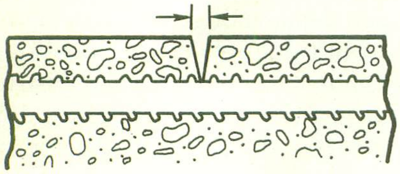
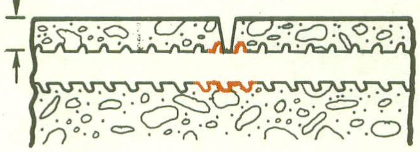
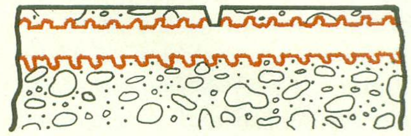
DEBATE:

Crack width, cover, and corrosion







This debate was held at the 1984 ACI fall convention in New York, New York. It was sponsored by ACI Committees 222, Corrosion of Metals in Concrete, and 224, Cracking. David Darwin was the moderator and the participants were David G. Manning, Eivind Hognestad, Andrew W. Beeby, Paul F. Rice, and Abdul Q. Ghowrwal.

The American Concrete Institute expresses its appreciation to the moderator, David Darwin, for his assistance in editing the transcription of the tape of the debate.

Why a debate?

Darwin: Current design provisions tie crack width to steel stress, bar spacing, thickness of the cover, and exposure conditions, indicating that an increased severity of exposure requires a narrower crack width. Under current provisions a narrower crack width can be obtained with a reduced cover. However, a reduced cover on reinforcing steel may lead to a decrease, not an increase, in corrosion resistance. What is the correct approach? There are a number of important questions involved, which we hope to expose to discussion today.

Background of the problem

by David G. Manning

Manning: If we begin by looking at the normal situation of steel in concrete, we find that reinforcement is surrounded by concrete having a high pH (somewhat greater than 12.5), which results in the formation of a microscopic passive oxide layer. We also note that the cover provides or impedes the transport of oxygen, moisture, and aggressive ions, and the net result is that we have no corrosion. The cover itself provides both chemical and physical protection to the steel.

There are two important ways in which that protection can be lost. One is through carbonation, which can lower the pH to less than 9. the other is penetration by aggressive ions, of which chlorides are the most important. Once we exceed a certain critical value the passive oxide film can dissolve, and in the presence of oxygen and moisture, corrosion is possible.

Carbonation is the reaction between concrete and acidic gases in the atmosphere. We find that the depth of carbonation follows a square root time law

$$d_t = A\sqrt{t} .$$

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The factor A is a function of the permeability of the concrete, the relative humidity of the concrete, and the concentration of gas in the service environment. According to calculations by Lawrence,* the depth of carbonation may vary between 19.5 and 0.05 mm (0.77 and 0.002 in.) in the first year. For low water-cement ratio concrete, the depth of carbonation in the first year will typically be on the order of 1 to 2 mm (0.04 to 0.08 in.). Consequently, carbonation is not a serious problem for the initiation of corrosion, except when dealing with very poor quality concrete or very shallow covers.

The more important means by which that passive film is destroyed is through penetration of aggressive ions into the concrete pore water. Again the depth of penetration approximately follows the square root time law

$$d_2 \approx B\sqrt{t}$$

As in the case of carbonation, the factor (in this case B) is a function of the permeability of the concrete, the relative humidity of the concrete, and the ion concentration. The reason we have an approximately equal sign is that there is some chemical interaction between the chlorides and the constituents of the cement paste.

Let us turn our attention to the mechanism of corrosion of steel in concrete. If we look at the simplified model of the corrosion process shown in Fig. 1, we see that there are four essential components of an electrochemical cell. These are the anode, where ions go into solution and electrons are released; the conductor, which is the reinforcing steel itself, permitting the transfer of electrons from the anode to the cathode; the cathode, where the electrons are consumed in the presence of oxygen and moisture; and the electrolyte, which is the moist concrete which permits the movement of ions between the cathode and the anode.

I would like to make a point at this stage: it is the reactions at the cathode which control the overall rate of corrosion of steel in concrete. I will explain the significance of this later.

Let me recap what has been said. We have established that corrosion protection is a function of the amount of cover, the quality of the concrete cover, and particularly the water-cement ratio of that cover. The processes which influence corrosion are more or less controlled by diffusion processes, whether we are talking about carbonation, the penetration of aggressive ions, or the supply of oxygen which is needed to support the cathodic reactions.

But, I have said nothing about the role of cracks. There are two very different theories about the effect of cracks on the corrosion of steel in concrete. Theory No. 1 states that cracks significantly reduce the service life of structures by permitting access of chloride ions, moisture, and oxygen to the reinforcing steel, not only accelerating the onset of corrosion but providing space for the deposition of corrosion products.

The opposing argument, Theory No. 2, is that while cracks may accelerate the onset of corrosion, such corrosion is localized and confined to the intersected reinforcing bars. Since chloride ions eventually penetrate uncracked concrete and initiate more widespread corrosion, after a few years of service there is little difference between the amount of corrosion in cracked and uncracked concrete. As is usual in the case of two extreme positions, I do not believe that either reflects the real situation in its entirety, but I would like to suggest to you that Theory No. 2 is much closer to the actual situation than Theory No. 1.

It is useful here to try and point out some of the factors which might be expected to influence the effect of cracks on corrosion, and we can group those into two categories. First, those which are a function of the crack itself. These include width, depth, shape, orientation of the crack with

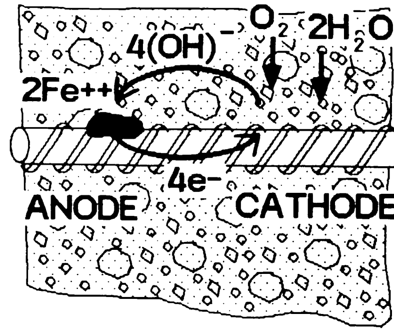

Fig. 1 — Simplified model of corrosion process.

*Lawrence, C. D., "Durability of Concrete: Molecular Transport Processes and Test Methods," *Technical Report* No. 544, Cement and Concrete Association, Wexham Springs, July 1981, 24 pp.

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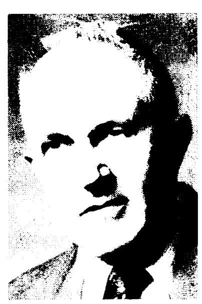

***Paul F. Rice** is vice president of engineering, Concrete Reinforcing Steel Institute, Schaumburg, Ill., and has been with that association since 1958. A Fellow of ACI and former member of the ACI Board of Direction and the Technical Activities Committee, he is currently secretary of Committee 315, Detailing. He holds engineering degrees from North Dakota State University and Massachusetts Institute of Technology and was technical director of ACI prior to moving to CRSI. His honors include the Arthur J. Boase Award from the Reinforced Concrete Research Council.*

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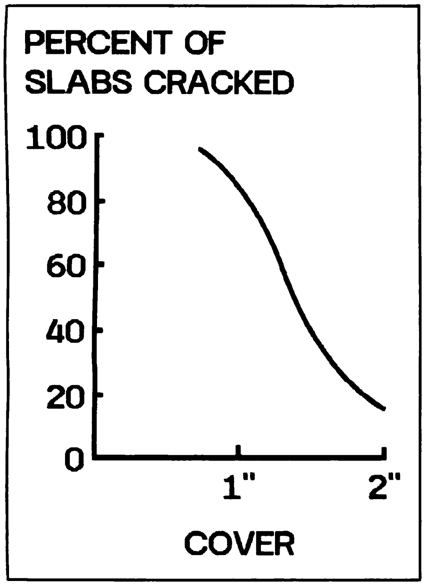

*Fig. 2 — Effect of cover on settlement cracking.**

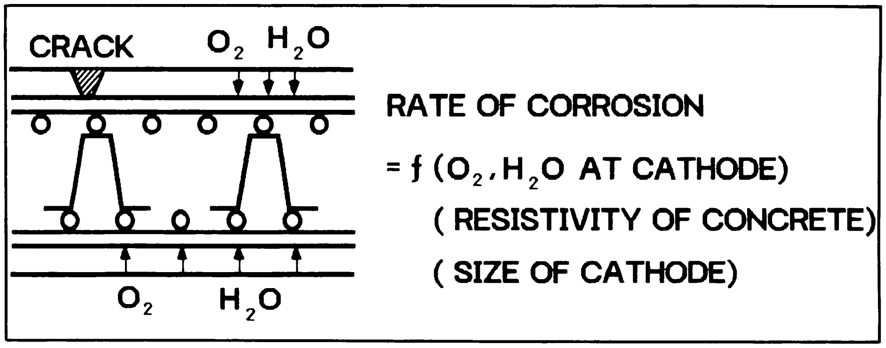

Fig. 3 — Factors affecting the rate of corrosion.

respect to the reinforcing steel, the intensity of the crack, and the origin of the crack. Second, the group of factors which revolve around the type of structure and the quality of concrete. These include service environment, the type of structure itself, the means by which passivity is lost, the resistivity of concrete, and whether or not the structure is under cathodic protection, either intentionally or unintentionally.

I do not propose to deal with all of these factors, simply because of the restrictions of time; but I will discuss two of the more important.

Shape of the crack. It is reasonable to expect that the width of the crack at the reinforcing bar would determine the extent of corrosion. Unfortunately, most research reports, and indeed most code provisions, deal with crack width at the surface of the concrete. And the fact is that the crack width at the bar is not in any way uniquely related to the crack width at the surface. In fact, the crack width at the bar is a function of the origin of the crack (such as flexural stress, tensile stress, or settlement of plastic concrete), the amount of cover, the stress in the steel, the reinforcement ratio, the arrangement of the bars, the diameter of the bars, and depth of the tensile zone.

Orientation of the bar. This is one of the most significant aspects of the effect of cracks on corrosion for cracks which are generally not more than 1 mm (0.04 in.) wide. Where the crack is transverse to the reinforcement, we see localized corrosion. A good rule of thumb is that corrosion is limited to about three bar diameters. If the concrete is of low permeability, then the corrosion reactions will in fact slow and may eventually cease. However, where the crack is longitudinal and coincides with the bar, the passivity is lost at many locations. Corrosion can then proceed unchecked and will in fact accelerate.

Probably the most common cause of longitudinal cracking (cracks which coincide with the bar) is settlement of plastic concrete, where the continued consolidation of the concrete is restrained by the reinforcing steel, resulting in cracks directly over the bars. Even here, we find that the extent of settlement cracking is a function of the cover, as shown in Fig. 2. As the cover increases, the tendency towards settlement cracking decreases substantially. There is also a small influence of concrete slump and bar diameter, but cover is by far the most important consideration in the occurrence of settlement cracking.

Referring to Fig. 3, I would like to explain why cracks may not be quite as significant as might first be assumed. If we take a look at the case of a transverse crack intersecting reinforcing steel, then at early stages, the wider the crack the greater the corrosion. However, for continued corrosion to occur, oxygen and moisture must be supplied to other parts of the same bar or to bars which are in electrical contact with that bar. As we have already stated, the rate of corrosion is a function of the reactions at the cathode and the resistivity of the concrete; there is certainly some interrelationship between these two factors. The rate of corrosion is also a function of the size of the cathode. Consequently, if the combination of the density and the thickness of the concrete cover is adequate to restrict the flow of oxygen and moisture, and the resistivity of the concrete is high enough, then the corrosion processes can be slowed and may eventually be stopped.

In the short time available to me for this introduction, I have tried to show the interrelationships between corrosion, cover, cracking, and crack widths, and to emphasize the importance of the permeability and thickness of the concrete cover. I have said nothing about the structural implications of these considerations, and I will leave it to my colleagues to develop those ideas.

Background of Current Provisions

by Eivind Hognestad

Hognestad: I was probably asked to outline the current provisions regarding cracking and protection of reinforcement against corrosion because I was involved in writing those provisions over 13 years ago.

The 1971 ACI Building Code was designed to accommodate the use of reinforcing steels with a yield strength of 60,000 to 80,000 psi (413 to 552 MPa). Crack control was introduced through rules for distribution of reinforcement.

Also in 1971, Norwegian Veritas Criteria set the course for the design of the first major concrete offshore structure. Cracking was controlled by limitations on steel stress at service loads.

These two sets of criteria were written recognizing that numerous factors influence corrosion of reinforcement. First, environmental exposure ranges from interior to exterior and from severe freeze-thaw to tropical climates. The use of quality concrete is extremely important, involving such variables as low water-cement ratio, adequate air-void system, and thorough compaction of the concrete. Chloride content of the concrete must be held down: wash marine aggregates, and do not use seawater to mix concrete. Finally, various corrosion inhibitors are becoming available; a simple precaution is to use a cement with at least 4 percent tricalcium aluminate.

Another set of factors influencing corrosion, concerns structural design. The importance of adequate concrete

*Dakhil, Fadh H.; Cady, Philip D.; and Carrier, Roger E., "Cracking of Fresh Concrete as Related to Reinforcement," ACI JOURNAL, *Proceedings* V. 72, No. 8, Aug. 1975, pp. 421-428.

cover has been borne out by numerous investigations. It is well-known that reducing cover to reduce crack width is madness in terms of corrosion protection. Reducing steel stress seems effective, on the other hand, because crack width is reduced proportionately. The stress field is important in terms of surface cracking due to flexure, versus cracks clear through a member due to membrane tension. Finally, sensible reinforcing details involve bars well distributed in zones of maximum concrete tension.

The resistance of reinforcement to corrosion can be enhanced in various ways. The use of epoxy coated bars is increasing. Bars clad with stainless steel are being manufactured experimentally in Europe, after completely stainless bars were found to be too costly. Galvanized steel is yet another alternative. Prestressing can significantly reduce cracking at service loads; in some cases prestressing even becomes a necessity to fulfill the design's intended function. Finally, there are cases where coating the concrete surface can provide added protection against reinforcement corrosion.

When the 1971 ACI Code, and the Norwegian Veritas Criteria were written, we did not know the limits beyond which crack width and steel stress would lead to corrosion of reinforcement. At that time we had only the first 10 years of data from exposure tests run by the U.S. Corps of Engineers at Treat Island, Maine. In the tidal zone, they exposed 76 reinforced concrete beams with 2 in. (50.8 mm) of cover. These beams were spring loaded to varying steel stresses. Fig. 4 shows the 25-year data published in 1984; the durability rating was 100 percent at the beginning of the exposure tests. As usual, crack widths were found to be essentially proportional to steel stress. However, the durability of the beams, as rated yearly by observers using a thorough rating system, did not decrease systematically as steel stress increased. There was even the peculiarity that the unloaded beams suffered worse than those loaded to 20, 30, or 40 ksi (138, 207, or 276 MPa).

Performance of the concrete beams was also observed in terms of squared sonic velocity from ultrasonic tests (Fig. 5). Again, increasing steel stress did not reduce performance significantly. Treat Island provides a very severe freeze-thaw exposure, but conditions are only mildly conducive to corrosion. Exposure on a beach in the Red Sea could have given quite different results. Even so, the Treat Island tests warn us not to get over-excited about crack widths and steel stresses.

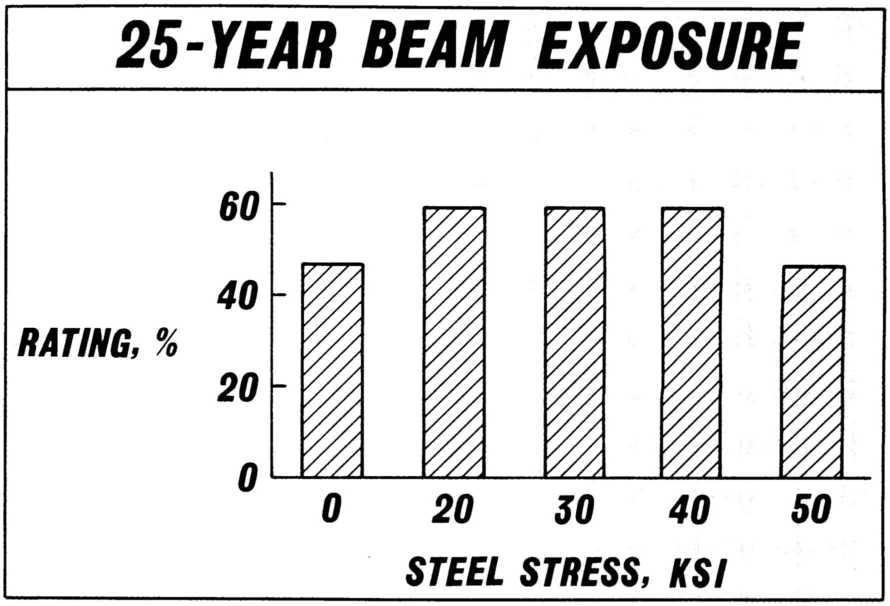

Fig. 4—The effect of steel stress on the durability of concrete beams at Treat Island, Maine.

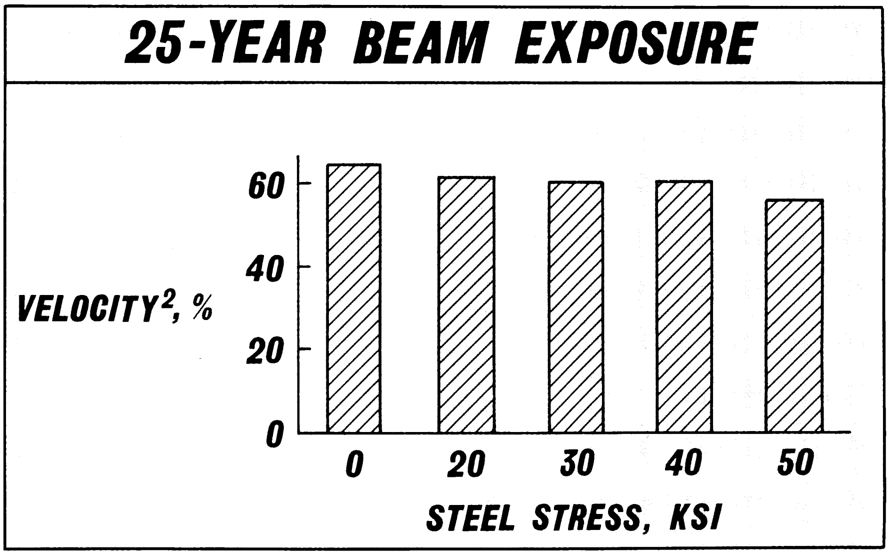

Fig. 5—The effect of steel stress on the sonic velocity squared in concrete beams at Treat Island, Maine.

The Gergely-Lutz expression for crack width (Fig. 6) was chosen as the basis for the 1971 ACI Code provisions for distribution of flexural reinforcement. In preference to other equations available at the time, this choice was made on the basis of simplicity and reasonable accuracy, considering that crack width is inherently subject to wide scatter. For Code purposes (the bottom equation in Fig. 6), the expression was written in a form emphasizing reinforcing details rather than crack width itself.

The limiting values for z, which imply crack widths of 0.013 in (0.33 mm) for exterior and 0.016 in. (0.41 mm) for interior exposure, could not be taken from exposure test data.

1971 ACI BUILDING CODE

GERGELY-LUTZ:

$$w = 0.076\,\beta f_s \sqrt[3]{d_c A} < \begin{matrix}0.013\text{ in.}\\ 0.016\text{ in.}\end{matrix}$$

CODE:

$$z = f_s \sqrt[3]{d_c A} < \begin{matrix}145\text{ kips/in. Exterior}\\ 175\text{ kips/in. Interior}\end{matrix}$$

Fig. 6—The ACI Building Code expresses reinforcement distribution as z, based on the Gergely-Lutz equation for crack width.

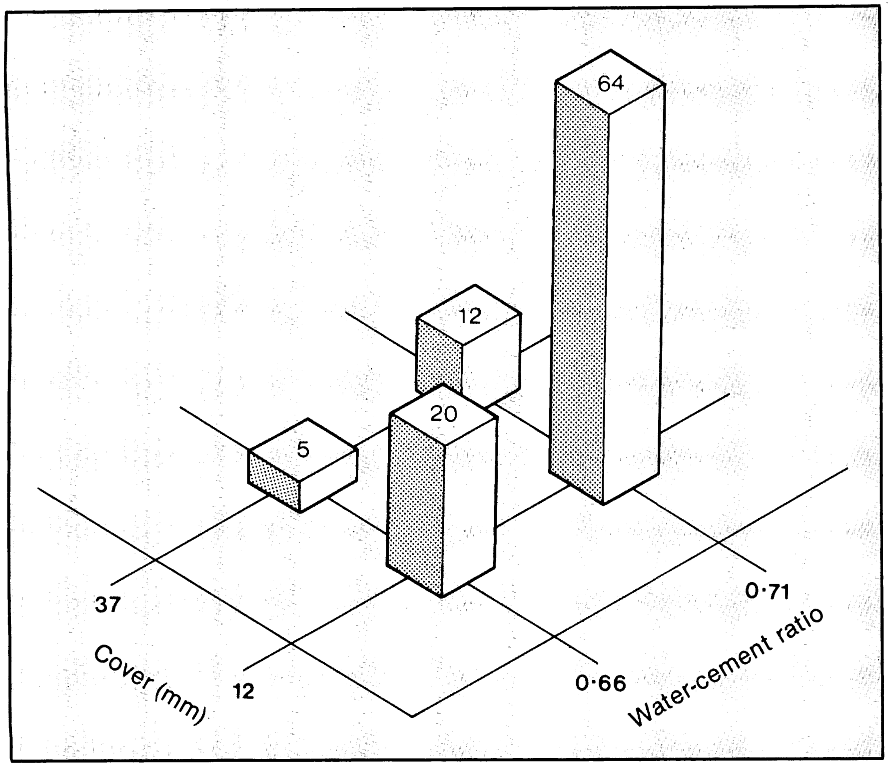

Fig. 7 — Relative corrosion versus cover and water-cement ratio. (Results by Baker, Money, and Sanborn[†])

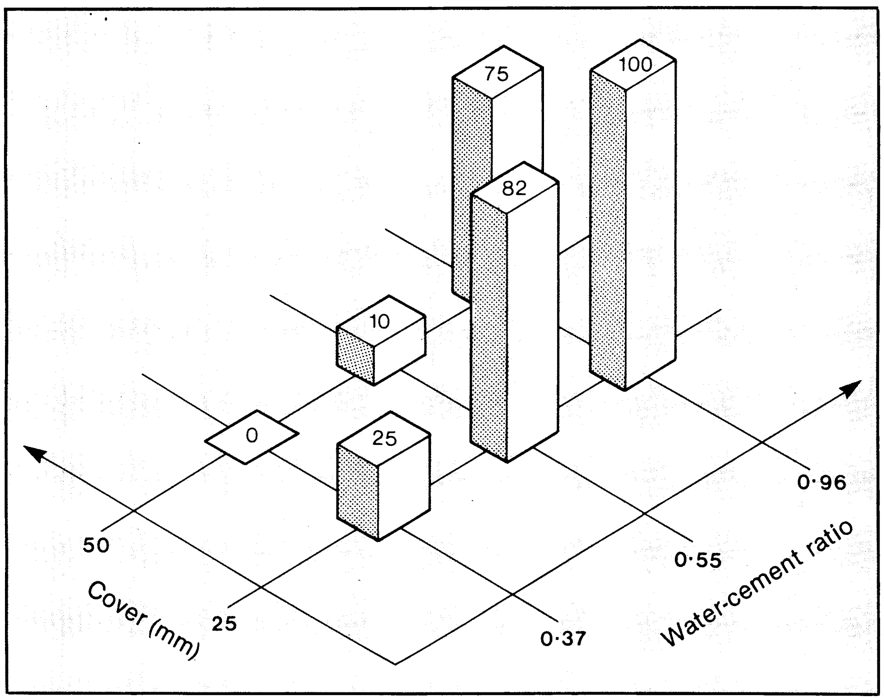

Fig. 8 — Relative corrosion versus cover and water-cement ratio. (Results by Lea and Watkins[‡])

Rather, values of z were derived from reinforcing details that had given satisfactory service in numerous existing structures reinforced with intermediate grade 40,000-psi (276-MPa) steel. These z-values were then studied in terms of the details they produced for higher strength steels.

The Norwegian Veritas Criteria of 1971 were initially written for a one million barrel oil tank built in Norway and towed to the Ekofisk field in the central North Sea. Though prestressed, the tank structure was also heavily reinforced with deformed bars. Cracking was controlled through limitations on reinforcing steel stress depending on three major service conditions: towing, normal service on location, and extraordinary loads (such as 78-foot storm waves). Stress limitations also depended on location within the structure. For example, no membrane tension was permitted in oil chambers, even under storm wave loading. The Ekofisk tank has now provided over 10 years of excellent service with negligible maintenance.

The initial Veritas criteria have been refined to cover almost twenty large offshore structures. The concepts involved were also reflected in the "Guide for the Design and Construction of Fixed Offshore Structures" reported by ACI Committee 357 in 1978.*

Concluding this brief survey of the code provisions, originally written over thirteen years ago, it may be noted that corrosion of steel is considered in design by six major means:
1. Quality concrete
2. Adequate cover
3. Limit chlorides
4. Sound reinforcing details
5. Steel stress limitations
6. Prestressing in some cases.

Design Considerations by Andrew W. Beeby

Beeby: Design codes aim to protect reinforcement against corrosion by three methods: the specification of minimum covers, minimum qualities of concrete, and maximum crack widths. Just how these factors are specified varies from code to code.

For example, concrete quality may be defined in terms of strength only, or possibly, as in the latest revision of the British code, in terms of water/cement ratio, cement content, and strength. Similarly, crack width limits are defined in different ways. Sometimes it is done explicitly and sometimes the limits are implicit in detailing rules.

Before discussing crack control further, I wish to reinforce something which was said by both Dave Manning and Dr. Hognestad: this is the vital importance of cover and concrete quality in providing protection to the steel. Fig. 7-9 summarize the results

*ACI Committee 357, "Guide for the Design and Construction of Fixed Offshore Concrete Structures," (ACI 357R-78)(Reaffirmed 1982), American Concrete Institute, Detroit, 1978, 26 pp.

†Baker, E. A.; Money, K. L.; and Sanborn, C. B., "Marine Corrosion Behavior of Bare and Metallic-Coated Steel Reinforcing Rods in Concrete," *Chloride Corrosion of Steel in Concrete*, STP-629, ASTM, Philadelphia, 1977, pp. 30-50.

‡Lea, F. M.; and Watkins, C. M., "The Durability of Reinforced Concrete in Sea Water," *Research Paper* No. 30, National Building Studies, Department of Scientific and Industrial Research, Her Majesty's Stationery Office, London, 1960.

of three major investigations of these factors.

In each figure, the height of the columns indicates the relative amount of corrosion that had occurred by the end of the exposure period. The horizontal axes indicate the cover and the water/cement ratio, which is used as a measure of concrete quality. These results leave no doubt about the beneficial effect of increasing the cover or the concrete quality.

No such clear relationship can be demonstrated between crack width and amount of corrosion. Indeed, the possible existence of such a relationship has been a subject of debate over many years and it has yet to be finally resolved. Personally, I am a believer in the theory, outlined by Dave Manning earlier, that no relationship exists between crack width and amount of corrosion.

However, it is not my intention to argue this point at this time. I intend to show, even if there is some increase in corrosion with increased crack width, the rules included in codes of practice for crack control are still likely to be harmful rather than helpful.

If exposure tests are carried out on cracked specimens and, at the end of the exposure period, the amount of corrosion at each crack is measured, a frequency distribution can be produced which, for any particular size of crack, indicates the probability of a particular amount of corrosion being exceeded.

Fig. 10 shows such a distribution obtained by Schiessl.[†] Fig. 11 illustrates how schematically, having decided upon an acceptable limit to the amount of corrosion, the probability of occurrence of more corrosion than this can be estimated.

The next stage in the argument will be pursued by means of an example. Consider a slab for which the calculated crack width, using the formula in the commentary to the ACI code, is 0.022 in. (0.56 mm). Let us say that the slab is reinforced with #6 bars. For the particular environment considered, the required crack width is 0.013 in. (0.33 mm). What options are open to the designer?

One possibility, which will be called Option I, is to decrease the steel stress by increasing the steel area. Increasing the bar size from #6 to #8 gives almost exactly the required result, but in order to achieve this, the area of steel has had to be increased by 54 percent above that required for strength. For obvious economic reasons, I believe that this is not an option that will normally be taken.

The alternative possibility (Option II) is to use smaller bars at closer spacings while keeping the area the same. In this case, the required calculated crack width will just be obtained if the bar size is halved (to #3 bars) and four times the number of bars are used. What has actually been achieved here is that the number of cracks has been increased by a factor of 1.7 and the number of bars by 4, thus increasing the number of points where a crack intersects a bar by a factor of $1.7 \times 4 = 6.8$.

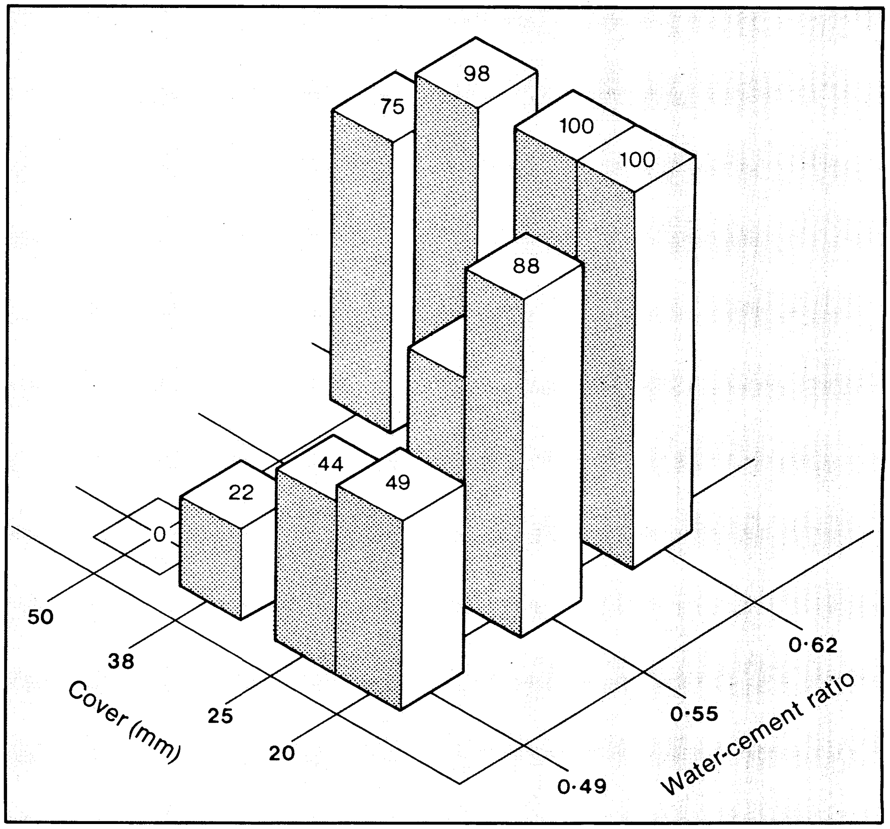

Fig. 9 — Relative corrosion versus cover and water-cement ratio. (Results by Houston, Atimtay, and Ferguson)*

If corrosion is independent of crack width, then we have made our slab 6.8 times more prone to damage by this change in detailing. In fact, it can be seen that for the change in the reinforcement to be an improvement, the reduction in crack width from 0.022 to 0.013 in. (0.56 to 0.33 mm) must reduce the risk of excessive corrosion by more than 85 percent.

While it may be argued by some people that crack width has an effect on corrosion, none suggest that the influence is anything like sufficient to meet this requirement. There seems no way in which reducing crack widths by rearrangement of the reinforcement could lead to an improved durability. Only reducing the crack widths by the uneconomical Option I could hope to do this.

There are further disadvantages in controlling cracking by Option II. Firstly, since crack width is related to cover, attempts to control cracks encourage the use of small covers, whereas an increased cover, even with an increased surface crack width, gives better durability. Secondly, the use of small, closely spaced bars leads to details which are difficult and expensive to construct and which, because of difficulties in compacting the concrete around the bars, may well lead to lower quality concrete with a consequent increase in corrosion risks.

In summary, the type of crack control provisions included in most codes of practice are more likely to reduce durability than they are to improve it.

Finally, I would like to make one minor clarification of my position. There may be, and in many cases there are, good reasons for controlling the widths of cracks other than corrosion protection.

For example, it may be necessary to control widths to avoid leakage or because large cracks would impair the appearance of a structure. My remarks relate only to the value of crack control as a corrosion protection measure.

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*Houston, J.; Atimtay, E.; and Ferguson, P.M., "Corrosion of Reinforcing Steel Embedded in Structural Concrete," *Research Report* No. 112-1-F, Center for Highway Research, University of Texas at Austin, 1972.

†Schiessl, P., "Admissible Crack Width in Reinforced Concrete Structures," *Preliminary Reports* II, Inter-Association Colloquium on the Behavior in Service of Concrete Structures, Liege, 1975, Contribution II, 3-17.

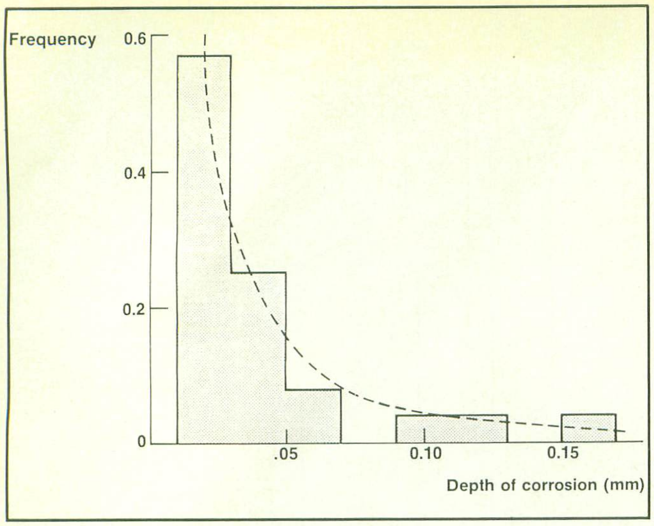

Fig. 10 — Distribution of corrosion for 0.2 mm cracks.

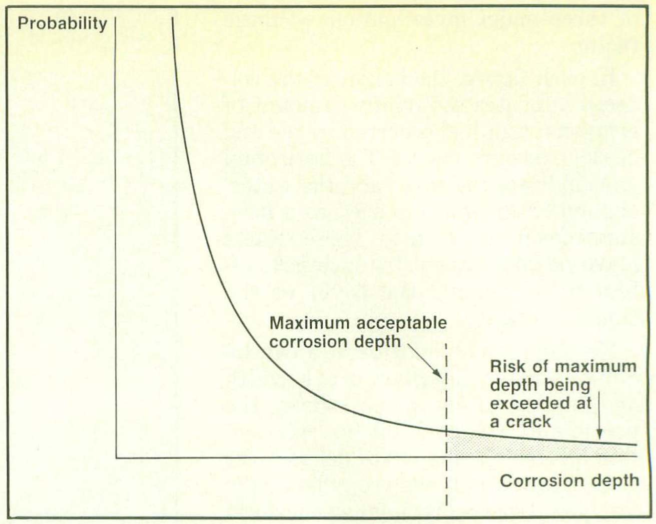

Fig. 11 — Probability of corrosion versus corrosion depth.

Crack Control of Sanitary Engineering Structures
by Paul F. Rice

Rice: Three principles have been developed over the past few years by ACI Committee 350 to limit crack width using Grade 60 reinforcement for design of more economical sanitary engineering structures intended to retain liquids, some aggressive in reaction with both steel and concrete. These are:

—Minimum percentages of "shrinkage and temperature" reinforcement have been related to distance between construction (or control) joints.

—Under working stress design for flexural tension, bar size, spacing, cover, and working load stress have been related to calculated $z \simeq 115$.

—For strength design, service load stresses are automatically reduced by simple sanitary exposure factors applied to the required strength (U) for reinforcement in tension to achieve the approximate z values desired.

The early curing environments to which reinforced concrete structures are exposed immediately after final set occurs include many variables. Temperatures of concrete and air, humidity, actual water-cement ratio, free water leaking out versus rising, unsymmetrical end or continuous restraints (top, bottom, or both), admixtures, casting procedures, vibration, and congested reinforcement are usual factors affecting shrinkage.

Most of these effects are unpredictable, occurring at random, and not susceptible to close control by analysis, design, or specifications. A simplified mechanism to account for the most usual conditions of horizontal shrinkage cracking in cantilevered walls was adopted by Committee 350 as a common basis for selecting minimum amounts of Grade 60 shrinkage and temperature steel. (See Fig. 12)

Based upon collective observations of shrinkage and temperature cracking (vertical) and collective experience affecting crack formation by varying amounts of horizontal steel and joint spacings, Committee 350 set a lower limit at joint spacings of 20 ft (6.1 m) or less.

An upper limit at spacings of 60 ft (18.3 m) or more was adopted from earlier research and comparison to minimum steel employed in continuously reinforced concrete pavements. The curves shown in Fig. 13 depict these limits and the steel requirements for intermediate joint spacings for both Grades 40 and 60.

Reductions for use of sizes of shrinkage and temperature reinforcement with diameters 0.625 in. (16 mm) or less, generally permitted in ACI 318-83, are not shown since these sizes are not usually practicable for heavy walls or slabs used in major sanitary structures.

In working stress design, crack control for use of Grade 60 reinforcement arranged in single layers for slabs or walls as flexural members involves the relationship of three variables: service load stress, bar spacing, and cover. (See Fig. 14)

Committee 350 employed the crack control formula in ACI 318-83 with an average value, $z \simeq 115$, as a ready-made solution to relate the three variables for working stress design. It was, however, necessary to establish an upper limit of 2 in. (51 mm) for cover or 2.5 in. (63.5 mm) for d_c. This limitation is more or less implicitly established by minimum cover requirements in ACI 318-83.

Additional cover required for long-life protection against exposure to aggressive chemicals in sanitary structures may be regarded as "sacrificial protection." Service load stresses may vary from 24 ksi (165 MPa) for all bars at the committee recommended maximum spacing of 12 in. (305 mm) with usual cover employed in sanitary engineering structures to a maximum of 30 ksi (207 MPa) at lower spacings. (See Fig. 15)

In strength design, additional comparisons of calculated z factors for values of d_c from 0 to 3 in. (0 to 76 mm) convinced Committee 350 that other factors influencing corrosion resistance override the value of a simple limit for z.

Limitations on d_c are implicit in the use of Section 10.6 in ACI 318-83. The ACI 318-83 Commentary explanation alone is insufficient for application of the z-equation with very large or small values of d_c.

For example, application of Eq. (10-4) in ACI 318-83 to typical sections of continuously reinforced concrete pavement yields z-factors approximately equal to 300. Close up photographs of bars removed from such pavement after 20 years of service in Illinois show no rust although they were exposed to an estimated yearly total of 7 tons (4 Mg) of deicer salt per 2-lane mile.

Rusting did occur on transverse bars under longitudinal joints where joint movement made joint sealing ineffective. Approximately 15,000 equivalent 2-lane miles (24,135 km) of continuously reinforced concrete pavement built in the last 25 years, still in service, indicate that the Illinois experience is probably typical.

A number of questions are raised about β in the Commentary equation for W. β is built into the 318-83 equation for z as 1.2 which should alert

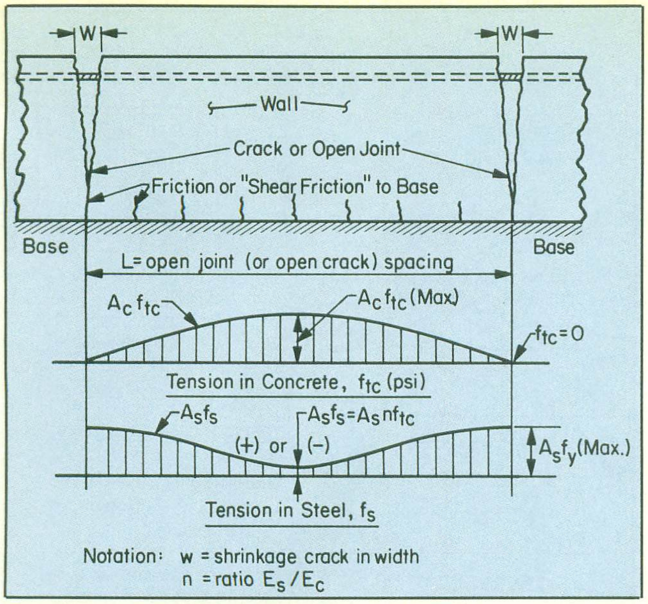

Fig. 12 — Simplified mechanism for shrinkage cracking.

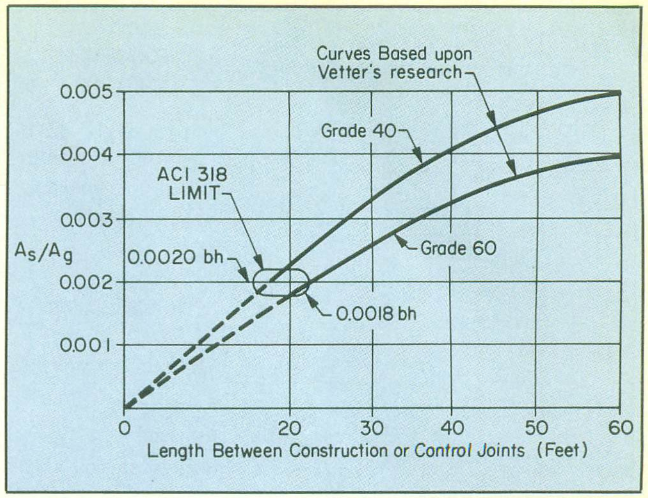

Fig. 13 — Extrapolation of ACI 318 crack control to sanitary engineering structures.

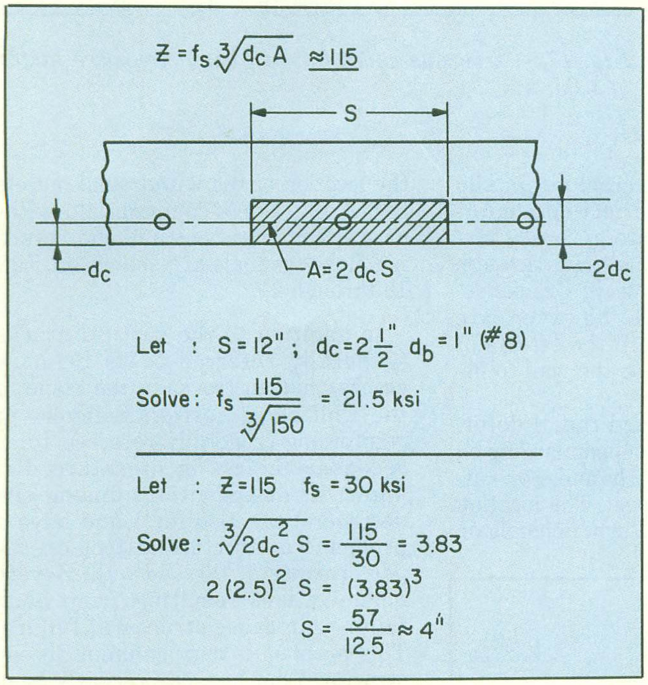

Fig. 14 — Crack control calculations.

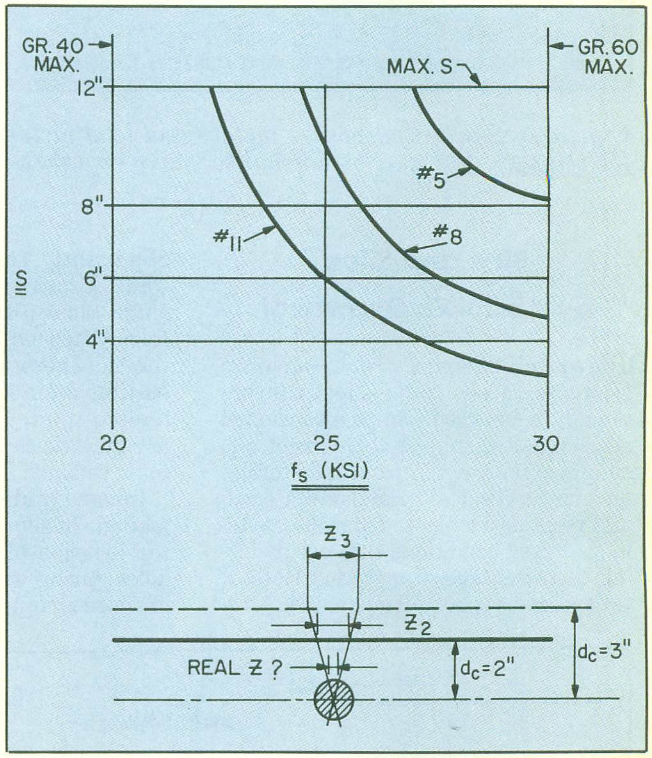

Fig. 15 — Bar spacing versus steel stress for z = 115.

users of the Code that the z limits are not "hard" or exact. The Commentary suggests a shift of all z limits by 1.2/1.35 for floor slabs.

Floor slabs in ACI 318-83 probably means thin slabs, 3 to 6 in. (76 to 152 mm), and not, for example, 18 in. (457 mm). Sanitary wall thicknesses on major units range from 12 to 36 in. (305) to 914 mm). If 1.2 is beam average for positive moment bottom bars, then the probability arises that it is different for top bars in T-beams, more like 1.35.

The problem of automatically attaining a reasonably low z-factor near the value of $z \simeq 115$ was finally solved by the creation of a sanitary exposure factor (Fig. 16). This factor, 1.3, is applied to all load combinations, U, creating tension in the flexural and shear reinforcement.

For "pure tension" only, as in circular tanks, the sanitary exposure factor recommended is 1.65. Fig. 17 shows z-factors computed for various d_c (cover thickness) using service load stresses within the range desired by ACI Committee 350.

In conclusion, sanitary engineering structures designed to retain liquids, which attack both steel and concrete aggressively, and to serve for periods of 50 to 100 years, deserve special attention above and beyond ordinary Building Code requirements.

Committee 350 has labored for nearly 20 years to develop these special requirements based upon experience and to adapt them to use Grade 60 reinforcement for maximum economy in reduced tonnage of the reinforcement required.

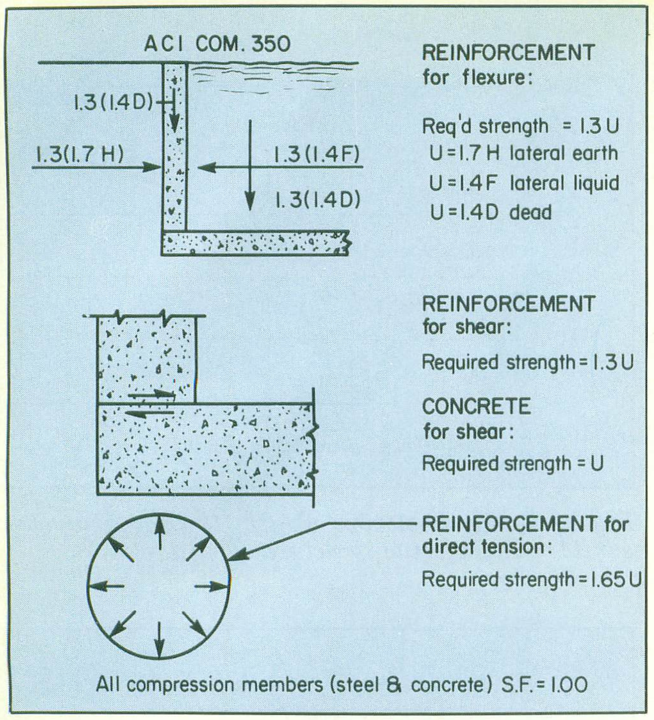

Fig. 16 — Sanitary exposure factors and load factors for strength design. (For normal sanitary exposures)

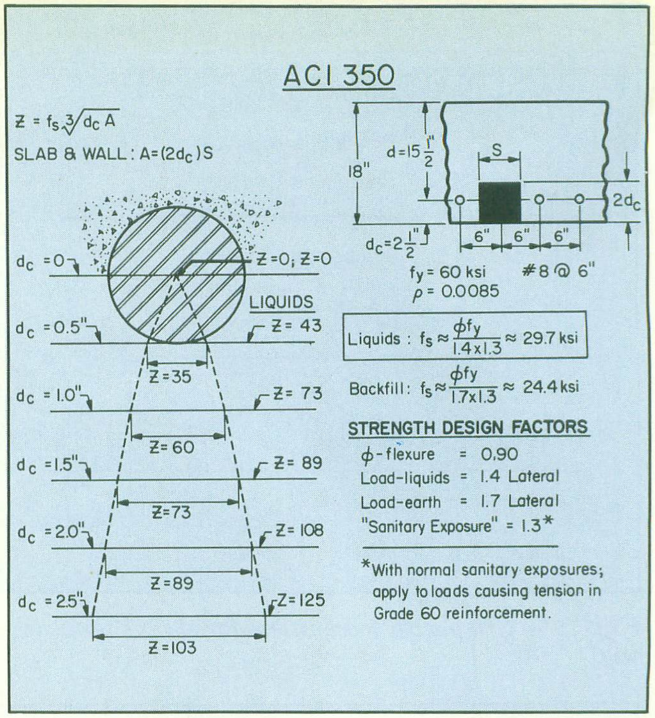

Fig. 17 — z versus cover for sanitary exposure factor of 1.65.

Bins and Silos
by Abdul Q. Ghowrwal

Ghowrwal: *Common structural causes of cracks in silo walls*—Most conventionally reinforced and post-tensioned silo walls are subjected to direct circumferential tension from the storage and withdrawal of granular material, and bending moments from the asymmetric flow conditions that occur during the discharge of material. Both of those loading conditions cause predominantly vertical cracking in silo walls. Cracks from direct circumferential tension alone are generally uniformly spaced and may run through the thickness of the wall. Cracks resulting from flexure, however, are limited to areas where the effects of asymmetric flow cause the wall to deform radially.

Inward and outward radial deformations of silo walls depend largely on the location of flow channels or ratholes during withdrawal. The location of these channels, in turn, depends on the location of the withdrawal outlets in the silo. Some common silo withdrawal configurations and the resulting flow channels are shown on Fig. 18 through 21.

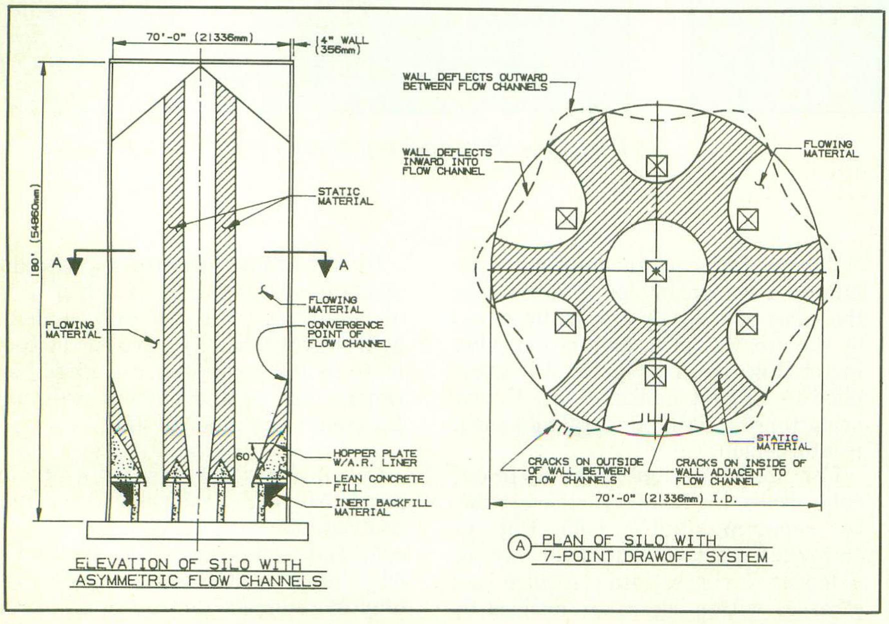

Fig. 18—Seven point silo drawoff system.

In addition to the vertical cracks (generally referred to as primary cracks), hidden cracks in the plane of the wall along horizontal layers of reinforcing (generally referred to as secondary cracks in literature, but known as delaminations among silo designers) can also form and have a pronounced structural effect on the performance of the silo wall. Severe delaminations resulting from bond failure can cause structural failure. This problem is very common in silo structures but has only recently been determined to have attributed to silo wall failures. The primary cause of bond failure is high tensile stress in the reinforcement as a result of high bending moments combined with axial tensile forces. Accurate prediction of the magnitude of the bending moments resulting from asymmetric flow conditions has only recently been possible.

Crack width—Since most silo walls are designed for conservative overpressures, the width of cracks, caused only by axial tension due to uniform lateral pressure from storage and/or withdrawal, has generally been minimal and predictable. If the bending moment is small, the level of stress in the reinforcing produced by uniform tension alone is not significantly different from what can be expected at the design stage. However, the levels of stress in the reinforcement greatly increase when walls are also subjected to flexural stress. Fig. 22 and 23 illustrate a typical silo wall subjected to both bending and circumferential tension. The formation of cracks and the calculation of crack width under this combined loading is unique to the design of silo walls, since few other structures are subjected to this loading combination.

Until a few years ago, it was virtually impossible to compute the magnitude of bending moments in silo walls subject to asymmetric flow. This was due to a lack of understanding of the effect of bulk solids on the structural performance. As a result, many silo walls designed only for uniform pressures, yet subjected to bending moments, developed crack patterns that were not predicted. Some of these cracks appeared to be much wider on the inside faces of silo walls than on the outside faces. Two principle explanations for this difference have been accepted. First, the bending moments producing tension on the inside face are 30 to 100 percent greater than the bending moments producing tension on the outside face of the wall. Second, the cracks grow wider on the surface of the inside face of the silo wall due to abrasion from the flow of granular material.

The bending stiffness of a silo wall is reduced from 30 to 40 percent when cracked. This causes the moments producing tension on the inside faces of the wall to be redistributed to outside faces or to other locations where cracking is less severe. As a result of this redistribution, the cracks on the outside face of the wall also grow wider and more significant. Therefore, control of cracking in silo walls is necessary for two principal reasons:

1. Silos intended to store and handle hygroscopic material must have crack widths controlled to prevent ambient moisture from permeating through the cracks and into the stored material.
2. Cracks on the inside face of silo walls are known to have abraded wider and deeper from the flow of granular material. Uncontrolled and abraded cracks on the inside face may also allow the moisture present in certain granular materials stored in the silo to permeate into the wall. If the stored materials contain chlorides or other soluable salts, corrosion of the reinforcing in the wall is almost certain to result. Some highly abrasive bulk solids are known to have widened cracks from 0.04 in. (1.02 mm) prior to abrasion to almost 1 in. (25.4 mm) after abrasion.

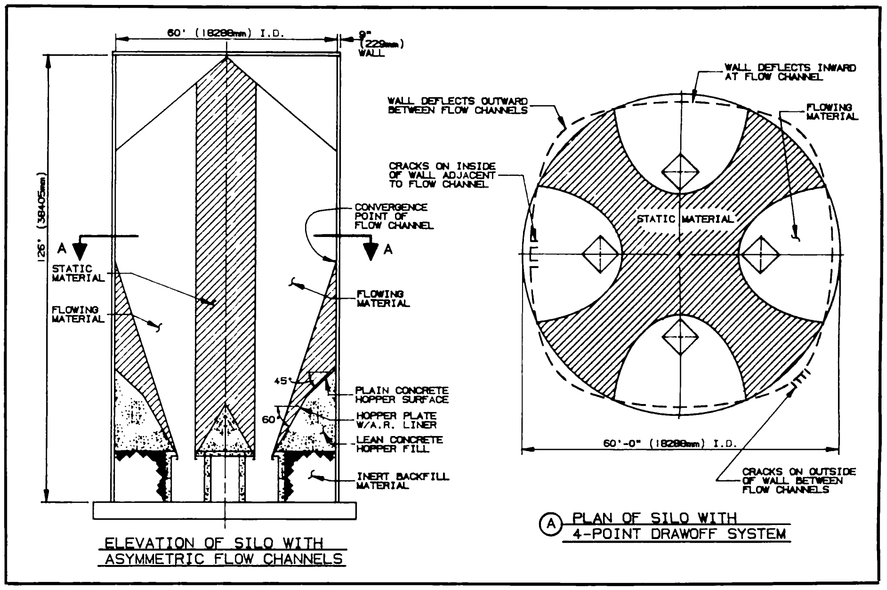

Fig. 19—Four point silo drawoff system.

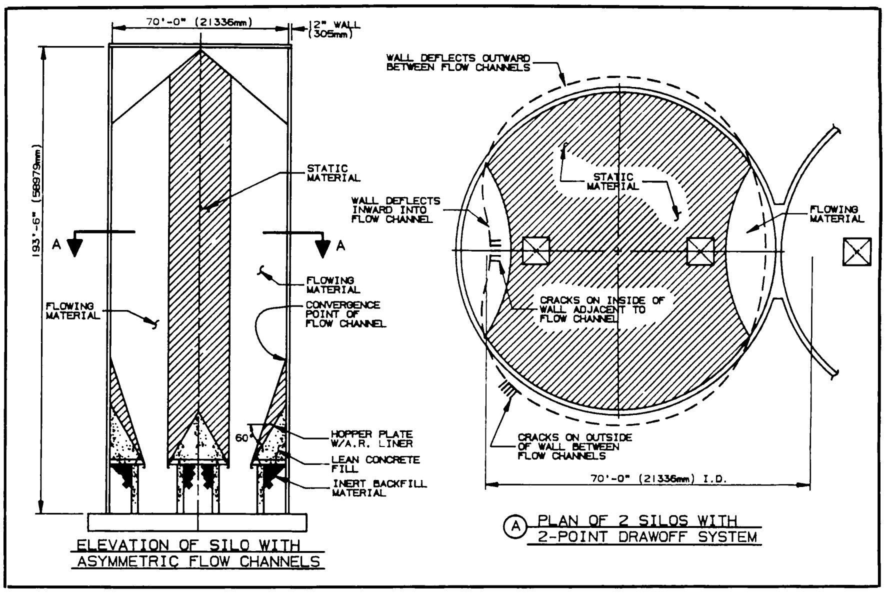

Fig. 20—Two point silo drawoff system.

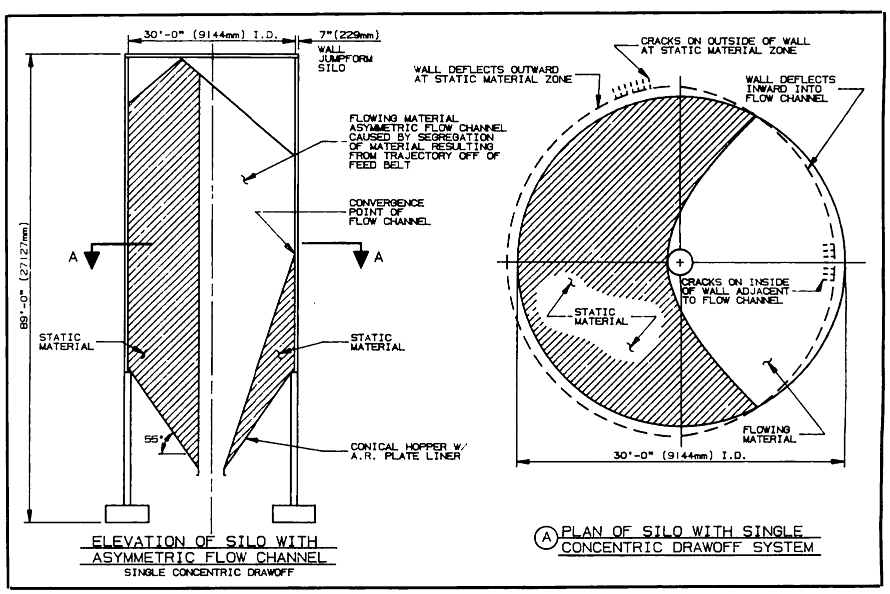

Fig. 21—Single concentric silo drawoff system.

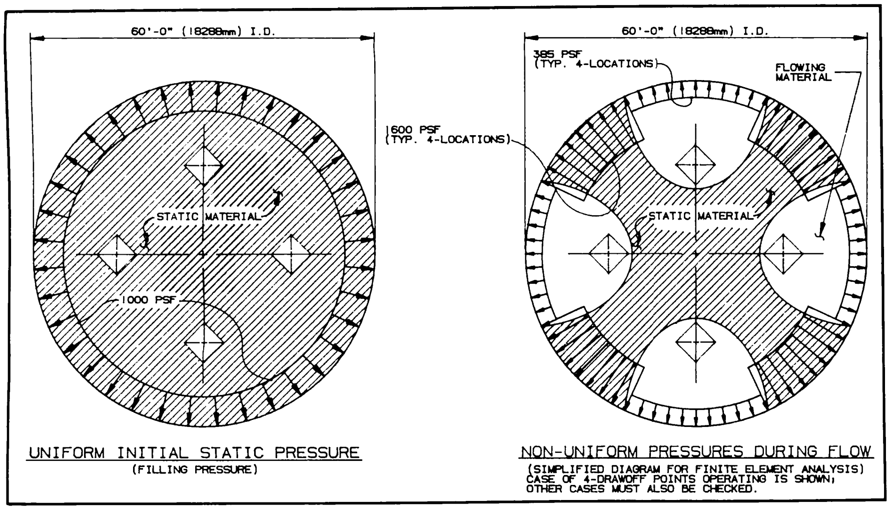

Fig. 22—Horizontal pressures on silo wall when full and during drawoff.

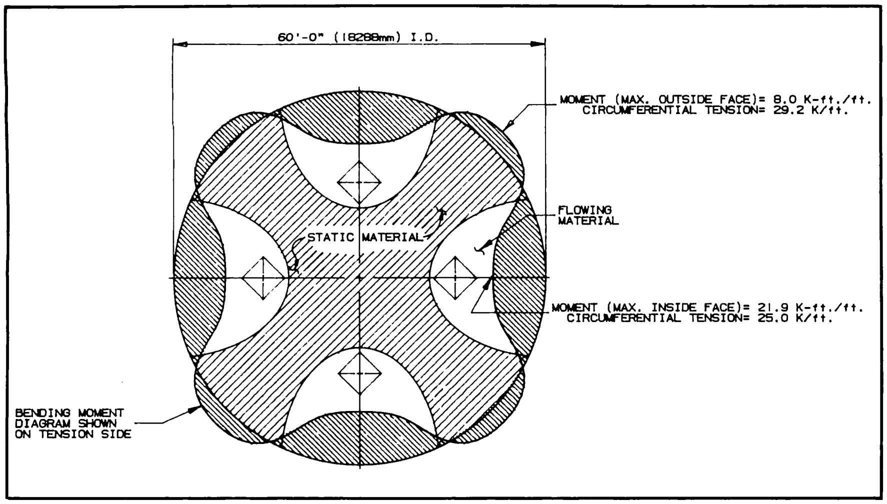

Fig. 23—Bending moments in silo wall during drawoff.

Clearly, excessive cracking in silo walls is undesirable. It reduces stiffness, enhances the possibility of deterioration and corrosion, and ruins the silo's appearance. Crack widths must meet the limits accepted in the industry. This requires accurate prediction of the bending moment caused by the withdrawal of materials, since the primary variable affecting crack width is the tensile stress and strain in the reinforcing steel.

Other variables affecting crack width in silo walls are the thickness of concrete surrounding each bar; the area and arrangement of the tensile steel reinforcement (inside face and outside face placement); and the spacing of reinforcing bars.

Cover—Adequate concrete cover in reinforced concrete silo walls is very important. The inside faces of many silo walls are subjected to abrasion from the flow of granular material. Although no specific data is available, field observations indicate that 1 to 2 in. (25.4 to 50.8 mm) of erosion of concrete surfaces can occur in just a few years, from the abrasion of coals, clinkers, and cullets.

Another reason for maintaining adequate cover is that bond strength increases with cover thickness. Linear relationships appear to exist between bond strength and concrete cover for bar sizes #7 to #11. Field observations indicate that, regardless of size, reinforcement placed near either face of the wall has more readily detectable delaminations than that placed at the center of the wall. This agrees with the conclusion that bond strength is directly proportional to splitting stress and to the amount of cover; therefore, bond failure can be directly related to the amount of cover. In conclusion, the amount of cover on principal reinforcement in silo walls is very important in terms of durabiity and structural performance.

Corrosion—Reinforcement needs greater protection against corrosion in silo walls than in other types of structures. Due to the fact that almost the entire circumferential length of the silo wall is subject to axial tension, any part of the wall affected by corrosion can have severe consequences on the structural performance of the entire silo.

It is well known that chloride greatly increases the likelihood of corrosion. In silos, several failures have been attributed to the presence of chlorides; the only dispute has been how the chlorides entered the wall.

The major sources of chlorides are chloride-containing admixtures, and bulk materials stored in the silos which themselves contain chlorides. The use of calcium chloride to accelerate the setting time of concrete is not unusual, especially in winter slipform construction. This may create variable concentrations of chlorides in the silo walls and increase the possibility of corrosion. Bulk materials containing chlorides, such as certain coals, and a sufficient amount of free moisture have contributed to the penetration of chlorides into silo walls.

The penetration of salts to reinforcing steel and other embedded items is affected by the thickness of the concrete cover. Thicker covers are better than thinner covers. However, if the stored materials contain large amounts of soluble salts, even an average cover of 2 to 3 in. (50.8 to 76.2 mm) may not be adequate.

Achieving a zero chloride content for the mix is impossible in practice. Chlorides are among the most abundant materials on earth and are present in variable amounts in all concrete ingredients. The proper approach is to limit the total chloride in the mix (i.e., in the aggregate, cement, mixing water, and admixtures) to a value less than what would promote corrosion.

Conclusion—Due to the assymetrical loading conditions that exist during material withdrawal, and the abrasive action of flowing granular materials, it is extremely important to consider crack width, cover, and corrosion in the design of reinforced concrete silos.

Discussion

Arthur G. Maylan (Florida Department of Transportation, Tallahassee, Florida): The new Sunshine Skyway Bridge is currently under construction in Florida. It is on an accelerated completion schedule because the original bridge was damaged due to a tanker hitting one of the piers, bringing down a large portion of the superstructure. The bridge has bell-shaped footings with 4 in. (102 mm) of cover over the reinforcing steel. After the footings were cast, some cracks appeared which measured about 0.5 mm (0.02 in.) wide by several feet long. Because a number of individuals believe David Manning's Theory No. 1, that cracks significantly reduce the service life of the structure, we found ourselves trying to explain that the service life of the structure would be more than a few years. I feel that more people need to understand and accept Theory No. 2; that the presence of cracks does not necessarily mean the structure will undergo rapid corrosion in just a few years.

Beeby: I strongly agree that it would be most useful if both engineers and nontechnical people could be made to understand that cracking is not necessarily serious and that, in fact, reinforced concrete is designed on the assumption that it will crack. In England we suffer from exactly the same sort of problem that Mr. Maylan has described; as soon as anyone finds a crack, no matter how small, serious doubts are cast on the adequacy of the structure and only extensive repair operations will satisfy the client. This misunderstanding is causing a great deal of money to be spent unnecessarily on repair work. Quite a number of people are now working in many countries to publicize a more enlightened view of the seriousness of cracking. In the end, I believe this will have an effect, but it is a slow process.

Brian Hope (Queens University, Kingston, Ontario, Canada): I must comment on the last two discussions. David Manning pointed out that the orientation of the crack with respect to the reinforcing bar is very important. If the crack is transverse, the zone of rust is relatively small. However, if the crack is along the bar, a different situation exists entirely. While the mere presence of cracks may not be detrimental, the orientation of those cracks must be taken into consideration.

Manning: I agree that this is an important point. There is overwhelming evidence to suggest that where cracks are transverse to the reinforcement, the crack width is of secondary importance, and the zone of the bar which is affected by corrosion fits in very well with the rule of thumb of three bar diameters. The amount of corrosion that takes place before the corrosion effectively ceases is not of structural consequence. Quite the reverse is the case when the crack coincides with and is directly above the reinforcing steel. Then the number of sites available for corrosion is much greater, and there is no means of inhibiting or confining the corrosion process. As a result, the corrosion processes continue, the corrosion products are deposited in the crack, and the associated expansion causes the crack to widen. That is why I made the point in my presentation that corrosion is, in fact, accelerated when the crack coincides with the reinforcing bar.

"To place the importance of cracking into perspective, we must look at it as a modification of the overall permeabiity of the concrete between the steel and the surface. This concrete generally has 3 to 6 percent air voids and is highly permeable even without the crack."

—Jonathan Wood

Beeby: I agree completely with Dave Manning that cracks which run along the line of a bar pose a more serious corrosion risk than do cracks transverse to bars. Nevertheless, there is still no evidence to suggest that, even in this case, the amount of corrosion will be related to the crack width.

Jonathan Wood (Mott, Hay and Anderson, Croydon, England): To place the importance of cracking into perspective, we must look at it as a modification of the overall permeability of the concrete between the steel and the surface. This concrete generally has 3 to 6 percent air voids and is highly permeable even without the crack. If the concrete is allowed to dry out before the cementing process is fully developed, capillary pathways are left through that cover to the steel, and they can be more damaging than the cracks. So we must not overlook the permeability of the concrete cover and the way the crack modifies the permeability.

Robert E. Philleo (Consulting engineer, Annandale, Virginia): One speaker said we should express Theory No. 2 in terms laymen could understand. I would like to have it expressed in terms that I can understand. As I interpret the two theories, Theory No. 1 says that when there are cracks, there is corrosion at the cracks, and Theory No. 2 says that when there are cracks, there is corrosion all over. I would like to have that clarified. I agree that permeability must be important, but I am surprised that it does not seem to be as important as it should be. If you decrease the water-cement ratio from 0.50 to 0.40, there is a tremendous decrease in permeability. The difference in the corrosion potential is significant, but not nearly as great as the permeability effect. I would also like to know why the quality of the concrete is not a more important factor than it is.

As chairman of the ASTM committee that writes the portland cement specifications, I am interested in the importance of a factor in cement itself that was mentioned earlier, that is the C_3A (tricalcium aluminate) content. The benefits of a high C_3A content are often cited, presumably because it supports and maintains a higher pH level. But there are more people out there who want a low C_3A content than who want a high C_3A content — for quite other reasons than corrosion. I would like to hear some discussion on how important it is to have a high C_3A content, and whether or not anybody wants cement in which they are assured of some lower limit on C_3A content.

Beeby: Basically, Theory No. 2 suggests that the function of a crack is to allow carbonation or, more importantly, chlorides to penetrate to the reinforcement. When either of these reach the bar surface, corrosion may start (provided adequate moisture and oxygen are available). The cracks therefore act as corrosion initiators.

Once corrosion starts, an electrolytic cell is set up (similar to a battery). The bar surface becomes differentiated into two types of areas: anodic areas where carbonation or chlorides have reached the steel and where metal will be corroded, and cathodic areas where oxygen and water will be combined to form hydroxyl ions. These hydroxyl ions may be considered to flow through the pore water from the cathodic areas towards the anodic areas, where they will combine with the metal ions to form rust. The raw materials required to make the corrosion process operate are oxygen and water. These are used at the cathodic areas, which are the areas of bar away from the cracks, surrounded by sound concrete. Once corrosion has started, no new materials need to be provided to the anodes for it to continue. Hence

". . . it would be most useful if both engineers and nontechnical people could be made to understand that cracking is not necessarily serious and that, in fact, reinforced concrete is designed on the assumption that it will crack."

—Andrew W. Beeby

the cracks play no further part in controlling the corrosion process once it has been initiated.

Darwin: Before we move on, is there someone who would like to discuss C_3A content?

Hognestad: In the absence of a cement chemist arising, I will say a few words as a structural civil engineer, an amateur in cement chemistry. As chlorides enter the concrete in sea water used in the mix itself, or from the outside, available C_3A will consume some of the chlorides and render them insoluble. For example, I have recently examined concrete made with sea water and a Type V cement that has virtually zero C_3A. In this concrete, the soluble and total chlorides are almost equal. In other words, nothing was done to reduce the amount of available chloride.

Some investigations were made by the U. S. Corps of Engineers during and after the two world wars where sea water was purposely used to mix concrete for military purposes and for short durations. It was found that reinforced concrete submerged permanently suffered no damage due to the chlorides in the concrete, even in tropical waters. The problem occurs when we get out of the water, out of the ground, and into an atmosphere, where both moisture and oxygen are available. Then and only then do we have corrosion difficulties.

Frank Heger (Simpson, Gumpertz & Heger, Inc., Arlington, Massachusetts): Regarding the equation for crack width in the ACI Building Code, we have had a lot of opportunity to examine the application of equations like this to concrete pipe. The industry routinely tests concrete pipe to a 0.01 in. (0.25 mm) crack width, and our firm has had the task of trying to develop methods of predicting the 0.01 in. (0.25 mm) crack load of concrete pipes. We have modified equations like the z-equation in the Code by going back to the original research. This was necessary because applying that equation directly from the Code to a slab-type structure or to pipe will give you incorrect results.

Most concrete pipe uses welded wire reinforcement with relatively close circumferential spacing [2 in. (51 mm) spacing is very common]. With spacing that close, stresses are very high at the 0.01 in. (0.25 mm) crack level, which is narrower than the crack widths used in the Building Code. The stresses in the steel are dependent upon the percentage of reinforcing steel, at least in these structures. For typical percentages of reinforcing steel in pipe (½ percent and less), the stress levels are very high before a 0.01 in. (0.25 mm) crack is reached.

If you increase the percentage of reinforcing steel to the so-called Class IV level of pipes, the stress level reached at a 0.01 in. (0.25 mm) crack drops significantly, down to 35,000 to 40,000 psi (240 to 275 MPa), depending on the kind of reinforcement. The z-equation may work perfectly well for typical cases in building construction, but you cannot extend it to pipe without difficulties. I suggest that those who want to do this look into what has been done in the concrete pipe industry. We have written a paper that gives some more information on this.*

Robert E. Price (Openaka Corporation, Inc., Denville, New Jersey): We have been engaged recently in the investigation of a number of prestressed concrete pipes. Prestressed pipe is made by wrapping a prestressed wire around a concrete core and then covering it with a 0.75-1.0 in. (19-25 mm) mortar coating. We have investigated corrosion in buried pipe which has been in service from 10 to 20 years. The wires are spaced from 0.5 to 1.0 in. (13 to 25 mm) apart in most structures. We have found that cracks running along the wires, from 0.002 to 0.005 in. (0.05 to 0.13 mm) wide, generally do not produce corrosion in the wires themselves. But when they cross the prestressed wire, corrosion begins to occur. This seems to contradict what Dr. Manning says — that corrosion is more serious for cracks along the reinforcement than across it.

It is presumed that the corrosion of steel and concrete is under cathodic control. In many cases we see large cracks and delaminations in the cover coat of these pipes, which have produced absolutely no corrosion because the pipes have been below the water table. However, if the pipe is only half submerged in the ground water, the top is liable to be severely corroded, particularly if the ground water rises and falls. Much of this directly contradicts what has been presented, but these are the experiences we have had.

". . . anyone interested in protecting concrete structures from corrosive and marine environments should investigate the use of ferrocement sheet. After 20 years of using ferrocement produced by shotcrete laminating methods, we have found that a 3 mm (0.12 in.) cover will protect the mesh against corrosion, and that several layers of the mesh will in turn protect anything inside that.

—Martin E. Iorns

Martin E. Iorns (Ferrocement Laminates, West Sacramento, California): I suggest that anyone interested in protecting concrete structures from corrosive and marine environments should investigate the use of ferrocement sheet. After 20 years of using ferrocement produced by shotcrete laminating methods, we have found that a 3 mm (0.12 in.) cover will protect the mesh against corrosion, and that several layers of the mesh, whether it is expanded metal lath or another type, will in turn protect anything inside that. So I think we are about ready to throw the book away on cover assignments and begin using thin shell concrete in corrosive environments.

The November 1983 issue of *Concrete International* was devoted to ferrocement, and ACI Committee 549, Ferrocement, is now working on the final draft of a guide so that designers will have something to follow.

Muhammad Faruk Zein (Al-Muhandis Nizar Kurdi Consulting Engineers, Riyadh, Saudi Arabia): Since most of our speakers share the opinion that the z-value is not as significant as suggested in the past, are any changes under way to reflect the importance of Theory No. 2, or at least to explain that the z-factor should not be relied upon blindly? If so, will they be reflected in new ACI publications?

Darwin: As chairman of Committee 224, Cracking, I am tempted to answer that myself; however, since I am the moderator, I will turn it over to Dave Manning, another member of the committee who is also the chairman of Committee 222, Corrosion of Metals in Concrete.

Manning: I am not the best one to answer that because I have carefully tried to stay away from the structural implications of what I had to say earlier. I will make a quick response to Bob Price's comments and then turn the question back to someone more qualified than I to answer it. By way of clarification, Bob, all the comments

*Heger, Frank J., and McGrath, Timothy, "Crack Width Control in Design of Reinforced Concrete Pipe and Box Sections," ACI JOURNAL, *Proceedings* V. 81, No. 2, Mar.-Apr. 1984, pp. 149-157.

that I have made to date have been with respect to conventional reinforcement and not prestressing strand. I am interested in knowing whether the failures that you report were the result of a very localized reduction in the cross sectional area at the crack, which resulted in failure of the strand. I suggest to you that the use of prestressing strand in a corrosive environment with 0.75 in. (19 mm) of cover of a moderate quality mortar, is simply asking for trouble. That is clearly not adequate protection for prestressing strand in a corrosive environment.

Price: I agree that only 0.75 in. (19 mm) of cover is rather risky. We have observed that the wire is corroded neither under the delamination nor at the crack, but at a location remote from the crack. The explanation for this is that corrosion is not occurring in the area of the delamination which is exposed to oxygen, but at the interface or just back from the interface. In one particular instance, we investigated a pipe in which there was a platter-sized delamination. Plotting the broken wires revealed that the break took place right around the delaminated area — it outlined it very neatly. In an article in NACE Materials Magazine, Jack Grable dealt with this phenomenon of steel corrosion occurring in partially embedded reinforced concrete structures. He stated that very high rates of corrosion will occur at the interface and not in the exposed area.

Darwin: Let me comment on ACI's position on crack width. One of the reasons we are having this debate is because a number of the committees, especially Committee 224, Cracking, are concerned about the way the *z*-factor has been used in some instances to reduce cover in order to reduce crack width under conditions of high potential corrosion. Committee 224 has said specifically in several documents* that the crack width, or crack width as interpreted through *z*, is not going to protect reinforcement against corrosion.

Roger L. LaCroix (SGE, Rungis, France): I was very impressed by Andrew Beeby's presentation. I wonder whether sufficient attention has been paid to the quality of the cover concrete, because some research has shown that the water-cement ratio of the cover concrete is not the same as the internal concrete. This might be the reason that certain old structures behave very well with less than ½ in. (13 mm) cover while others behave very poorly with a greater amount of cover.

Beeby: I agree absolutely that cover and the quality of the concrete in this cover are of extreme importance. Work currently being carried out in our laboratories and laboratories elsewhere suggests that curing is probably at least as important as any of the other factors which have been discussed today. It must be remembered that, while poor curing may have a relatively small effect on the quality of the total mass of the concrete, it can have a very large influence on the relatively small thickness of concrete protecting the bars. This indicates that, from the durability point of view, curing may be very much more important than was previously thought. Also, the thinner the cover, the more important is the curing. Experience has shown that with high quality, dense, well-cured concrete, it is possible to protect the steel with very low cover thicknesses. In some cases, such as with ferrocement, cover less than ½ in. (13 mm) has proved perfectly satisfactory.

Theodore R. Crom (The Crom Corporation, Gainesville, Florida): As a builder of prestressed shotcrete tanks, I want to personally confirm what the speakers have said about the importance of quality and curing, and to berate slightly those who say that to reduce corrosion we have to go to more and more cover, without looking at quality. The normal response of those who have had poor experience with dry-mix shotcrete is thicker is better. To change this response, ACI Committee 506, Shotcreting, and E 902-7, a subcommittee of the Certification Committee, are working toward nozzleman certification. Instead of having to say "go from 1 to 2 in. thick" or "2 to 3 in. thick" which is wasting the client's money, we want to develop techniques for improving the quality of shotcrete. Shotcrete has a very low water-cement ratio, high impermeability, high cement content, and good density — it has all the things you need, if you can get the quality of application. These committees are developing methods of determining and acquiring this quality. I urge those who work with shotcrete to look into what these committees are doing; certification of nozzlemen means field testing of the candidates and an ultimate improvement in the quality of shotcrete.

From the audience: Can a member of one of the committees elaborate on the effect of cyclic loading on corrosion or crack width?

Darwin: We know that both long-term loading and cyclic loading will increase crack width. Typically, the crack widths we talk about are short-term crack widths, and over a period of two years, we can expect that the crack width will double.

Craig Williams (Masters Builders, Cleveland, Ohio): I am a little confused on whether cracks are good or bad, but I think we all agree that they are bad. Maybe we can tolerate them. Those who have an interest in crack-free structures might consider the use of an expansive cement concrete, which can help eliminate cracks.

Regarding concrete quality, we have just completed some testing at the Construction Technology Laboratories using the Strafull method. This is a method for chloride penetration testing. The specimens are submerged in a chloride solution, potentials are read at intervals, and everything ultimately fails. Plain concrete fails after a certain length of time, and you can evaluate the effect of admixtures or other ingredients on that basis. We do not have the published document yet, but we did run one series with 0.30 lb (136 g) of chloride intentionally added to the mix, and that curve plotted right along with the plain concrete — indicating that quantity of chloride had no effect. The length of service of another specimen, which had 0.06 percent chloride, was extended by several weeks. The logical theory is not that the chloride is particularly good, but that the concrete was improved by the addition of the admixture, and the permeability was reduced. I believe that the whole concept of improving the concrete is highly critical.

"It was found that reinforced concrete submerged permanently suffered no damage due to the chlorides in the concrete, even in tropical waters. The problem occurs when we get out of the water, out of the ground, and into an atmosphere, where both moisture and oxygen are available. Then and only then do we have corrosion difficulties."

—Eivind Hognestad

*ACI Committee 224, "Causes, Evaluation, and Repair of Concrete Structures," (ACI 224.1R-84), American Concrete Institute, Detroit, 1984, 20 pp. and ACI Committee 224, "Control of Cracking in Concrete Structures," (ACI 224R-80) (Revised 1984), American Concrete Institute, Detroit, 1980, 42 pp.

Regarding curing, ASTM is reviewing a new method of evaluating curing compounds or the effectiveness of any curing method which is based on relative absorption. I think this is something that will emphasize the need to reduce permeability and keep it to a minimum.

I would like Mr. Beeby to comment on the distance between the anode and cathode. He pointed out that the cathode is internal in the concrete, and it seems to me that accessibility of the oxygen to the cathode would come not only from the surface but also from the crack. Perhaps he can clarify this point.

hancement of C_3A content greatly reduced the chloride content within the cement, from which we conclude that in that particular application, the use of cements high in C_3A were beneficial in lowering the chloride content. But we hesitate to make any general or sweeping statements about the use of high C_3A cements, because we know that in many applications, the cements will be used in situations where they could potentially experience attack, not only from chloride, but also from sulphate. That seems to be a real problem, because we know that when you go to high C_3A contents in the cement, you lose protection against sulphate. It is a question of trading one thing off for another. Although I must point out that I have never done an experiment to find out whether there could be an optimum level in the C_3A content of cement.

". . . the use of prestressing strand in a corrosive environment with 0.75 in. (19 mm) of cover of a moderate quality mortar, is simply asking for trouble. That is clearly not adequate protection for prestressing strand in a corrosive environment."

—David G. Manning

Beeby: The distance between the anode and cathode in a corrosion cell can vary over a very wide range. In typical cases where corrosion has initiated at cracks, the anode may extend for two or three bar diameters around the cracks while the rest of the bar remains cathodic. There are, however, cases where very different conditions can occur. For example, in a bridge deck with top and bottom steel, it is possible for the top mat, which has been contaminated by chlorides, to act as an anode while the bottom mat acts as the cathode. There have been suggestions that in the type of structures used for oil extraction in the North Sea, it could be possible for the whole underwater area to act as a cathode driving a relatively small anodic area in the splash zone. In the case mentioned first, where cathode and anode are separated by only a few bar diameters, then some oxygen penetrating down the cracks may be able to permeate along the bar to cathodic areas, but I think this is likely to be a second order effect.

Fredrik P. Glasser (University of Aberdeen, Aberdeen, Scotland): There are one or two cement chemists here, and I think they are reluctant to speak up. Two questions have been raised about the relationship between cement quality and durability. The first concerns the role of tricalcium aluminate in cement. We have done some rather simple experiments in which we took cement of different C_3A content and mixed it with water containing chloride. After the cement had set for 60 or 90 days we removed the pore fluid in the cement. We found that the en-

Secondly, it seems to me that engineers are missing an important field of activity when it comes to the protection afforded by slag cement. Our experiments, as well as those performed in other places, particularly Scandinavia, show that the internal oxidation potentials of slag cements can be lowered by several hundred millivolts relative to the same cement without a slag addition. This is an important element in short-term electrochemical corrosion in concrete cements which contain substantial amounts of slag. My question is, is this a permanent effect or a temporary effect? And if temporary, for how long?

Robert Douglas Hooton (Ontario Hydro, Toronto, Ontario, Canada): Some work was done by Feldman at the National Research Council in Ottawa on magnesium chloride exposure and degradation of the paste in the concrete. He concluded that a Type II sulphate-resistant cement was less durable in the presence of chloride, but he attributed that to the fact that Type V cements contain more C_3S and C_2S and thus liberate more calcium hydroxide in the hardened product. That calcium hydroxide can be leached out and thus open up the structure to attack by the chlorides. You get not only a surface degradation, but a degradation throughout. The effect of adding fly ash, slag, or silica fume was to use the calcium hydroxide, decrease the permeability, increase the resistivity, and improve the chloride resistance.

From the audience: A very important subject related to cracks is the repair of bridges. Our company, in cooperation with the New York State Department of Transportation, inspected and made recommendations for the repair of bridges along the Hutchison River Parkway. In many instances, we found longitudinal cracks and exposed, corroded steel. I agree with the person who felt that cracks do not necessarily mean that the whole structure is damaged. We took material samples from the structures to measure the strength of the existing concrete and steel, and from the results of the tests, we decided to save bridges that were constructed 50 or 60 years ago. These bridges are in good condition and structurally sound, even with the cracks. The bridges will be repaired by epoxy injection of the existing cracks, replacement of the corroded steel, and application of a shotcrete overlay over the entire surface. Perhaps at another convention we could present material that will contribute to the knowledge of the repair of existing structures.

Darwin: The 1985 fall convention in Chicago, Illinois will emphasize repair and rehabilitation of structures.

From the audience: I have a question for Mr. Beeby. I do not understand why the anode and not the cathode is the location that is coincidental with the crack. It seems to me that the cathode, which invites the oxygen and water, would coincide with the crack and not the other way around.

"It must be remembered that, while poor curing may have a relatively small effect on the quality of the total mass of the concrete, it can have a very large influence on the relatively small thickness of concrete protecting the bars."

— Andrew W. Beeby

Beeby: The effect of carbonation or chlorides penetrating down a crack to the reinforcement is to depress the potential of those areas of bar relative to the potential elsewhere; they therefore become anodes. I have heard of circumstances where the system may work the other way around, such as in artificially fully carbonated concrete, but this is a special and rare condition.

Hope: Let me comment on the role of C_3A and calcium chloride admixture.

We have found that the chemical reaction takes place very quickly, usually within 24 hours of mixing. Therefore, it does not take 60 or 90 days; it is a very quick reaction. Some of the chlorides get bound with the C_3A phase in the first 24 hours. As far as slag cement is concerned, we believe that the corrosion rate is reduced over the long term, primarily because of the reduced permeability which leads to an increased resistivity. For example, if you use a silica fume cement, you can have high corrosion potential, but virtually no corrosion because of the very high resistivity of the overlay material and of the silica fume cement material.

Robert W. Gaul (Adhesive Engineering Company, San Carlos, California): I have a question in reference to Andrew Beeby's comments regarding unnecessary repair of cracks. If you are not sure that conditions exist which will prevent the potentially serious damage that might occur because of these cracks, then is it not inexpensive insurance for the owner to fix the crack when it is first discovered? The crack can be fixed effectively by epoxy injection for a fraction of the cost of the structure, or a fraction of the cost of the anticipated possible repair.

Many of us are hesitant to suggest crack repair to the owner, once the structure has been designed by a competent engineer and built by a competent contractor. I suggest we warn the owner ahead of time that cracks may occur, and inform him of what the cost of repair might be if he wishes to repair the cracks. Then he is aware of the problem and his alternatives from the start.

Robert West (Wiss, Janney, Elstner Associates, Inc., Princeton Junction, New Jersey): I am concerned that I may have inferred something from Andrew Beeby's remarks that is incorrect. If I understand him correctly, his argument is that to use a greater number of smaller bars to provide the required steel area increases the number of cracks and thus increases the risk of corrosion. What about the relationship of cover depth to bar diameter and the risk of damage that may occur from using larger bars which can develop a greater tensile stress in the area of the cover?

Beeby: You are quite right; I did not mention that there may be an improvement in performance due to an increase in the ratio of cover to bar diameter. However, this may be offset by the smaller bars being more sensitive to corrosion damage than the larger ones. The CEB Model Code,* for example, requires more stringent measures for the protection of small diameter bars than it does for large ones. The exact balance of factors here is therefore not clear. Nevertheless, in my particular example, the use of smaller bars has increased the number of corrosion sites to such an extent that it is unlikely that any decrease in risk due to higher cover/bar diameter ratio could be sufficient to compensate.

"If you are not sure that conditions exist which will prevent the potentially serious damage that might occur because of the cracks, then is it not inexpensive insurance for the owner to fix the crack when it is first disovered? The crack can be fixed effectively by epoxy injection for a fraction of the cost of the structure, or a fraction of the cost of the anticipated possible repair."

—Robert W. Gaul

Darwin: I would like to take this opportunity to ask a question that I have had sitting on my desk for about 18 months. The current provisions in ACI 318 are not realistic in terms of connecting exposure conditions with crack width. Two different values of z are used in the Building Code: one for interior exposure, and one for exterior exposure. If we limit our consideration to structures other than liquid retaining structures, what should the limitation on crack width accomplish? The limitation is applied with Grade 60 steel and not with Grade 40 steel. We use the limitation with Grade 60 steel to help distribute the reinforcement and have nice-looking cracks. What should be our goal? What is a tolerable crack?

Hognestad: That gives me a chance to come back to the z-values established 15 years ago. This is a 1983 Code and it is about unchanged from 1971. It begins with the distribution of flexural reinforcement. Section 10.6 provides for the distribution of flexural reinforcement to control flexural cracking in beams and one-way slabs. Further down it says these provisions may not be sufficient for structures subject to very aggressive exposure or designed to be watertight. Essentially, the concept we are talking about here first appeared in Sweden in 1938-39 when the late Professor Granholm set out to build girders across a laundry in the basement of the main Stockholm hospital. The spans were a little over 60 ft (18.3 m). He had manufactured for him deformed reinforcing bars with a yield point of about 86,000 psi (593 MPa) and he designed by what we now call strength design, so that his service stresses were well over 30,000 psi (207 MPa), predominantly due to dead weight. His beams were narrow and deep, with a relatively large number of reinforcing bars in them. Up to this date, they are performing very well in spite of the high humidity of the laundry.

A similar design was used in the roof of the PCA structural and fire laboratories, completed 26 years ago. Those beams are also relatively narrow; 4 ft (1.2 m) deep in the middle and 3 ft (0.9 m) deep at the ends, with a span of 60 ft (18.3 m). They are reinforced with eight #9 bars with a yield strength of 84,000 psi (580 MPa). As far as strength is concerned, rather than eight #9 bars, we could have used two #18 bars. However, design by the z-equation led to the reasonable number of medium-sized bars, using a minimum cover of 1.5 in. (38 mm).

Perhaps the road toward design criteria for the future could be to differentiate between various exposures, types of member, and types of service. But before you know it, you would have created a very lengthy table. There is a limit to how many specific situations a code can cover. And there is always the question of good engineering practice at the end of it all.

Darwin: I want to mention that what Committee 350 has done, Committee 224 suggested doing several years ago. Committee 224 offered Committee 318 three different recommendations to allow the use of additional cover to improve corrosion resistance. One of the recommendations was to limit the value of the cover that is used for the calculation of z to 2 in. (51 mm). The suggestions were made as preliminary recommendations, and the response from Committee 318 was to make them final recommendations before they would look at them. They have remained as preliminary recommendations for about three years. I think it is time for Committee 224 to finalize its recommendations!

**CEB-FIP Model Code for Concrete Structures*, 3rd Edition, Comité Euro-International du Béton/Fédération Internationale de la Précontrainte, Paris, 1978, 348 pp.