helps to reconsolidate the concrete adjacent to the bars, reducing voids caused by settlement and bleeding and, thus, improving bond. Altowaiji et al. concluded that from a practical standpoint, structures in which revibration has its greatest advantage (structures placed with high slump concrete) are the least likely to receive proper consolidation at any stage.

The detrimental effects of revibration to bottom-cast bars are probably due to the fact that settlement and bleeding help to consolidate the concrete around these bars, while revibration only serves to disrupt the concrete. Thus, full-depth revibration appears to be a poor construction practice. When used, revibration should be limited to the upper portions of a placement, probably no deeper than about 6 in. [150 mm] below the concrete surface. This limitation allows the effects of settlement and bleeding to be counteracted around top-cast reinforcement without damaging the bond of deeper bars. The use of a vibrator to tie together two lifts of concrete is a common application of revibration under these guidelines.

BAR POSITION

As early as 1913, Abrams observed that bar position during concrete placement plays an important role in the bond strength between concrete and reinforcing steel. Top-cast bars have lower bond strengths than bars cast lower in a placement (Altowaiji et al. 1984, 1986, Brettmann et al. 1984, 1986, Clark 1946, 1949, Collier 1947, CUR 1963, Donahey and Darwin 1983, 1985, Ferguson and Thompson 1962, 1965, Larnach 1952, Luke et al. 1981, Menzel 1952, Menzel and Woods 1952, Thompson et al. 1979, Untrauer 1965, Untrauer and Warren 1977, Welch and Patten 1965, Zekany et al. 1981). The provisions of ACI 318-83 and the AASHTO Bridge Specifications (1983) recognize this, after a fashion. "Top-reinforcement", reinforcement with more than 12 in. (305 mm) of fresh concrete cast in the member below the reinforcement, requires a 40 percent increase in development length. Most research, however, indicates that while an increased depth of concrete below a bar reduces bond strength, the effect of a shallow top cover is of greater significance. The importance of shallow top cover on the top-cast bar effect is emphasized by the fact that the strength reduction becomes progressively greater as the cover is decreased.

The lower bond strength of top-cast bars may be explained as follows: The greater the depth of concrete below a bar, the greater will be the settlement and bleeding at the bar, simply because there is more concrete beneath the bar to settle and bleed. The effects of settlement and bleed on the concrete around a bar are aggravated by increased concrete slump and decreased cover above the bar. Dakhil, Cady, and Carrier (1975) found that longitudinal settlement cracking increased above top-cast bars with increased slump and bar size, and decreased top cover (Fig. 12).

Menzel (1952) observed settlement cracks over 1-in. (25-mm) dia. top-cast bars with a 2 in. (51 mm) cover for specimens placed with 6-in. (152-mm) slump concrete that was consolidated by hand. Cracks were not observed in lower slump concrete that was vibrated internally.

As noted earlier, Menzel (1952) also observed that the bond strength of top-cast reinforcement decreases as specimen depth increases (Fig. 8). He observed the greatest reduction for high slump concrete consolidated by hand rodding, and the least for low slump concrete consolidated with vibration.

The importance of cover on the reduction in bond strength for top-cast bars is demonstrated by tests in the Netherlands (CUR 1964) that used modified cantilever beam specimens. As shown in Fig. 13, the ratio of top-cast bar to bottom-cast bar bond strength decreased significantly as cover decreased.

Ferguson and Thompson (1965) conducted beam tests to compare the ratio of top-cast to bottom-cast bar bond strength. They noted that the bond strength of top-cast bars decreased with increasing slump. Although they did not note the fact, their test data also show a clear trend of decreased strength with reduced top cover (Fig. 14) (Donahey and Darwin 1983).

Zekany et al. (1981) conducted splice tests in 16-in. (406-mm) deep beams with both top and bottom-cast bars. Splice strength decreased for both top and bottom-cast bars with increasing slump, but the decrease was consistently greater for the top-cast bars.

In their study of casting position (Fig. 2), Luke et al. (1981) clearly demonstrated that as the depth of concrete below a bar increases, the bond strength decreases (Fig. 3). They observed the greatest incremental decrease in strength for top-cast bars. As noted earlier, they also observed that bond strength decreased with increasing slump, but that the decrease was most pronounced for top-cast bars. For bars cast below the specimen mid-depth, slump appeared to have little effect.

Brettmann et al. (1984, 1986) also found that as the amount of concrete below a bar increases, bond strength decreases. The measured decrease was least for low slump concrete and greatest for high slump concrete obtained without the use of a superplasticizer (Fig. 15). They observed a reduction in bond strength for bars with as little as 8 in. of concrete below the bar (Fig. 5), bars that would not be defined as "top reinforcement" under the provisions of ACI 318-83. In addition, they observed little difference in bond strength between bars placed 8 and 15 in. (203 and 381 mm) above the bottom of the forms, with the exception of concrete that was not vibrated, which showed a measurable decrease. Bond strength was lowest for bars placed 36 in. (914 mm) above the bottom of the forms. As shown in Fig. 16, the largest reductions were obtained for high slump, nonvibrated specimens. The decrease in bond strength was greater between bottom-cast and top-cast bars than between bars with 8 and 36 in. (203 and 914 mm) of concrete below the bars. These results indicate that the choice of 12 in. (305 mm) of concrete below a bar for the 40 percent increase in development length for top reinforcement is arbitrary. Based on concrete depth alone, there seems to be a gradual decrease in bond strength with no sharp drop-off.

In similar tests run by Zilveti (1985), a "top-bar effect" was obtained for top-cast bars with as little as 6 in. (152 mm) of concrete below the bars.

COVER AND BAR SPACING

Concrete cover and bar spacing play an important role in bond strength. This point is not recognized by ACI 318-83, except that bars spaced laterally 6 in. or more are allowed a 20 percent reduction in development length. The role of cover and bar spacing lies largely in the ability of concrete to resist splitting forces caused by a bar undergoing bond failure. The lugs on a bar act as wedges which split the concrete at pullout. For shallow covers, splitting cracks extend to the surface (Fig. 17a), while for close bar spacings, splitting cracks extend between bars (Fig. 17b).

Orangun, Jirsa, and Breen (1977) recognized the importance of both bar spacing and cover in their detailed analysis of existing test data. Although not yet adopted, their work has provided the impetus for recommended changes in the development length provisions of ACI 318-83. These provisions, virtually unchanged since 1971, are based on the ultimate bond stress provisions of ACI 318-63.

The importance of cover has been emphasized by other investigators. Untrauer (1965) pointed out that bars with covers below $1\frac{1}{4}$ in. (32 mm) have bond strengths less than that predicted by ACI 318-63. Concern for problems with bars with close spacing and low cover prompted Ferguson (1977) to recommend that designers adopt conservative modifications to the development length provisions of ACI 318. As pointed out earlier in the discussion of top-cast reinforcement and illustrated in Fig. 13 and 14, the effects of cover are especially important for top-cast reinforcement.

A major goal of the research performed by Donahey and Darwin (1983, 1985) was to determine the effects of cover thickness and cover type on the bond strength of top-cast bars placed in structural slabs, with special emphasis on methods of construction for reinforced concrete bridge decks. As described earlier, they considered bars with $\frac{3}{4}$ and 3 in. (19 and 76 mm) of monolithic cover and bars with a laminar cover consisting of a $\frac{3}{4}$ -in. (19-mm) monolithic cover topped with a $2\frac{1}{4}$ -in. (57-mm) high density concrete overlay. As illustrated in Fig. 4, cover thickness and type had a significant effect on bond strength, resulting in an over 40 percent reduction in bond strength for a reduction in monolithic cover from 3 to $\frac{3}{4}$ in. (76 to 19 mm). A measurable reduction in bond strength was also obtained for the laminar cover. Donahey and Darwin felt that the reduction in bond strength for the laminar cover was due to the formation of settlement cracks above the bars in the initial $\frac{3}{4}$ -in. (19-mm) layer, coupled with the tendency of finishing equipment to remove coarse aggregate from the concrete above the bars. The lower coarse aggregate content aggravated shrinkage cracking above the bars. The longitudinal settlement and shrinkage cracks served as

incipient bond cracks and contributed to a reduction in bond strength. Donahey and Darwin also pointed out that should the bond between the first and second courses of concrete be unsatisfactory, the effective cover would drop to $\frac{3}{4}$ in. (19 mm).

CONCLUSIONS

This survey of bond research indicates that construction practices can strongly affect the bond strength between reinforcing steel and concrete. Concrete slump, consolidation procedures, thoroughness of consolidation, bar placement, and concrete cover play important roles in the strength of reinforced concrete structures. The key conclusions are summarized below.

1. For concretes with the same compressive strength, bond strength decreases with increasing concrete slump.
2. The addition of a superplasticizer solely to increase slump has relatively little effect on bond strength, if the concrete sets up rapidly after consolidation. For the same concrete strength, bond strength is lower for high slump, superplasticized concrete than for low slump concrete.
3. If good vibration practices are followed, an increased density of vibration will increase bond strength. Vibration is especially important for high slump concrete, whether or not the high slump is obtained with a superplasticizer. If high slump concrete is not vibrated, a significant reduction in bond strength is obtained.
4. If low slump concrete is used, traffic-induced vibrations increase bond strength in bridge deck repairs. If high slump concrete is used, traffic-induced vibrations may decrease bond strength.
5. Bond in initially well-consolidated, low slump concrete may be damaged by revibration. Revibration is of the greatest benefit to top-cast bars in high slump concrete.
6. Top-cast bars show a reduced bond strength with as little as 6 to 8 in. (152 to 203 mm) of concrete below the bars. The magnitude of the reduction increases with reduced cover, increased concrete slump, and increased concrete depth below the reinforcement.
7. Bond strength decreases with increased concrete depth below bars. Top-cast bars are the most sensitive.
8. Increased cover increases bond strength.

9. Monolithic cover provides a higher bond strength than laminar cover of the same total thickness.

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