CORROSION OF REINFORCING STEEL EMBEDDED IN STRUCTURAL CONCRETE

by

James T. Houston
Ergin Atimtay
and
Phil M. Ferguson

Research Report No. 112-1F

Research Project Number 3-5-68-112
Crack Width-Corrosion Study

Conducted for

The Texas Highway Department

In Cooperation with the
U. S. Department of Transportation
Federal Highway Administration

by

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

March 1972

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ACKNOWLEDGMENTS

During the course of this four year study many have participated in the various phases of the research program. Among those making contributions were numerous students, including Mr. Murilo A. Miranda, Mr. R. J. Chen, Mr. Sher Ali Mirza, and Mr. C. N. Krishnaswamy. Technical staff assistance was provided by Mr. Gorham Hinckley, Mr. George Moden, Mr. Jerry Crane, and Mr. Harman Ramsey.

The authors wish to express their thanks to those members of the Texas Highway Department and Federal Highway Administration who made helpful suggestions during the research study. Contact representatives of those agencies were, respectively, Mr. H. D. Butler and Mr. Glenn McVey.

PREFACE

This report describes the research techniques and results obtained during a four year project (1967-71) entitled "Crack Width-Corrosion Study" (Study No. 3-5-68-112). This report is a complete presentation of the research as no other formal report preceded the termination of the research program.

Support for this study was provided by the Texas Highway Department and the Federal Highway Administration, U. S. Department of Transportation.

James T. Houston
Ergin Atimtay
Phil M. Ferguson

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ABSTRACT

Results of a literature review provide a broad overview of more than twenty parameters affecting the process of corrosion of reinforcing steel in concrete. In addition, a four year experimental program utilizing a relatively severe salt spray exposure to promote corrosion in numerous loaded beams and unloaded slab specimens is reported. Experimental parameters used in the study included quality of concrete (cement type, water-cement ratio, aggregate type, relative permeability), placement of steel and concrete (concrete cover, bar size, bar spacing, casting position), and exposure and loading conditions (concrete cracking, level of steel stress, prestressing, rates of corrosion). The study of these parameters provides information useful in the selection of the concrete quality and structural detail required to produce corrosion resistant structures.

KEY WORDS: corrosion, chloride, rusting, reinforcing steel, stress, cover, concrete, quality, durability, water-cement ratio, permeability, aggregates, cracking, structural.

SUMMARY

This four year study effectively points out that the ability of structural concrete to protect embedded reinforcing steel against salt water chloride corrosion is primarily a matter of selecting the proper materials and concrete mix design, and using appropriate structural design especially with regard to clear cover and bar size.

The ability of concrete to inhibit corrosion of reinforcing steel is essentially determined by its watertightness or permeability. The relative permeability of concrete was generally found to be reduced as the water-cement ratios of the various concretes were reduced, and this in turn produced more corrosion resistant structural members. The watertightness of the concrete was also shown to be significantly dependent upon the type of coarse aggregates used with the more permeable concretes being produced by use of selected crushed limestone and lightweight aggregate. The development of a simple water penetration test provides an effective method for future evaluation of highway concretes produced of varying mix design and aggregate properties.

A significant finding of this study showed that although corrosion protection was directly related to depth of cover over reinforcement, a more meaningful parameter in this regard was the ratio of the clear cover to bar diameter (C/D). Greater corrosion protection was provided by beams and slabs having high values of C/D with good protection resulting for C/D values greater than about 3.0. This finding is of importance since normal design practice calls for specific minimum concrete cover regardless of bar size whereas this study shows that a given cover may provide adequate protection for a small bar but may be totally inadequate for a relatively large bar. In addition to C/D effects, it was determined that the initial rate of corrosion of reinforcement was very dependent upon concrete cover. For example, the decrease of a 2 in. cover down to 1 in. resulted in a four fold increase in the initial rate of corrosion.

Although flexural cracking of concrete was found to promote corrosion of the reinforcement at the crack location, the severity of the long term corrosion damage to the bars was primarily dependent on the depth of concrete

cover. Large cracks usually found in conjunction with large cover promoted early corrosion at the crack locations, but further development of the corrosion as well as longitudinal cracking of the cover over the bars were inhibited for the larger covers. Narrower cracks generally associated with shallow covers had little influence on the overall corrosion. In that instance the bars were rather uniformly rusted with extensive longitudinal splitting of the concrete cover over the bars.

Only a slight increase in corrosion resulted as a consequence of stressing the beam reinforcement through flexural loading. These observations indicate that the existence of stresses in the reinforcing bars (up to 36 ksi) and the flexural cracks produced by these stresses were of less importance as corrosion accelerating hazards than had been expected.

IMPLEMENTATION STATEMENT

The implementation of the results of any corrosion study must be preceded by a rational comparison of projected field exposure conditions with those used during the research study. As a result, reasonable accuracy can be achieved in projecting the service life of structures subject to corrosion exposure. It should be recalled that the findings of this research program are based upon the evaluation of structural elements daily exposed to one thorough wetting with a three percent salt spray solution. In addition, the implementation suggested primarily applies to reinforcing bars since only a few prestress specimens were studied.

With regard to concrete mix design and structural design, the parameters of water-cement ratio and the ratio of clear cover to bar diameter were found to be significant in affecting corrosion of reinforcing bars. For corrosion protection of structural elements relatable to the conditions of this research program the following tentative design specifications are suggested:

(1) water-cement ratio (maximum) 5.5 gal/sk

(2) concrete cover* (minimum) 3.0 in. or at least 3D**

In general it was determined that only slightly more corrosion occurred on stressed reinforcing bars in comparison to unstressed bars. In conjunction with the effects of stressing, flexural cracking of the concrete cover was not the significant factor in the long term corrosion durability, as had been expected. More important with respect to cracking was the amount of cover provided. Although corrosion of reinforcing bars was usually initiated at flexural cracks, the larger concrete covers served to inhibit the continued development of corrosion along the bar. Crack widening by increases in steel stress from 20 ksi up to 35 ksi produced only a very slight, if any, increase in corrosion.

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* Concrete cover specifications should include a provision for rating the permeability of the concrete.

** A clear cover to bar diameter ratio (C/D) = 3.0 is probably adequate for D $<$ 1.0 in. and bars $<$ No. 8.

In addition to structural design, the production of watertight concrete mixtures is equally important in providing corrosion resistant structures. Furthermore, to be truly meaningful, cover specifications should include some means of rating the relative permeability of the concrete. A simple penetration test developed during this study provides such a measure, and limited results indicated that concretes of greater water penetration permit greater corrosion of reinforcement. This test also showed that for a given water-cement ratio, use of different types of coarse aggregate produced concretes of greatly different penetration. Such data pertaining to a number of aggregates being considered for use in corrosion resistant concrete structures should be valuable in properly selecting the most suitable material. For this reason verification of the indicated corrosion-permeability relationship is merited; more specific data would help in developing a suitable specification.

The authors feel that useful information concerning other corrosion related parameters resulted from this study. However, due to the limitations of the data, no implementation can be recommended. Such parameters include cement type, bar spacing, position of casting and prestressing. An abbreviated summary concerning these and other parameters is given in Sections 1.3 and 1.4 of this report. In each case additional research study may provide definitive recommendations and specifications for corrosion resistant structures.

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CHAPTER I

INTRODUCTION

1.1 Nature of the Problem

During the past five to ten years, considerable concern has been expressed for the numerous and relatively widespread instances of deterioration of concrete highway structures, especially bridge decks. Concrete failures of this type usually take the form of surface scaling, spalling, and cracking. Attempts to determine the causes of early deterioration of concrete highway structures have, in summary, covered the full range of concrete technology including materials selection, mix design, placement, finishing, curing, reinforcement cover and placement, stresses, cracking, and effects of various environmental conditions to name a few. It is perhaps certain that all of the above-mentioned parameters affect to varying degrees the physical integrity of the structure. And when it is considered that replacement of a deteriorated structure may cost as much as fifteen times that of the original construction,[1*] the necessity for properly controlling these parameters is obvious.

In many field studies of concretes showing surface deterioration, corrosion of reinforcing steel has also been noted. Although corrosion is not normally thought to produce scaling, it has been observed to produce spalling and cracking of structural elements. Corrosion is particularly serious for structural members, since it is normally progressive and ultimately leads to the necessity of replacement or to complete failure. Failures take the form of loss of stress-bearing concrete due to spall-off or to loss of stress-carrying steel due to excessive depth of rust penetration. In certain types of steels such as prestress cable, corrosion can produce sudden, brittle type failures without the buildup of excessive coatings of rust which normally serves as a warning of impending danger.

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*Supercript numbers refer to references listed in Section 4.9 of the Appendix.

In Texas the incidence of corrosion induced defects on highway bridge structures has been relatively low, even though many of the surveyed structures were built of non-air-entrained concretes.[2] Localized areas of higher incidence of corrosion damage do exist, however. Examples are the Gulf Coast region and isolated areas where deicing chemicals have been used.

With respect to other modes of concrete deterioration, Texas has been less fortunate. Scaling, spalling, and cracking in highway structures have caused serious concern. This concern receives impetus from the fact that surface deterioration of concrete results in a condition favorable to the promotion of corrosion of the underlying steel.

It is therefore apparent that corrosion of reinforcing steel is an integral part of the whole of those factors controlling the durability of a concrete structure. In fact, the prevention of corrosion of steel in concrete may prove to be the most effective way of producing truly durable concretes in all respects. This follows from the fact that high quality concretes and adequate cover over reinforcement are necessary for corrosion prevention.

1.2 Objectives and Scope of the Study

A review of the literature reveals that the subject of corrosion is extremely complex and has received much attention resulting in a great many published studies. When the field is narrowed to corrosion of reinforcement in concrete, the number of publications is still relatively large. However, when one restricts his interest to corrosion studies of specimens specifically designed to simulate real elements of highway structures, the number of available publications becomes quite small and none specifically involving Texas aggregates and mix designs are available.

It was, therefore, the objective of this study to provide pertinent corrosion data from realistic specimens at varied steel stress levels using concretes typical in Texas highway structures. Since no prior data existed for these circumstances, it was thought desirable to conduct a somewhat exploratory research program in which a relatively large number of the important corrosion variables were included. These variables were: type of reinforcing steel, cement type, water-cement ratio, aggregate type, concrete water tightness, bar size and spacing, cover, casting position, concrete cracking, steel stress, and prestressing. The authors realize that each of these parameters would normally merit detailed study in individual investigations. However, it was felt that the more general approach was the more efficient technique to isolate the most critical parameters in this particular situation.

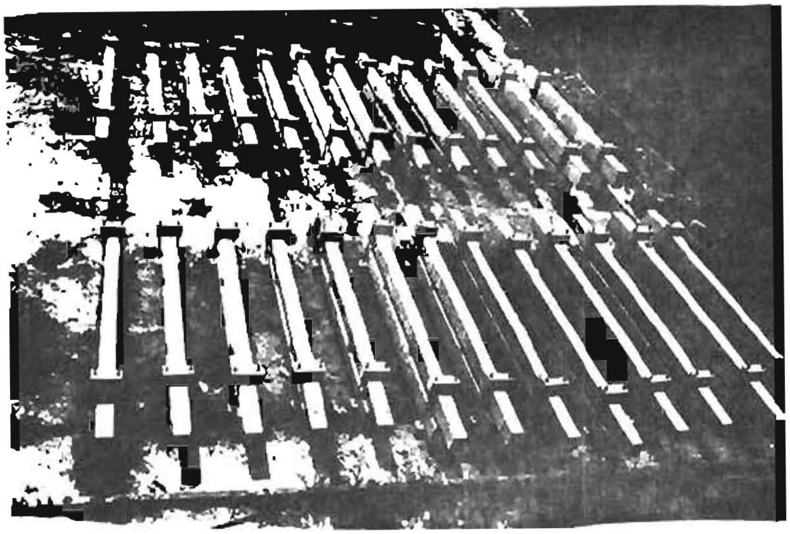
The program undertaken spanned four years and involved 82 structural elements. They included 34 normal weight and 6 lightweight loaded beams. Also included were 36 normal weight and 6 lightweight slab specimens. These specimens were subjected to daily spraying with a 3 percent salt solution for various periods of time, ranging up to 34 months. Two views of the testing area where the specimens were sprayed are given in Fig. 1.2.1. The development of transverse and longitudinal cracking in the specimens was recorded up to the time the specimens were removed for sawing and corrosion analysis of the reinforcing bars. At this time selected specimens were either sawed or cored for the determination of relative permeability of the concretes. For detailed information on the values of the parameters studied, the reader is referred to those specific topics in Chapter III of this report.

1.3 Research Findings

It is emphasized that the research findings reported here are based upon relatively severe corrosion exposure conditions. Application of these findings for other exposure conditions should be preceded by a rational comparison of the relative severity of the exposure.

Concrete Quality Effects

1. For the severe exposure conditions of this study, the corrosion of unstressed bars in slab specimens was significantly reduced for concretes of low water-cement ratio, that is, to a value of 5.5 gal./sk. Further reducductions below a water-cement ratio of 5.5 gal./sk. were not significantly effective in further reducing corrosion.



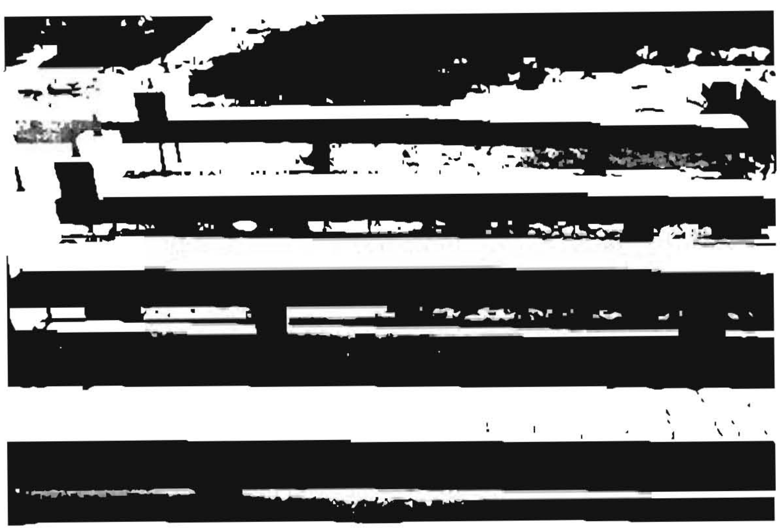


Fig. 1.2.1. Loaded flexural specimens in test area

2. Water-cement ratios of 5.5 gal./sk. gave full corrosion protection for 24 months for #6 bars in uncracked concrete of 2 in. cover. For otherwise similar conditions, a water-cement ratio of 7.0 gal./sk. allowed corrosion of up to 75 percent of reinforcing bar area.

3. The lightweight aggregate concretes of this research program provided corrosion protection to reinforcement comparable to that of the crushed limestone concretes. A more specific conclusion cannot be given due to data limitations.

4. Slightly greater corrosion of reinforcing bars resulted for concretes made with Type V cement in comparison to Type III cements.

5. For a limited number of comparable specimens, concrete made with Type III cement was slightly less water tight than that of Type V cement.

6. The relative permeabilities of the various concretes of this study generally decrease with decreasing water-cement ratio, although at the lowest water-cement ratios used, an unexplained reverse trend was noted in two instances.

7. At a water-cement ratio of 5.5 gal./sk. the most watertight concrete was produced from siliceous aggregates, while the least watertight concrete was produced from the lightweight coarse aggregates. Crushed limestone concretes were of intermediate permeability. At 6.25 gal./sk. the comparisons were similar except that the crushed limestone and lightweight aggregate concretes exhibited approximately equivalent water tightness.

8. For a given type of aggregate and exposure conditions, generally greater corrosion resulted as the relative permeability of the concrete increased as indicated by the water penetration test developed during this study.

Effects of Placement of Steel and Concrete

9. For a given bar or prestress cable size, the corrosion resulting from the salt spray exposure was inversely related to the amount of concrete cover.

10. The research data indicated that for a given concrete cover, larger reinforcing bars (#11) were less resistant to corrosion attack than were the smallest bars (#6).

11. A new parameter combining the influence of clear cover and bar size was found to be significant in providing a more conclusive design-related corrosion relationship. The parameter C/D (clear cover divided by bar diameter) was found to be inversely related to reinforcement corrosion.

12. High values of C/D generally resulted in low levels of longitudinal splitting for given amounts of rusting, while low values of C/D tended to produce large amounts of splitting at low levels of corrosion.

13. For a water-cement ratio of 6.25 gal./sk. a C/D ratio of 1.0 was found to be inadequate, while values of 2.5 or more provided good corrosion protection for exposures of 24 months.

14. In a comparison of four flexural specimens of similar dimensions and steel percentages, the use of four #8 bars in place of two #11 bars resulted in a significant reduction in bar corrosion and longitudinal splitting of the concrete cover.

15. Reinforcing bars initially cast in the top zone of flexural beams experienced greater corrosion than those initially cast in the bottom zone of the beams.

Exposure and Loading Effects

16. In many cases corrosion of reinforcement was initiated at large flexural cracks (in excess of 50×10^{-4} in.). However, the limitation of crack widths to values below, say, 40×10^{-4} in. did not insure corrosion protection, especially for beams with shallow cover (1 in.).

17. A relatively uniform corrosion of reinforcement resulted when shallow covers were accompanied by closely spaced, narrow flexural cracks. (For example, 1 in. cover, crack widths less than 40×10^{-4} in. with cracks spaced at 4 to 5 in.)

18. Early corrosion was initiated at the relatively large, widely spaced flexural cracks associated with beams having larger covers. (For example, 2 in. cover, crack widths greater than about 70×10^{-4} in. with cracks spaced at 8 to 12 in.)

19. Even though early corrosion develops at flexural cracks, large covers were effective in minimizing continued corrosion by inhibiting the development of longitudinal splitting.

20. In general, only slightly more corrosion occurred on the stressed bars as compared to the unstressed bars of flexural beams. The increased corrosion for the stressed bars was apparently caused by the presence of flexural cracks in the concrete of the stressed portions of the beams.

21. For exposure periods of 24 months there was little difference between the corrosion resulting on flexural reinforcement stressed to 20, 30, and 35 ksi. It is apparent that for the reinforcing bars used in this study, neither stress corrosion nor the normal increase in crack width with increasing stress was a significant factor in the corrosion process, for the stress levels used here.

22. Flexural beams with shallow cover and low C/D ratios generally exhibited severe, corrosion-induced, longitudinal splitting of the concrete cover over the reinforcement. Similar specimens with high C/D ratios were more resistant to longitudinal splitting than those of the previous case.

23. There was no significant difference between the corrosion of stressed 3/8 in. prestress cable and that of unstressed #6 bars in slab specimens. It is projected that, if bar and cable diameters are equal, somewhat greater corrosion would likely result for the prestressed cable.

24. The corrosion damage of the prestressed slab specimens was almost always initiated at the cable cutoff points, even though the exposed cable stubs were coated with heavy grease.

25. For unstressed #8 bars in flexural beam specimens, the initial rate of corrosion was inversely related to the concrete cover. The corrosion rate at 1 in. cover was more than four times that for 2 in. cover.

1.4 Recommendations

1. The authors feel that permeability is perhaps the most significant indicator of the ability of a concrete to inhibit corrosion. In conjunction with 7 above, it is recommended that selected aggregates used in various regions of the state be evaluated with regard to concrete permeability and

corrosion of reinforcement. As a result, those types of materials producing low permeability concretes can be identified and used to advantage in construction applications particularly susceptible to corrosion damage.

2. It is felt that aggregate type, maximum size, and grading significantly affect the ability of the concrete to provide corrosion protection. In view of the limited data available in this program for study of these parameters, additional research of these specific parameters is required.

3. The effect of use of different types of cement upon corrosion of reinforcement is not well-defined in previous studies. Since the effect may be significant in reducing corrosion damage, further research is needed.

Placement of Steel and Concrete

4. A very significant corrosion parameter was found to involve the interaction of bar size, cover, and bar spacing. Since the preliminary findings of this study have direct design implication, it is desirable that the interaction be more clearly defined by additional study. For example, a more complete range of C/D parameters should be investigated. Also, only very limited data were obtained with regard to bar spacing effects, the results of which merit additional study.

5. The effectiveness of various concrete consolidation techniques should be evaluated with regard to producing low permeability, corrosion resistant, concrete covers. This parameter could be evaluated in the study recommended in 1 above.

Exposure and Loading

6. In future corrosion research with regard to the effects of concrete cracking, data should be obtained at somewhat shorter exposure periods than typically used here.

7. It is suggested that future studies of the effect of steel stress upon corrosion should employ a higher maximum stress level than used here. Also, other grades of steel should be studied.

8. The regions near the cutoff points of prestress cables were most susceptible to corrosion damage and effective means of protecting the cable stubs should be determined.

9. Attempts should be made to determine the relationship between experimental exposure used here and field exposure conditions for various structural applications within the state.

10. An effective means of repair of concrete structures in which corrosion has developed should be determined.

11. Methods of treating existing reinforced concrete structures to enhance corrosion resistance should be determined.

Specifications

The authors recognize that the uncertainties with regard to the relationship between the research methods and actual field exposure are significant. However, it is also felt that at least limited recommended specifications are merited as a result of this research program.

For protection of concrete structures subjected to relatively severe salt spray corrosion exposure, the following tentative specifications are suggested*:

1.	Water-Cement Ratio	(maximum)	5.5 gal./sk.
2.	Concrete Cover**	(minimum)	3.0 in. or at least 3D***

*Complete specifications are provided by various agencies. Only those parameters for which appropriate research data were obtained are mentioned here.

**Concrete cover specifications should include a provision for rating the permeability of the concrete.

***This research study indicates that a clear cover to bar diameter ratio (C/D) = 3.0 is probably adequate for D < 1.0 in. and bars < #8.

CHAPTER II

LITERATURE SURVEY OF PARAMETERS INFLUENCING CORROSION

2.1 General

This chapter is concerned primarily with summarizing a review of the literature related to the various parameters affecting the corrosion of reinforcing bars in concrete. It was found that a considerable number of references are available for this purpose and that most important parameters are well-researched.

In presenting the review summary, the order of commentary will follow the general outline of parameters as given by Fig. 2.1.1. The authors feel that the parameters shown in Fig. 2.1.1 represent a fairly complete listing for corrosion of steel in concrete. Those readers not familiar with the electrochemical nature of this type of corrosion are referred to Sec. 4.1 of the Appendix, which summarizes the basic nature of the corrosion reactions involved.

2.2 Reinforcing Steel

2.2.1 Metallurgical Factors. It is perhaps common knowledge that many elements alloyed with steel produce increased corrosion resistance. The major corrosion inhibiting elements include copper, nickel, and chromium, most of which are present in negligible proportions in reinforcing steels. Specific combinations of these and other elements have been found to improve corrosion resistance of steels, but from a practical standpoint have had little application in commercial reinforcing steels.[3]

Localized metallurgical differences in the atomic structure of the steel result in differential energy fields within the steel which promote the formation of the anodic and cathodic regions necessary for electrochemical corrosion. These regions are, in effect, different materials in contact with one another. Energy fields are usually associated with

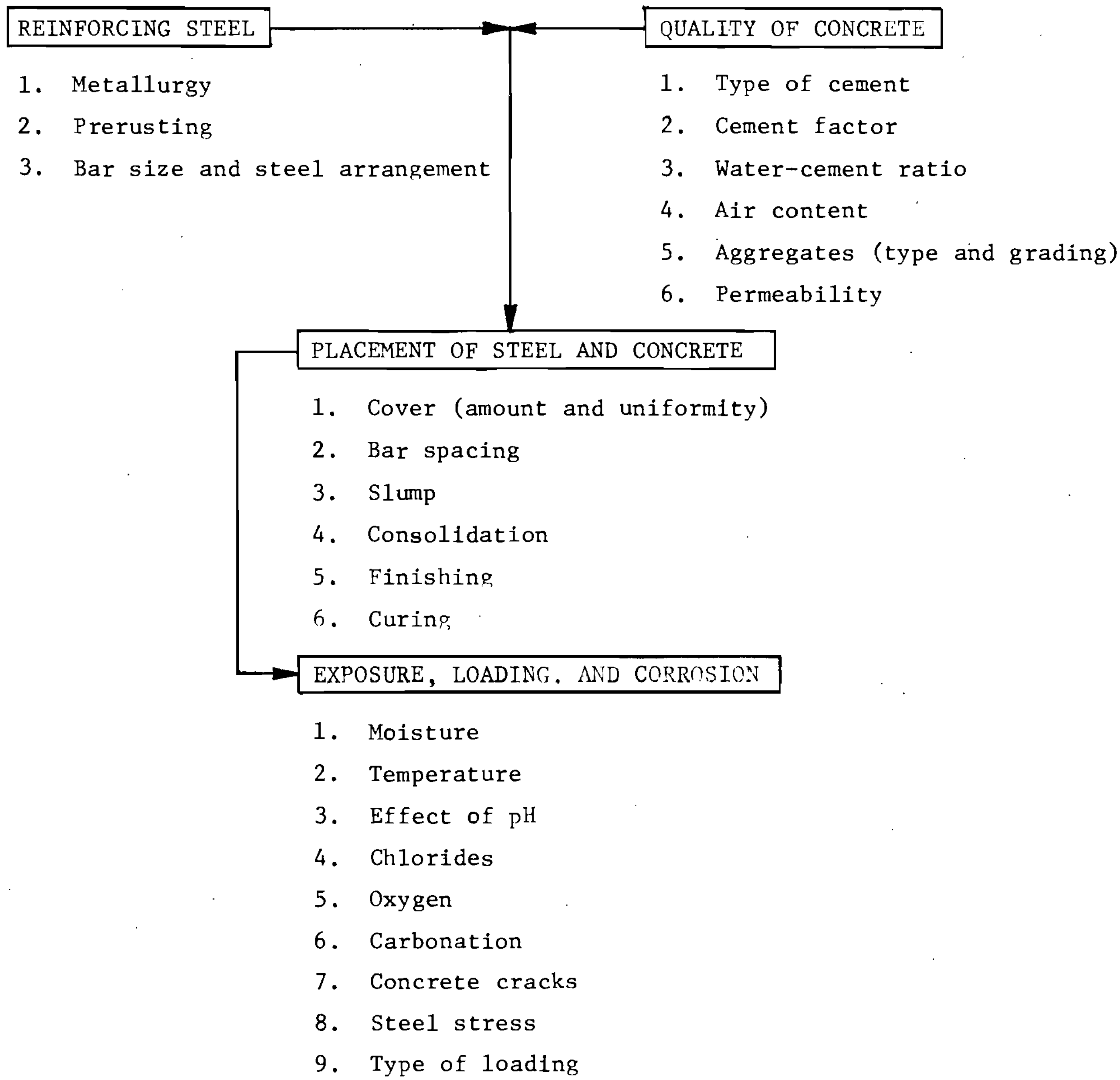


Fig. 2.1.1. Summary of the various parameters found to influence the corrosion of steel in concrete.

dislocations, mismatched grain boundaries, inclusions, impurities, metallurgical phase boundaries, etc. For example, it has been determined that the ferrite phase of steel is readily attacked, while cementite is resistant to corrosion.[3] Where both phases exist adjacent to one another, the cementite would become the cathode and the ferrite would be the anode if a corrosion cell developed. It should be recognized that differential energy field sources for corrosion cells are present in all commercial steels and therefore a means of inhibiting corrosion must be found other than attempting to homogenize the metals, which is impractical and of questionable effectiveness.[3] For this reason it is fortunate that the effect of these various energy fields upon the corrosion of reinforcing steel is minimal as long as the pH of the surrounding concrete remains relatively high (in the range of 10 to 13).[4]

In addition to the corrosion cell sources associated with the basic atomic structure of the metal, the surface of the reinforcing bar offers additional opportunities for cell formation. Such factors as surface roughness, scratches, cuts, and particularly mill scale are frequently responsible for the initiation of corrosion.[3] Unfortunately, mill scale formed during the hot rolling of the steel does not result in a continuous scale coating. As a result, surface areas coated with mill scale are cathodic to the uncoated adjacent areas.[3,5,6,7]

In certain applications metallic coatings offer corrosion protection to surfaces of steel. However, such cathodic coatings as nickel and copper are not effective in reinforcing steels since they are relatively expensive and would likely be damaged during construction, therefore creating serious localized corrosion conditions.[5] Cadmium and zinc are anodic to steel and can be used as sacrificial coatings.[4] Galvanized coatings on reinforcing bars are perhaps practical, but to be effective, the coating must be of adequate thickness.

2.2.2 Prerusting of Reinforcement. The condition of the reinforcing bars prior to embedment has been the object of considerable discussion. According to ACI 318-63, Building Code Requirements,[8] it is required that loose, "flaky" rust must be removed from reinforcing steel prior to use and that normal rough handling generally removes injurious rust. On the other hand, ACI 318-71 Building Code Requirements[9] are less restrictive with respect to prerusted reinforcement in that use of prerusted bars is allowed so long as ASTM requirements on deformation height, dimensions and brushed bar weight are met. Prestressing steel is required to be free of "excessive" rust.

Furthermore, it has been reported that normal rust actually increases bond. Researchers have found that for 14-day-old concrete the use of prerusted welded wire fabric resulted in less bond slip in comparison to clean wire.[10] However, the long-term effects of the use of prerusted bars is not well-defined. This is especially critical for exposed structures. In fact, it has been suggested that prior rusting of prestress tendons can cause serious corrosion after encasement in grout.[11] The same concern could be expressed for prerusted reinforcing bars in exposed structural elements.

2.2.3 Bar Size and Steel Arrangement. Relatively few corrosion studies were found to have included variables related to bar size and steel arrangement. In one study it was determined that a welded grid of reinforcement was no more susceptible to corrosion than individually insulated bars.[12] Others report an observed relationship between bar spacing and corrosion induced cracking on freeway bridgedecks.[1,13] In that study reinforcing bars spaced one foot apart generally developed trenchlike spalls, while those spaced six inches apart tended to develop weakened planes.

2.3 Quality of Concrete

2.3.1 Type of Cement. Reviewed studies of the effect of cement type upon corrosion are somewhat inconclusive. For example, in one study it was concluded that cement type has little effect, if any, on calcium chloride induced corrosion of reinforcing steel.[14] On the other hand, Type I cement, high in tricalcium aluminate, has been reported to provide considerably more protection against chloride induced corrosion than does Type V cement.[15] Also, blast furnace slag cement has been suspect of promoting corrosion of steel in concrete.[7,11]

Since the physical durability of concrete is important in maintaining a corrosion resistant structure, other factors associated with cement type may be of importance. For example, researchers report that the durability of concrete in seawater is dependent upon the alumina content of the cement. Generally, it is reported that high alumina cements produce more durable concretes when exposed to seawater.[16,17] High alumina cements may also be of advantage where carbonic acid induced corrosion is significant.[18]

However, sulfides of high alumina cement have been reported to cause embrittlement of prestressing tendons under certain circumstances.[11,19] Also, long time durability tests of portland cement concretes in seawater have indicated that cements lowest in tricalcium aluminate are relatively more durable.[17] Finally, the fineness of cement has not been shown to be a significant parameter in long time durability studies.[20]

2.3.2 Cement Factor. Almost all sources reviewed agree that corrosion protection is increased by increases in cement factor.[7,12,21,22,18] Recommendations for cement factors for adequate protection against corrosion varied from 4.1 to 7.0 sacks per cu. yd., although the most generally recommended minimum value was 6.0 sacks per cu. yd.

In a few instances the influence of cement factor on corrosion and corrosion related parameters was reported to be minor. Air permeability of concrete was found to be only slightly affected by cement factor.[23] When seawater was used in making mortar for corrosion specimens, cement factor was found to have little effect on the pH of the mortar.[24] It is generally agreed that corrosion is inhibited as long as the pH of concrete surrounding reinforcing steel is in excess of about 10. This same study reports that there was no regular relationship between cement factor and the percentage of rusting of reinforcing steel.[24] However, this study did generally report best corrosion resistance for the richer mixes.

2.3.3 Water-Cement Ratio. There is some disagreement as to whether or not water-cement ratio directly influences corrosion. There are those who report that water-cement ratio strongly affects corrosion through its influence on permeability of concrete.[20,25] Other studies indicate that depth of carbonation of concrete is affected by water-cement ratio. A six year test program showed that increasing the water-cement ratio from 0.6 to 0.95 by weight increased the maximum depth of carbonation from 5 to 25 mm.[23] Carbonation has the effect of lowering the pH of concrete, thereby decreasing its ability to protect steel reinforcement from corrosion. Of those agreeing that water-cement ratio is directly related to corrosion, various recommended values are suggested. These values of recommended water-cement ratios range from 4.5 gals per sack for severe exposure to 8.0 gals per sack for relatively safe exposure.[11,23]

A few researchers have reported that water-cement ratio does not itself control the rate of corrosion of reinforcement.[22,24] In these studies it was determined that consistency of the concrete was more predominant in controlling corrosion than was water-cement ratio. Very dry mixes of low water-cement ratio as well as very fluid mixes of high water-cement ratio exhibited greater corrosion in comparison to mixes of intermediate consistency and water-cement ratio.[22]

2.3.4 Air Content. Few of the references surveyed used air content of concrete as a primary variable in studies of the nature of corrosion of embedded reinforcement. However, it is well-known that air entrainment is very valuable in producing durable concrete for exposed structures such as bridge decks.[26,20,1,27,28] In general, entrained air is found to reduce bleeding, decrease permeability, improve workability, and improve freeze-thaw durability.[28] Bridge deck durability studies have shown that delamination and scaling are less prevalent when air entrained concretes are used.[28,27,20,26] It can therefore be concluded that by maintaining the physical integrity of the concrete covering the reinforcement, entrained air does provide some degree of increased protection against corrosion. It must be emphasized that the uniformity of the entrained air content is particularly important where the possibility of corrosion is high because adjacent areas of non-uniformly air entrained concrete may promote the formation of corrosion cells.[7]

2.3.5 Aggregates (type and grading). The type of aggregate used in making concrete is generally agreed to be of significant importance in producing durable structures.[20] With respect to corrosion it has been pointed out that nonreactive aggregates are an essential ingredient in high quality concrete.[23,11] Other properties of aggregates are also reported to influence the corrosion resistance of concrete. For example, porous aggregates tend to produce permeable concretes which promote corrosion attack of reinforcing steel.[11] However, several studies report conflicting results for the effect of lightweight aggregates upon corrosion related concrete durability.[7,26,29]

It is apparent that the use of certain types of lightweight aggregates results in poorer corrosion resistance in comparison to silicious gravel aggregates.[7,29] The same relationship would likely result for a comparison of very permeable limestone aggregate and gravel aggregate.

The grading of concrete aggregate is as important as aggregate type in producing the dense, impermeable concrete necessary for corrosion protection. Several studies surveyed indicate that the use of coarser graded aggregates provides better corrosion protection in concrete.[7,30,23,22] For example, in one study a fine aggregate with a fineness modulus of about 2.2 and having about 15 percent finer than a No. 100 sieve produced a porous, low density concrete in comparison with an aggregate having a fineness modulus of about 3.7 with about 2 percent passing the No. 100 sieve.[23] It should be emphasized at this point that a small amount of fine aggregate passing the No. 100 sieve is helpful in producing concretes of low permeability.[28,31]

2.3.6 Permeability. Probably the single most important parameter influencing the corrosion of reinforcement in concrete is the permeability of the concrete cover. Many references reviewed agree with the importance of permeability, but unfortunately, very few corrosion research programs included concrete permeability as a primary research variable.[7,5,32,20,12,11,23,6,18,33] Field studies have been conducted in which concrete of high permeability has been identified as responsible for the development of corrosion.[32,17] Concrete permeability is of great importance because corrosion is primarily controlled by the penetration of various liquids and gases into the cover to the level of the reinforcement. Also of importance is the uniformity of the permeability of the cover, since corrosion cells are frequently initiated in areas of nonhomogeneous concrete.[7,5]

There are a number of concrete mix variables which affect permeability and subsequently corrosion. These variables include water-cement ratio, cement factor, cement-aggregate ratio, and grading, maximum size, and porosity of aggregates. Also included are factors associated with fresh concrete such as mixing action, consistency, placement, curing, and age.[7,20,30,25,11,17,23,6] A graphic example of the influence of one of these parameters is the effect of water-cement ratio as exhibited by research which showed that the permeability of a concrete with water-cement ratio of 6.5 gal per sack is 2.9 times greater than that for concrete of 5.5 gal per sack.[20]

Useful recommendations for producing concrete of low permeability have been indicated in several references.[7,32,20,30,25,11,17,23,34,28] A summary of recommendations is useful and is therefore provided below.

1. Use concretes of low water-cement ratio. Specific values required for corrosion protection vary depending upon the amount of cover provided and quality of aggregates.
2. Use aggregates of low permeability.
3. Always employ the use of air entrainment.
4. Select the largest coarse aggregate size possible.
5. Use well-graded fine aggregate which is not deficient in minus No. 100 mesh particles. The higher values of fineness modulus are preferred.
6. The use of higher cement factors is of slight benefit in minimizing permeability and may produce greater shrinkage cracking.
7. The consistency of the fresh concrete should be relatively plastic.
8. Employ early continuously wet curing methods for as long as practical. Recommended minimum curing times vary with exposure conditions.
9. The use of a permeability specification in conjunction with cover requirements is desirable if a practical technique can be provided for measuring permeability.

2.4 Placement of Steel and Concrete

2.4.1 Cover (amount and uniformity). For specific recommended values of minimum cover, the reader is referred to Sec. 4.8 of the Appendix which contains a summary of corrosion related specifications used by several large agencies.

It is generally recognized that the amount of cover over reinforcement controls to a large extent the protection of the reinforcement from corrosion. In one field survey it was determined that 40 percent of the corrosion failures of reinforced concrete were due to insufficient cover.[23] However, assigning fault to insufficient cover alone is perhaps improper since researchers have stated that it is meaningless to specify cover without knowledge of the permeability of the concrete.[11] It has been reported that the effectiveness of any given amount of cover is dependent upon permeability and cracking of concrete.[35,7] Other test results indicate that increases in cover beyond about 2 inches do not significantly increase corrosion protection.[32] This statement may be partly supported by research which showed that bar level hydrostatic pressures required to crack concrete covers of 1-1/2 in. up to 4-1/4 in. were not greatly different.[1] Another study concluded that the rate of corrosion decreases for increasing cover up to 7/16 in., after which little change is noticed for greater amounts of cover.[21]

From the literature surveyed a number of different values of cover were found to give satisfactory protection. Those values ranged from 0.5 in. with dense concrete to 2.5 in. in aggressive environments.[32,36,11,23,37,18,34] Amounts of cover which were found to be ineffective as a result of field and laboratory testing ranged from 0.75 in. to 6 in., for which daily salt water spraying produced corrosion at the end of 2-1/2 years.

The effectiveness of cover has also been related to the slump of the fresh concrete. Equivalent protection is said to be provided by 1.5 in. cover of 3 in. slump concrete and 2.0 in. cover of 8 in. slump concrete.[26]

Deterioration of bridge decks is often exhibited by spalling of the concrete. Corrosion of the reinforcement frequently precedes or accompanies spalling. In several studies it was noted that corrosion induced spalling could be attributed to several factors, one of which was insufficient cover.[32,34,13]

Although there is disagreement concerning the influence of cracking upon corrosion, it seems likely that a certain degree of cracking does promote corrosion. In this regard, the formation of longitudinal cracks in the cover above reinforcing bars is perhaps dangerous from a corrosion standpoint. These types of cracks are reported to form early in the life of the concrete and are due to tensile stresses created as the fresh concrete subsides immediately adjacent to the reinforcing bars.[34] It seems very likely that this type of cracking would be decreased as the cover is increased.

The importance of uniformity of cover over reinforcement has been exhibited by research which concludes that nonuniform cover promotes the formation of various types of corrosion cells.[7] For example, differential oxygen cells and differential moisture cells would likely develop in areas of nonuniform cover. In those cases the oxygen rich and moisture rich zones would become the anodes of the corrosion cell. This type of corrosion promotion is important in highway structures where field studies have revealed considerable variation in cover on certain bridge decks.[1,34] In a few cases field measurements have indicated actual cover values as low as 1/8 in. Some researchers go as far to say that uniformity of cover is more important than the density of the concrete cover.[24]

2.4.2 <u>Bar Spacing</u>. The effect of placement and spacing of reinforcing bars upon corrosion has received relatively little attention. Field observation of deteriorated bridge decks has revealed that when the uppermost bars were spaced one foot apart, corrosion produced trench-like spalls parallel to the bar. When the bars were spaced at six inches apart, corrosion produced horizontal cracking between bars creating weakened planes.[1,13] In another corrosion study it was concluded that use of welded mats of reinforcement for preplacement spacing resulted in no greater susceptibility to corrosion than use of individually tied and insulated bars.[12] One other corrosion study recommends that bars should not be positioned by the use of bricks, wood or other porous, nonalkaline materials.[7]

Studies which are indirectly related to corrosion offer recommendations on the placement of reinforcement. In one such study it was suggested that placement of the smaller, more widely spaced, longitudinal temperature reinforcement on top of the larger transverse bars results in a decrease in bridge deck cracking over the bars.[20] Too closely spaced reinforcement tends to segregate the fresh concrete during placement resulting in porous covers.[23]

2.4.3 Slump. Researchers have found that consistency of fresh concrete has a significant effect upon corrosion of reinforcement, and it should be recognized that slump is not totally dependent on water-cement ratio and cement factor.[22] It is generally reported that plastic mixes provide the best protection against corrosion.[7,22,21] In one study it was concluded that as little as 1/4 in. cover of plastic concrete provided adequate protection.[21] Very wet mixes have been found to result in uniform rusting of reinforcement while dry mixes have promoted pitting corrosion.[7,22,17]

Of those references making recommendations for slump, a maximum of 3 in. slump was most often given as providing the best corrosion protection.[20,26,34]

2.4.4 Consolidation. Proper consolidation of the fresh concrete is important since it has a significant effect on the quality of the concrete adjacent to and above the reinforcement. If the concrete is inadequately vibrated, voids may result adjacent to the bars and thus promote the formation of corrosion cells.[7,18] Also, inadequate vibration may result in the delayed subsidence of concrete at the sides of the reinforcing bars, which promotes the formation of longitudinal cracking over the bars.[1,34] The effect of cracking on corrosion is discussed in Sec. 2.5.

From field observations it is reported that over-vibration of concrete decks is common.[1] Also, undervibration is usually worse than overvibration from a corrosion standpoint.[20] A modified method of consolidation which has been reported to have been successfully used in the field is revibration of a retarded concrete mix. This method of consolidation simply provides a more dense and therefore a more water tight and a lower porosity concrete.

Suggested recommendations to improve the consolidation of the concrete frequently follow those outlined by the Portland Cement Association.[28] In addition, the use of small diameter internal vibrators or external vibrators has been suggested as preferred.[34] It is also recommended that movement of reinforcement after initial set of concrete be minimized.[34]

2.4.5 Finishing. The methods used to finish concrete influences corrosion of reinforcement because it affects the permeability and cracking of the cover over the bars. For these aspects of quality of concrete, a number of bad practices and preferred practices have been cited. Overfinishing has been found to be a common fault[1,34] and results in accelerated scaling but does not seem to affect air content.[20] The addition of water or grout during the finishing operation is also detrimental[1,34] and frequently promotes cracking of the cover.[38] Late finishing is undesirable in that it tends to promote scaling.[1]

Of the various methods used to finish concrete, machine screeds and planes are reported to give the best results.[20,1] One literature survey reports that a single strike-off provides better scaling resistance than a second and final finish.[20] However, from one field study it was concluded that two floating operations was the superior method of finishing in that it sealed the surface after the initial drying shrinkage and consolidation had occurred.[38] Accurate alignment of screeds is also recommended in order that uniform, prescribed cover may be maintained.[34]

2.4.6 Curing. Proper curing is the last of the important steps necessary to produce a high quality, corrosion resistant concrete cover over reinforcement. Poor curing practices have been found to cause early deterioration of bridge decks.[20]

A number of curing practices have been found to provide high quality concretes. The early application of the cure is very important in reducing cracking and should be applied as soon as surface damage to fresh concrete can be avoided.[20,34,38] Continuous moist sprays have been reported as superior to curing compound sprays.[20,34] Extended curing periods are found to reduce the permeability of concrete[17] and minimum curing periods of five days have been recommended.[34] Longer curing times may be required depending on the environmental conditions.[28]

2.5 Exposure, Loading, and Corrosion

2.5.1 Moisture. In the basic corrosion mechanism moisture is a required element forming the electrolyte. Corrosion typical to that of reinforcement in concrete is inhibited in the absence of moisture, because ion transfer cannot occur.[7] Even in the presence of water containing salts, corrosion can be inhibited if the dissolved oxygen content of the water is very low.[24] Oxygen, which is an important element of the corrosion process, acts to depolarize the cathode. Normal levels of dissolved oxygen in water typically do not penetrate submerged concrete in sufficient quantities to keep corrosion reactions active.

It is generally concluded that partial immersion or alternate wetting and drying are the most favorable conditions to promote corrosion. Differential moisture concentrations around the steel may promote corrosion macrocells.[7] Water trapped around prestress cables during grouting causes severe corrosion hazards.[11] High humidity in the atmosphere combined with industrial gases is very aggressive to steel.[11]

Freeze and thaw action of entrapped water in the concrete cover may lead to the deterioration of the protective properties due to scaling of the cover. This action ultimately promotes the penetration of corrosive elements into the concrete to the level of the reinforcement.[20]

2.5.2 Temperature. The level of temperature not only affects the rate at which a corrosion reaction proceeds, but it also affects the quality of concrete as mixed.[39,28,20] Typically, most chemical reactions such as corrosion increase exponentially with increasing temperature.[39] However, in the case of corrosion reactions in the presence of water containing dissolved oxygen, the rate of corrosion may decrease temporarily as the temperature increases due to the removal of oxygen from the water.[39] In most other cases, increases in temperature brings about a rapid increase in the rate of corrosion.

The adverse affects of elevated temperature on the quality of fresh concrete may decrease the corrosion inhibiting properties of the concrete cover over the steel. High temperature at the time of mixing usually requires an increase in water content to produce a given workability, lowers concrete strength, makes control of air entrainment difficult, and promotes greater drying shrinkage and cracking.[28] All of these effects are detrimental to the ability of the concrete to provide corrosion protection to reinforcement.

2.5.3 Effect of pH. The pH is a measure indicating the acidity (0 –7.0) or alkalinity (7.0 –14.0) of a medium. Specifically, the pH value is the negative value of the logarithm of the hydrogen ion concentration.[40]

When a medium is acidic, that is pH less than 7.0, hydrogen ion activity is increased. Greater production of hydrogen gas at the cathode of a corrosion cell leads to increased corrosion rates.[3] On the other hand, if the medium is alkaline, that is pH greater than 7.0, a decrease in hydrogen ion activity is seen which promotes the formation of a strong, tight rust coating.[3] As a result of the latter case the vulnerability of steel to the continued attack of corrosive forces is decreased. When the pH value reaches 12.0 or more, total corrosion inhibition occurs.[7]

Experiments have shown that fresh concrete has an average pH of about 12.8 if made with ordinary tap water, and about 11.8 if made with seawater.[24] This makes fresh concrete a highly alkaline substance in which corrosion reaction involving steel does not normally occur. However, penetration of salts and carbon dioxide into concrete results in a gradual decrease in the pH value, so that corrosion reactions can become active. The rate of corrosion gradually increases as the pH value drops to 4.0. Below a pH of about 4.0 the corrosion reactions increase exponentially.[40]

2.5.4 Chlorides. Chloride ions do not directly take part in the corrosion reactions, but their role in the corrosion process is very significant. They help set up the chemical environment under which corrosion can take place. After corrosion begins they also determine to a great extent the rate and intensity of the reactions.

Two common sources of chlorides in concretes of bridge decks are sea spray and deicer application. Chloride ions find access to the steel through cracks and/or begin penetrating through the concrete cover. Mechanically, salts can produce expansive forces within the pores of concrete by the action of crystal growth. This is a common mechanism by which the durability of concrete is attacked.[1] Bridge deck observations have shown that scaling is promoted by deicing chemicals.[27]

Chlorides also act in a way to reduce the pH value of the concrete even though their action is not as strong as chromates, dichromates, or sulfates.[40,7,5,32] It has been shown that $CaCl_2$, which destroys the corrosion inhibiting properties of concrete and is frequently used in precast members to accelerate hydration, was the cause of many prestressed bridge failures in France.[23]

When chlorides reach the steel, the corrosion enhancing action is effective in two ways. First, it destroys the protective oxide film, "mill scale" around the bar surface, thus making the steel fully vulnerable to corrosive reaction. Second, it accelerates the corrosion reaction by increasing the conductivity of the electrolyte.[7,5,3,13]

An increasing chloride concentration of the electrolyte does not produce a corresponding increase in corrosion rate for all concentration levels. When chloride concentration increases beyond a certain value, oxygen solubility of the electrolyte is decreased. This produces a corresponding decrease in the corrosion rate. However, this phenomenon does not usually occur within practical concentration of chlorides in concretes.[7]

Salts deposited against steel in a nonhomogeneous distribution initiate corrosion cells similar to those caused by differential moisture and aeration.[7,5]

Chlorides also interfere with the formation of protective coatings in corrosion-active areas.[3] Corrosion products may act in a way to seal off moisture and oxygen and inhibit continued corrosion activity. But aggressive chloride ions destroy this coating and allow corrosion to remain active.

2.5.5 Oxygen. Oxygen is possibly the most important element in the corrosion process. It is originally responsible for the formation of the corrosion resistant mill scale which forms on hot rolled reinforcing bars. Also, it is responsible for the promotion of the corrosion process in several respects.

Differential concentrations of oxygen in concrete form the anodic (lack of oxygen) and cathodic (high oxygen concentration) areas to initiate the reactions.[5,6] Once the reactions have begun, oxygen is further active as a depolarizer.[5] The inhibiting effect of the hydrogen coating at the cathode is broken down by the action of oxygen and thus the corrosion process is continued. Also, oxygen actively takes part in the formation of corrosion products, the final form of which is the familar rust.

In the presence of abundant oxygen the protective nature of alkaline environment is somewhat reduced. It has been reported that higher alkalinity in concrete is required for adequate protection in the presence of abundant oxygen.[3]

2.5.6 Carbonation. Carbonation is a process by which carbon dioxide in the atmosphere penetrates the concrete and reacts with the calcium hydroxide produced by the hydration of cements. The result is a lower pH in the concrete because the mild carbonic acid formed in this process reduces the alkalinity of the concrete by reacting with calcium hydroxide and forming calcium carbonate.[40,7,3,41]

$$[\{CO_2 + H_2O\} \rightarrow H_2CO_3] + Ca(OH)_2 \rightarrow CaCO_3 + 2H_2O$$

The carbonic acid formed also directly attacks steel in a moist environment.[18]

$$Fe + CO_2 + H_2O \rightarrow FeCO_3 + H_2$$

If the carbon dioxide penetrates the concrete cover unevenly, differences in concrete pH result which promote the formation of corrosion cells.[3] Some researchers have also shown that carbonation tends to increase shrinkage and cracking in concrete. The result of this action is increased corrosion.[7]

The danger of carbonization is greatest in a dry concrete where pores are open for carbon dioxide penetration. Fortunately, in dry concrete the electrolytic action does not exist and corrosion does not normally occur.[23] Research results indicate that, for ordinary concrete, carbonation is not likely to proceed beyond a few mm depth.[24] Reported depths of carbonation are found to be 10-25 mm in a dry concrete of W/C ratio of 0.90, and 1-5 mm when the W/C ratio is 0.60.[23] As indicated by these data, high quality concretes are relatively dense and more resistant to carbonation than are lower quality mixes.

Suggested preventive measures against carbonization include the use of more dense concretes, increased cement factors, and use of aluminous cements.[18]

2.5.7 Concrete Cracks. Concrete cracking has almost always been associated with corrosion of reinforcing steel. Many building codes have required the control of crack widths for protection against corrosion. The effect of cracking can be viewed for flexural and longitudinal types of cracking separately.

Flexural cracks normally occur in reinforced concrete structures. Regardless of the width of such cracks, steel stresses at the crack are increased thus favoring the formation of corrosion cells due to the presence of differential strain energy fields.[7] Also, it is found that water, oxygen, and salts can reach the steel faster and more abundantly through wider cracks.[13] As a result, the pH of the concrete in the vicinity of a crack decreases and corrosion accelerates when the pH nears a value of 7.0. It has also been reported that significant corrosion takes place at cracks which are wider than 0.0004 in. and almost no corrosion occurs if the crack width is less, depending on the cover.[32,11,23] On the other hand, some experiments have also indicated just the contrary; wider cracks have shown less corrosion than narrower cracks, again depending on the depth and porosity of the cover.[6]

The controversy over the effect of flexural crack widths is not yet settled. There is experimental evidence for both opposing views, much of which is not differentiated as to whether the corrosion evidenced is due to atmospheric action or to chlorides. This subject will be further treated in Chapter III.

All researchers seem to agree on the importance of longitudinal cracking in the corrosion process.[23] The formation of longitudinal cracks can occur in the absence of flexural cracks. Also, flexural cracking is not necessary in order for chlorides and oxygen to penetrate to the level of the reinforcing steel.[35] The top portions of a bridge deck are normally more porous than other parts, due to the general effects of consolidation, finishing, bleeding, and drying shrinkage. Corrosive agents can easily penetrate a poor quality cover and initiate corrosion. Longitudinal cracking can be thought of as an indicator of active corrosion, because such crack formation is in many instances a direct consequence of the buildup of corrosive products. Such products exert expansive forces on the concrete cover and split it open along the reinforcing bar. Once the crack forms, ample moisture, oxygen, and salt can reach the steel to cause continued corrosion at an accelerating rate.

2.5.8 Steel Stress. The major effect of tensile steel stress in concrete reinforcement is that of producing cracks in the concrete. Therefore, the level of steel stress at which concrete cracks are produced is of importance with regard to corrosion in view of the crack-corrosion correlation indicated in Sec. 2.5.7. In the immediate vicinity of such cracks the steel stress is nonuniform along the bar length. This has the effect of setting up corrosion cells which tend to produce pitting.[3]

Beyond the effect of steel stress in causing concrete cracking, other corrosion related stress parameters have not been so thoroughly researched. For example, the quality of the concrete cover as measured by freeze-thaw durability has been reported to be affected by level of steel and concrete stress in some studies and not affected in others.[1] In those cases where the stress effect was exhibited, the zones of concrete under compressive stress were generally more durable. With respect to prestress cables, level of stress may be of significance in the presence of hydrogen embrittlement which produces stress corrosion cracking failures.[5] However, stress corrosion cracking is not normally associated with intermediate grade reinforcing steel bars.[42]

2.5.9 Type of Loading. Very little data exist concerning effects of type of loading upon corrosion in concrete. The action of static loading generally follows the corrosive mechanisms as discussed in the previous sections of this chapter. In addition, cyclic stressing of structures may lead to additional corrosion deterioration by the mechanical breakdown of protective oxide films or coatings produced by the corrosion process. This action enables the corrosion reactions to continue relatively uninhibited.

Corrosion fatigue failures may also occur by cyclic stressing in the presence of chlorides.[3] However, no significant difference has been observed between the corrosion deteriorations of specimens under low cyclic fatigue load and unstressed specimens.[1]

CHAPTER III

DISCUSSION OF CORROSION RESEARCH RESULTS

3.1 General

The research results presented in this chapter were obtained over a four-year period, during which various loaded and unloaded concrete specimens were exposed to daily spraying with salt water. The evaluation of the corrosion which occurred during this period has been expressed in terms of percentage of bar surface area rusted, R(%). Although the authors recognize that other parameters for evaluating corrosion are preferred, the nature of the corrosion of steel embedded in concrete was not easily adaptable to such parameters as weight loss or rust penetration rates. For details of the corrosion evaluation procedure, as well as other techniques used in this research program, the reader is referred to the Appendix, Chapter 4. The Appendix also contains a listing of the research data for those who wish to develop relationships other than those discussed in this chapter.

3.2 Reinforcing Steel

Since reinforcing bars from at least three different manufacturers were used during this four-year program, a possibility of composition variation existed. It was therefore decided to make a relative comparison of the elemental composition of the bars by determining the amounts of chromium, nickel, copper, and phosphorous present in selected samples. These elements are reported to influence the corrosion of steels.[4]

For this analysis a Phillips AMR-3 electron microprobe analyzer was used at a 30,000 volt potential. Four samples, including No. 6, 8, and 11 bars, were analyzed.

Results of the microprobe study indicated that no chromium, nickel, or copper was present in any sample and that a maximum of 1 percent by weight of phosphorous was present in isolated regions of two specimens. Two other samples showed nonhomogeneous areas of up to 0.5 percent phosphorous. It was, therefore, concluded that the critical composition of the various steels was reasonably uniform and that no serious related effects on the research data should be expected.

No other metallurgical studies were made as to metal phases present or grain structure, although these, of course, do influence corrosion behavior.[4,39]

3.3 Corrosion and the Quality of Concrete

3.3.1 Cement Type. During the last year of the research program, Type V portland cement was used in casting three reinforced slab specimens. Prior to this time Type III cement was used in all specimens. The water-cement ratios chosen were 4.75, 6.25, and 7.0 gal/sack. Companion specimens were cast of identical materials, except that Type III cement was used. After six months of exposure, the slabs were cored and then broken open for corrosion analysis of the reinforcement. Although the number of specimens was small, useful data were obtained with respect to permeability of the concrete and corrosion of the reinforcement. The importance of permeability in corrosion has been previously indicated.

Values for relative permeabilities of the concretes were determined by measuring the average depth of uniaxial penetration of water into a previously dried specimen. The permeability test was developed during this study and is more completely discussed in Sec. 3.3.4. and 4.7.

The water penetration test was performed on three 6-in. diameter cores from each of the three slabs made with Type V cement. The average depth of penetration for each slab is plotted as a function of water-cement ratio in Fig. 3.3.1. Also plotted in Fig. 3.3.1 are three data points obtained from tests of a total of twenty cores from eleven slabs made of Type III cement. The 28-day moist cure compressive strength of the concrete used in the slabs is given in parentheses at the side of each data point shown in Fig. 3.3.1

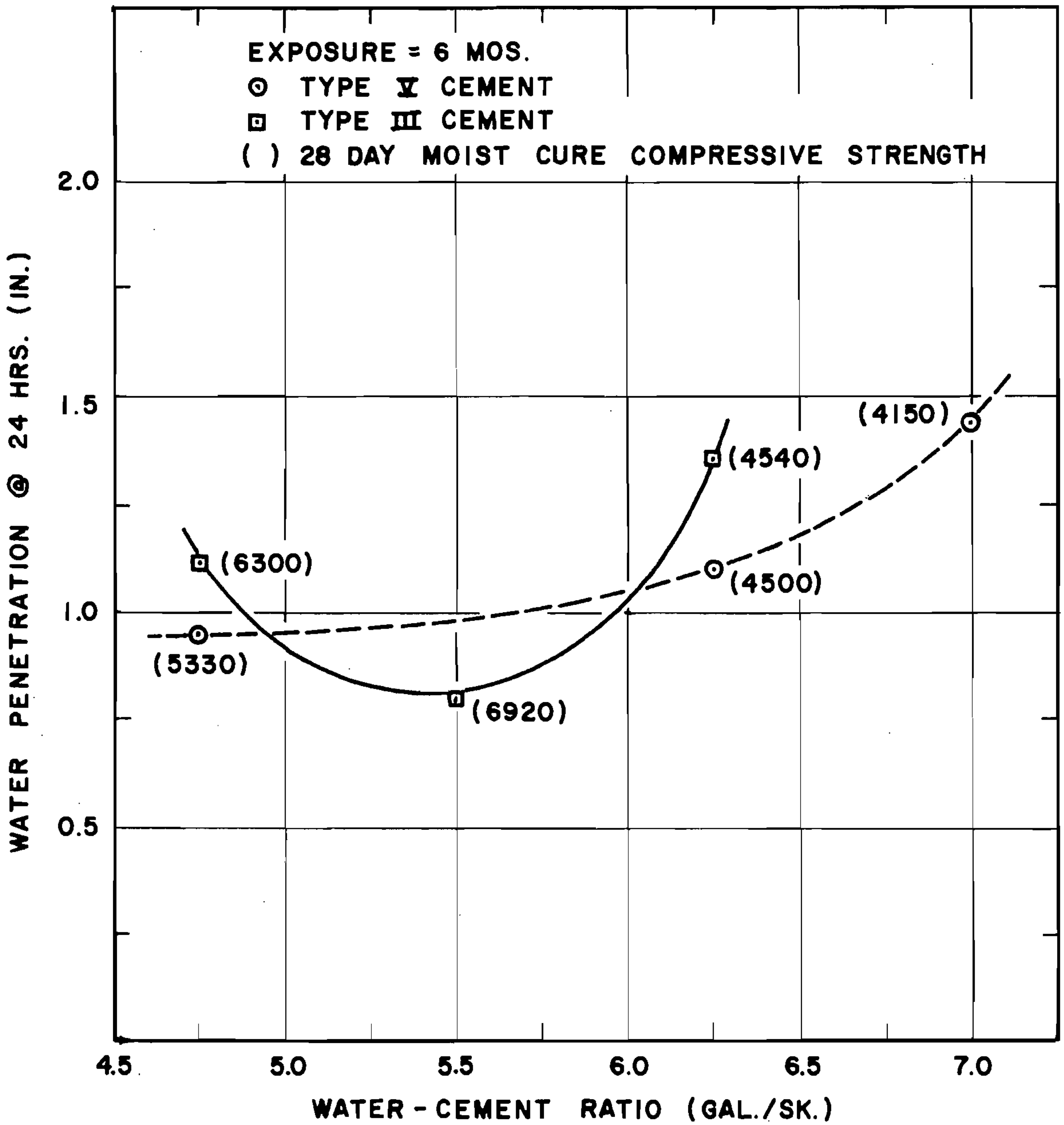


Fig. 3.3.1 Effect of cement type on water permeability of 1/2-in. crushed limestone concrete from slab specimens.

For the Type V specimens the penetration and strength values vary as might be expected for increasing water-cement ratios. However, the penetration and strength value at 5.5 gal/sk for the Type III specimen is perhaps in an unexpected position. One would normally expect a relationship similar to that shown for the Type V specimen. However, it should be noted that the strength of the Type III concrete is relatively high at this water-cement ratio. In addition, the slump and air content was relatively low for this particular mix at 2 in. and 3 percent.

In evaluating the data of Fig. 3.3.1 in light of the previous explanation, it appears that the permeability of the concretes made with Type III cement is slightly greater than that of similar mixes made with Type V cement. Also, the 28-day concrete strengths were highest for the Type III mixes at water-cement ratios comparable to the Type V mixes.

Following six months of salt spray exposure, specimens made of Type III and V were analyzed for corrosion of reinforcement following the procedure outlined in Sec. 4.6.2 of the Appendix. Comparable mixes were available at water-cement ratios of 4.75 and 6.25 gal/sk. The results of the analysis of four slabs containing a total of eight bars is presented in terms of the average area of corrosion of reinforcement for each slab in Fig. 3.3.2. All slabs of Fig. 3.3.2 had 3/4 in. covers and the cement factor, water-cement ratio, and bar size were constant within each of two groups shown.

From Fig. 3.3.2 it can be seen that the use of Type V cement resulted in greater corrosion than that of Type III for the limited number of specimens studied. This relationship is perhaps unexpected, since it was previously concluded that the Type V cement produced the more impermeable concretes. No satisfactory explanation of this behavior was determined except that the difference in chemical composition of the two cements may be responsible for the improved corrosion properties of the Type III specimens of this study. However, the somewhat contradictory information concerning the influence upon corrosion of alumina and tricalcium aluminate in cements, as discussed in the literature review of Sec. 2.3.1, provides no substantiating information applicable to these observed results. In this regard the reader

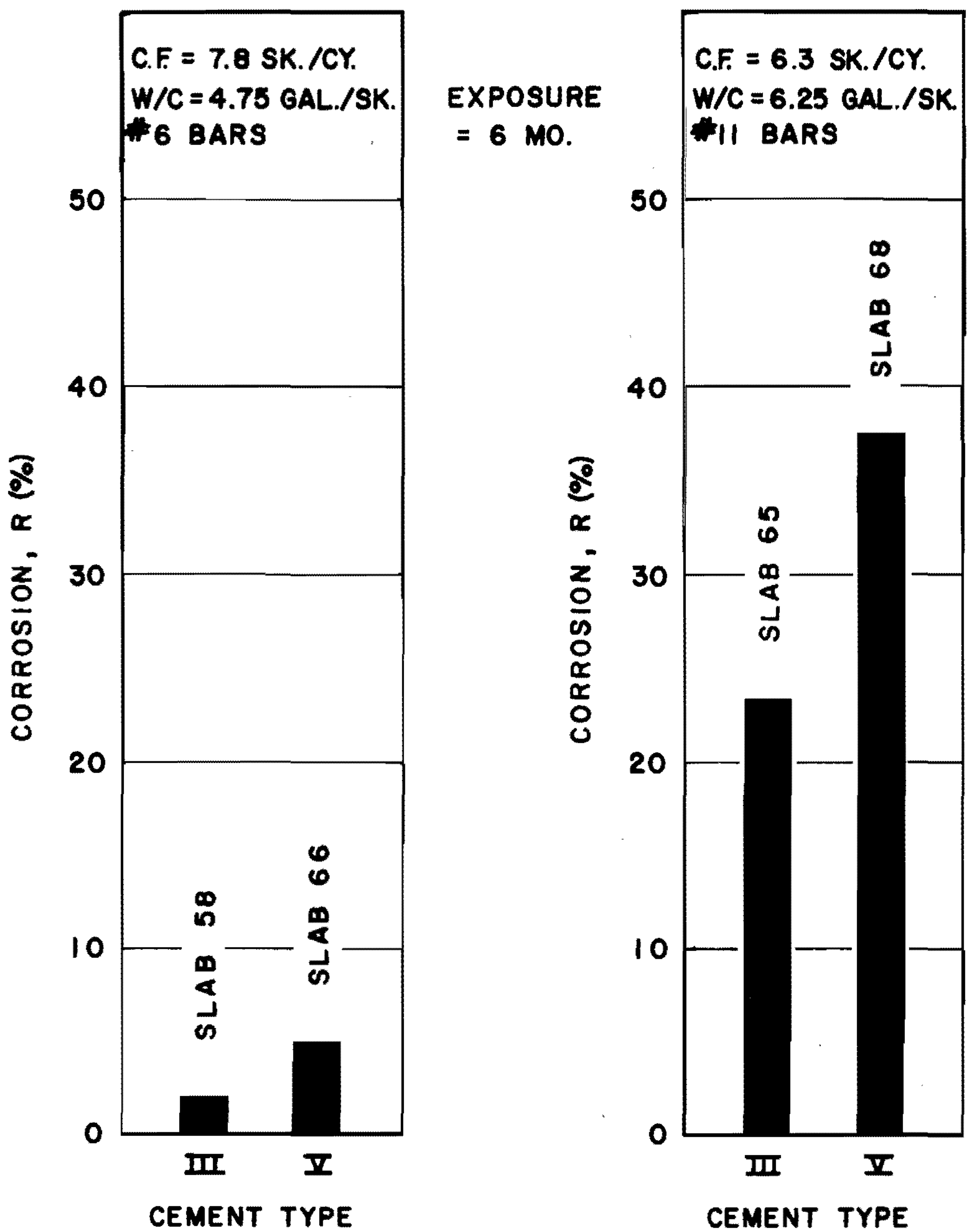


Fig. 3.3.2 Effect of cement type on corrosion of slab specimens with 3/4-in. cover.

should note that Type III cement contains approximately double the tricalcium aluminate content of Type V cement.[28]

3.3.2 Water-Cement Ratio. In analyzing the research data for effect of water-cement ratio, it was decided that the use of the unloaded slabs would be more meaningful since they are free of the influence of load induced cracking. The effects of structural cracking on corrosion are discussed in Sec. 3.5.1 of this study. A total of 28 slabs made of two different coarse aggregates and exposed for periods of 6 months and 24 months was available for the study of the water-cement ratio effect on corrosion. The results of this study are presented in Figs. 3.3.3 and 3.3.4.

For an exposure period of six months, 1/2 in. crushed limestone concrete slabs having covers of 3/4 in. and 1-1/2 in. were prepared at water-cement ratios of 4.75, 5.5, and 6.25 gal/sk, as shown in Fig. 3.3.3. Note that for the 3/4 in. cover the amount of corrosion is shown to be influenced by bar size. The bar size effect is more thoroughly discussed in Sec. 3.4.1 as the major emphasis of the present section is water-cement ratio effect.

From Fig. 3.3.3 it can be seen that for 3/4 in. covers there is a significant break in the corrosion relationship at a water-cement ratio of about 5.5 gal/sk. At ratios in excess of 5.5 the corrosion appears to be accelerated, while below 5.5 the resulting corrosion is relatively unaffected.

The corrosion results for the 1-1/2 in. cover slabs are shown near the bottom of Fig. 3.3.3 as all specimens showed essentially zero rusting. It is apparent that, due to the short period of exposure, the chlorides of the salt spray did not have sufficient time to penetrate the 1-1/2 in. cover even at the highest water-cement ratio of 6.25 gal/sk. This is contrasted to a maximum of about 24 percent rusting on #11 bars with 3/4 in. cover with the 6.25 gal/sk mix.

The corrosion results from #6 bars embedded in 3/8 in. siliceous gravel concretes for 24 months are given in Fig. 3.3.4. For these specimens, water-cement ratios of 5.5, 6.25, and 7.0 gal/sk were used. Note that for all cover values (3/4 in., 1 in., 1-1/2 in., and 2 in.) there is a significant decrease in corrosion as the water-cement ratio is decreased to 5.5 gal/sk. A similar result was seen in the data of Fig. 3.3.3. No firm corrosion

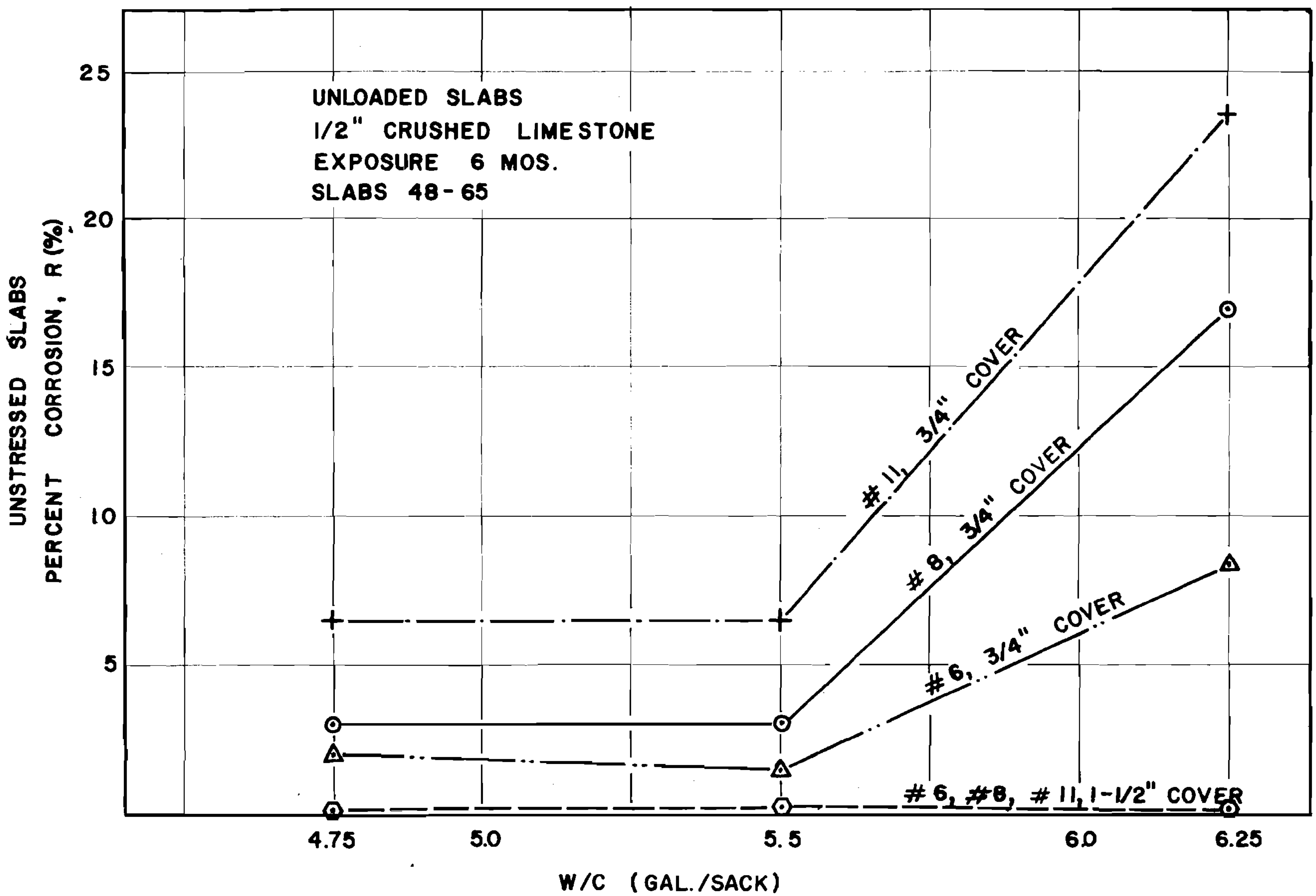


Fig. 3.3.3 Effect of water-cement ratio on corrosion of unloaded slabs made with crushed limestone aggregate.

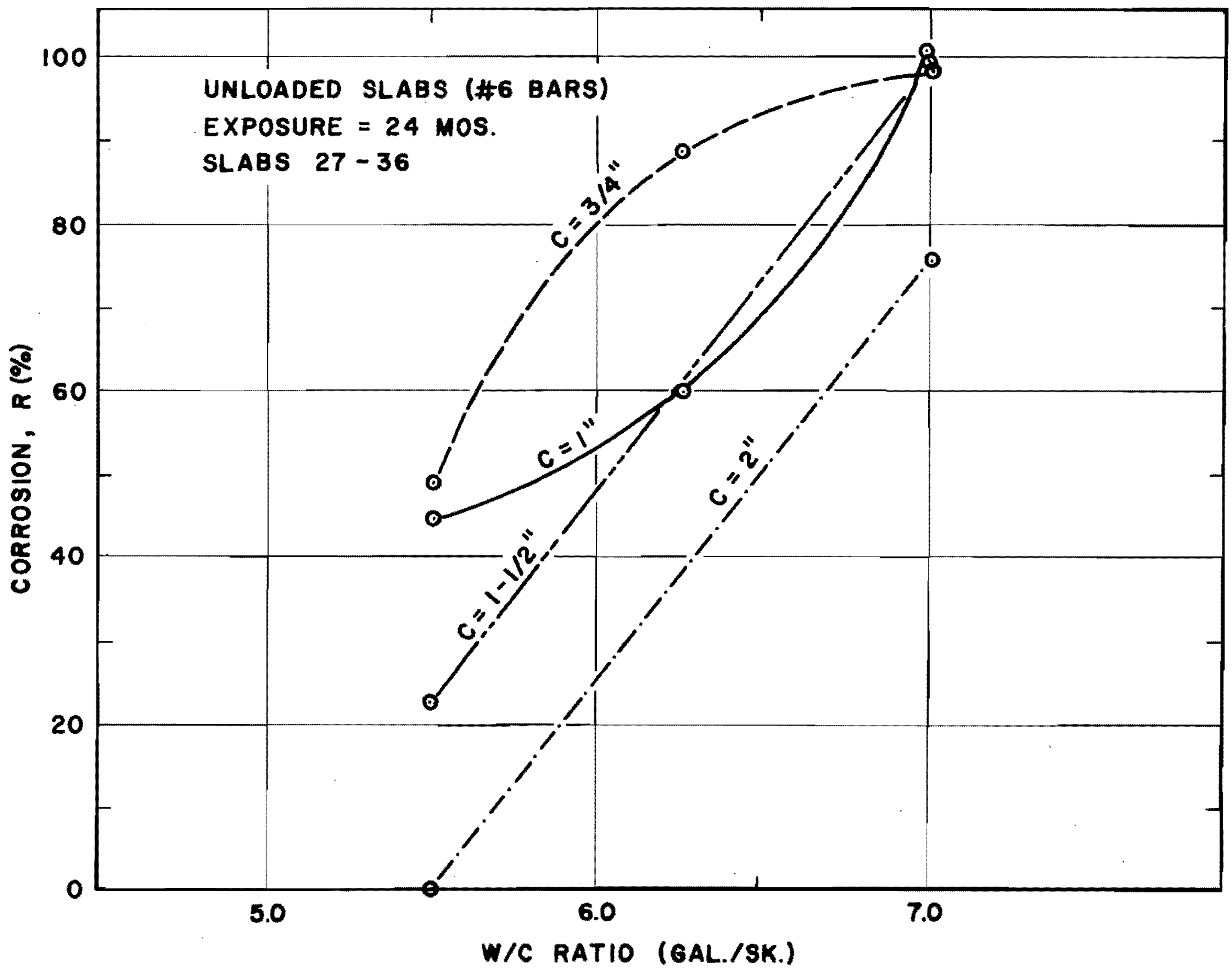


Fig. 3.3.4 Effect of water-cement ratio on the degree of corrosion of unloaded slabs made with siliceous aggregates.

projections can be made concerning the influence of lower water-cement ratios than those shown for the specific conditions of Fig. 3.3.4. However, it is likely that a continued decrease in corrosion or leveling off of the corrosion, as indicated in Fig. 3.3.3, would also be experienced for lower water-cement ratios of the siliceous aggregate concretes of Fig. 3.3.4.

From Fig. 3.3.4 it is interesting to note those conditions for which the extremes of corrosion were produced. For example, at a water-cement ratio of 7.0 gal/sk essentially 100 percent corrosion resulted for covers of 3/4, 1, and 1-1/2 in., while the 2-in. cover gave a relatively high value of about 75 percent. On the other hand, the 2-in. cover gave complete protection at the lowest water-cement ratio of 5.5 gal/sk, while the more shallow covers gave significantly less protection.

When it is considered that the exposure conditions for the slab tests of Fig. 3.3.4 were relatively severe in comparison to current field conditions in Texas, the indicated results should have value in projecting the relative effect of water-cement ratio in corrosion of reinforcement in structures under actual field conditions. For example, uncracked concretes made of relatively impermeable aggregates and having a water-cement ratio of 5.5 gal/sk with a 2-in. cover over the steel should provide excellent corrosion protection to structural concrete for periods of time well in excess of two years. In addition, the use of concrete having a water-cement ratio less than 5.5 gal/sk may not significantly lengthen the period of adequate corrosion protection if the data trend indicated for the 1/2-in. limestone concrete of Fig. 3.3.3 can be related to the siliceous aggregate concrete of Fig. 3.3.4.

Accurate projections for the maximum period of corrosion protection cannot be made from the data of this study; however, some idea of the corrosion rates experienced during this study is given in Sec. 3.5.4 of this report.

3.3.3 Aggregate Type. The inclusion of aggregate type as a corrosion test variable was not originally planned for this study. Relatively late in the research program lightweight aggregate and two additional grades of crushed limestone aggregates were added to provide a limited amount of data concerning the effect of this variable upon corrosion. As a result of these circumstances, direct comparisons of the corrosion data as a function of aggregate type is difficult because of variations in exposure time, cover, and bar size. Therefore, it is intended that the data presented here be used only in a limited manner for the relative evaluation of the aggregate type parameter. The data selected for this purpose are given in the four sections of Fig. 3.3.5 and represent both slab and uncracked, unstressed sections of beam specimens.

In Fig. 3.3.5(A) No. 11 bars were prepared with a 2-in. cover of 3/4-in. lightweight and 1-1/2 in. crushed limestone aggregate concrete having a water-cement ratio of 6.25 gal/sk. As can be seen in (A), less than 1 percent corrosion resulted for either type of concrete for an exposure period of 12 months. It can only be concluded from (A) that for 2-in. of cover, the lightweight and crushed stone aggregates performed comparably under the specified conditions of exposure.

In Fig. 3.3.5(B) the data for No. 6 bars in unloaded slab specimens made of concretes with 3/4-in. lightweight and 1/2-in. crushed limestone aggregates are presented. A direct comparison between the two is difficult because the cover value and exposure times are different. However, an interesting relative comparison can be made. In (B) it can be seen that a six month exposure on a 3/4-in. cover of 1/2-in. crushed limestone concrete produced five to six times the amount of corrosion found for a twelve month exposure of 1-in. cover of a 3/4-in. lightweight concrete of the same water-cement ratio. These results seem to imply that the lightweight concrete provided better corrosion protection than did the 1/2-in. crushed limestone concrete. However, due to the variation in the cover and exposure conditions, additional supporting data are necessary to make the conclusion firm.

In. Fig. 3.3.5(C) three concretes of 6.25 gal/sk water-cement ratio and 1-in. cover are compared. As in the previous case, direct comparison is difficult because of the difference in exposure time. Yet, by evaluating the results with the exposure difference in mind, useful comparisons result. First, the additional three months exposure on the 3/4-in. lightweight specimens produced about 2.8 times the corrosion of the 3/4-in. crushed

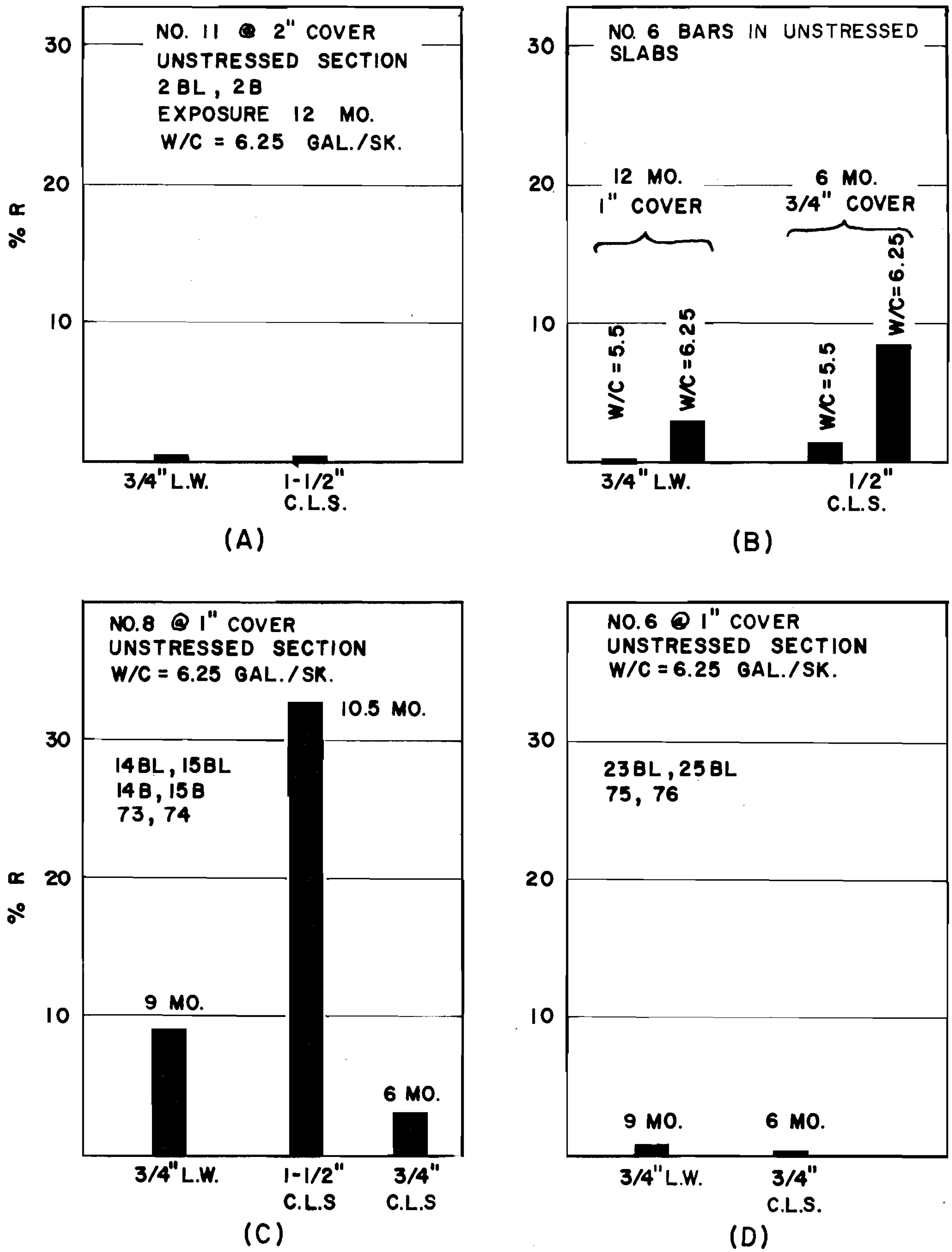


Fig. 3.3.5 Effect of aggregate type on corrosion.

stone concrete. On the other hand, an additional one and one-half months exposure on the 1-1/2 in. crushed stone specimen produced about 3.5 times the corrosion of the 3/4-in. lightweight concrete. In this instance as in that of (B) it again seems that a slight gain in corrosion protection might have been experienced for the lightweight concrete mix. Although this conclusion is only weakly supported by the research data, it does seem certain that the lightweight concrete used in this study was no less protective of the reinforcement than was the crushed stone concrete.

Finally, in (D) the unstressed sections of beams made with 3/4-in. lightweight and crushed stone aggregate concretes having a 6.25 gal/sk water-cement ratio and 1-in. cover are compared. In spite of the relatively shallow cover of 1-in., nine months of exposure produced less than 1 percent corrosion for the lightweight mix. In comparison, six months of exposure produced less than 0.5 percent corrosion for the crushed stone mix. This result tends to support the general trend that the lightweight concrete of this study provided at least comparable corrosion protection to that of the crushed stone mixes.

No suitable comparisons could be included in this section for concretes made of the siliceous coarse aggregate.

The reader is perhaps aware that the particle size distribution of a given aggregate type does affect the watertightness of the concrete mix. Unfortunately, it was not possible to study that parameter for a given aggregate type in this research program. However, an additional discussion of the effect of aggregate type upon permeability of concrete is given in the following section of this report.

3.3.4 Water Penetration. The relative permeability data reported in this section were obtained by the water penetration test as described in Sec. 4.7 of the Appendix. Penetration specimens were taken from plain concrete slabs and reinforced slabs at the time of their removal from salt spray exposure. Both sawed prism and cored cylinder specimens were prepared. Prism shaped specimens were approximately 12 in. x 12 in. x 6 in. thick and the cores were 6 in. long with 6 in. diameters. Concretes having water-cement ratios of 4.75, 5.5, 6.25, and 7.0 gal/sk were selected. These mixes were made with several types of coarse aggregate, including 3/8 in. siliceous aggregates, 1/2-in. and 1-1/2-in. crushed limestone and 3/4-in. lightweight aggregate. Water penetration data for these specimens are presented in Fig. 3.3.6 where each data point represents a single test with the exception of an average value plotted for the 1-1/2-in. crushed stone concrete at 6.25 gal/sk.

From Fig. 3.3.6 it is apparent that the water penetration data follow a somewhat unexpected relationship with respect to water-cement ratio for the 1/2-in. crushed stone and 3/4-in. lightweight aggregate concretes. In those cases there is an initial decrease in water penetration as the water-cement ratio is decreased to an intermediate value. Then, for the lowest values of water-cement ratio the penetration increases. Normally, it is reported that permeability decreases with decreasing water-cement ratio for a given aggregate type.[43] Attempts to explain this behavior in terms of mix design, slump and measured air contents were not successful. On the other hand, the data shown for the 3/8-in. siliceous aggregate concrete follow the expected relationship of decreasing penetration for decreasing water-cement ratio.

Comparison of the relative penetration for the different aggregate concretes is shown to be dependent upon the water-cement ratio. For example, at a water-cement ratio of 5.5 gal/sk it is clear that the least permeable concrete is the 3/8-in. siliceous aggregate mix followed by the 1/2-in. crushed limestone with the lightweight concrete being most permeable. The reader should keep in mind that the age difference in the concretes affects to some extent the permeability of the concretes with greater age producing less permeable concrete when all other parameters are equal. For a water-cement ratio of 6.25 gal/sk the relative water penetrations are not so clearly differentiated. Although the 3/8-in. siliceous aggregate concrete again shows the lowest relative permeability, the 1/2-in. crushed limestone shows a slightly greater permeability than the lightweight mix. Also, at this water-cement ratio the 1/2-in. and 1-1/2-in. crushed limestone concretes show about the same average penetration values. However, if the effect of age on permeability of the crushed stone mixes is considered, it is likely that for mixes of comparable age the 1-1/2-in. mix would exhibit the greater penetration.

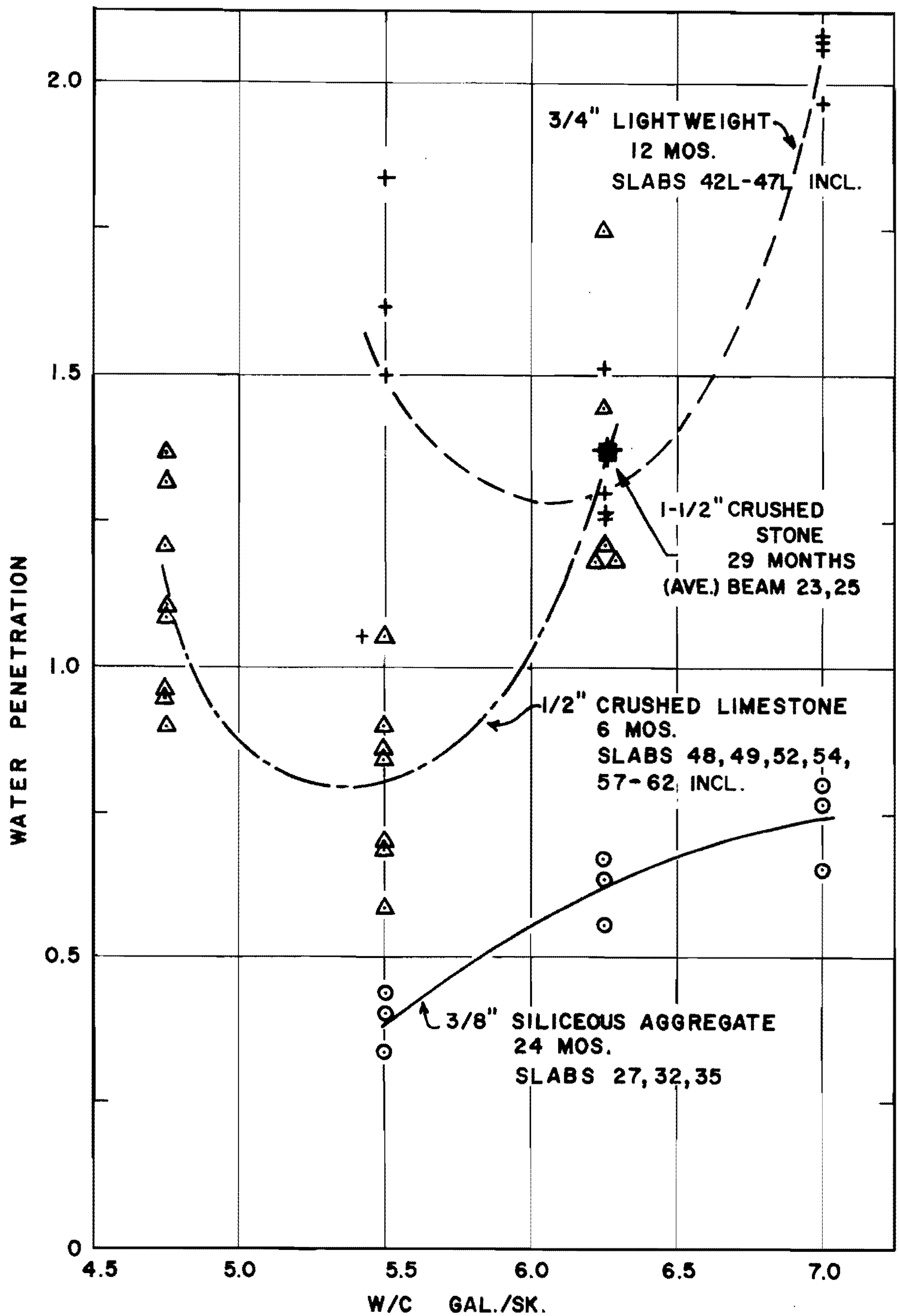


Fig. 3.3.6 Effect of aggregate type and water-cement ratio on water penetration into concrete.

At a water-cement ratio of 7.0 gal/sk a significant difference between the penetration values is seen for the siliceous and lightweight aggregate concretes. The penetration of the lightweight concrete is about 2.7 times greater than the siliceous aggregate concrete. No other aggregate comparisons were made at this water-cement ratio.

From the results of Fig. 3.3.6 the influence of the aggregate permeability upon the relative permeability of concrete is indicated. For example, even though the siliceous aggregate represents the smallest coarse aggregate used, this relatively dense aggregate produced more watertight concretes than any of the larger, more permeable aggregates for the exposure periods shown. It should be recalled from Chapter II that for a given aggregate, increasing the maximum particle size normally produces more watertight concretes. Apparently, this holds true only if the coarse aggregate is not significantly more permeable than the mortar fraction. This conclusion is also indicated by the comparison made for the two crushed limestone mixes at 6.25 gal/sk.

As has been mentioned previously, the inclusion of the water penetration study was initiated rather late in the research program. As a result, little data are available relating water penetration to corrosion. For those few specimens which were penetration tested, corrosion data were selected and plotted in Fig. 3.3.7. The reader should note in Fig. 3.3.7 that direct comparisons of corrosion for the three aggregates shown is not possible for several reasons. First, the cover values vary between the different types of concrete. Second, the concretes are of different age. However, Fig. 3.3.7 does provide useful information about the effect of water penetration upon corrosion for a given type of concrete.

For the 3/8 in. siliceous aggregate, the 24-month-old concrete shows a rapid increase in corrosion with increasing penetration, although the maximum 24-hour penetration for this concrete is a relatively low 0.75 in. Because there is a cover difference within this concrete type, the approximate corrosion relationship indicated by the dashed band is modified for the 1-1/2 in. cover data. It is perhaps significant to note that although this particular concrete does exhibit the lowest penetration values of all the specimens tested, the 1 in. cover was insufficient to prevent corrosion of the

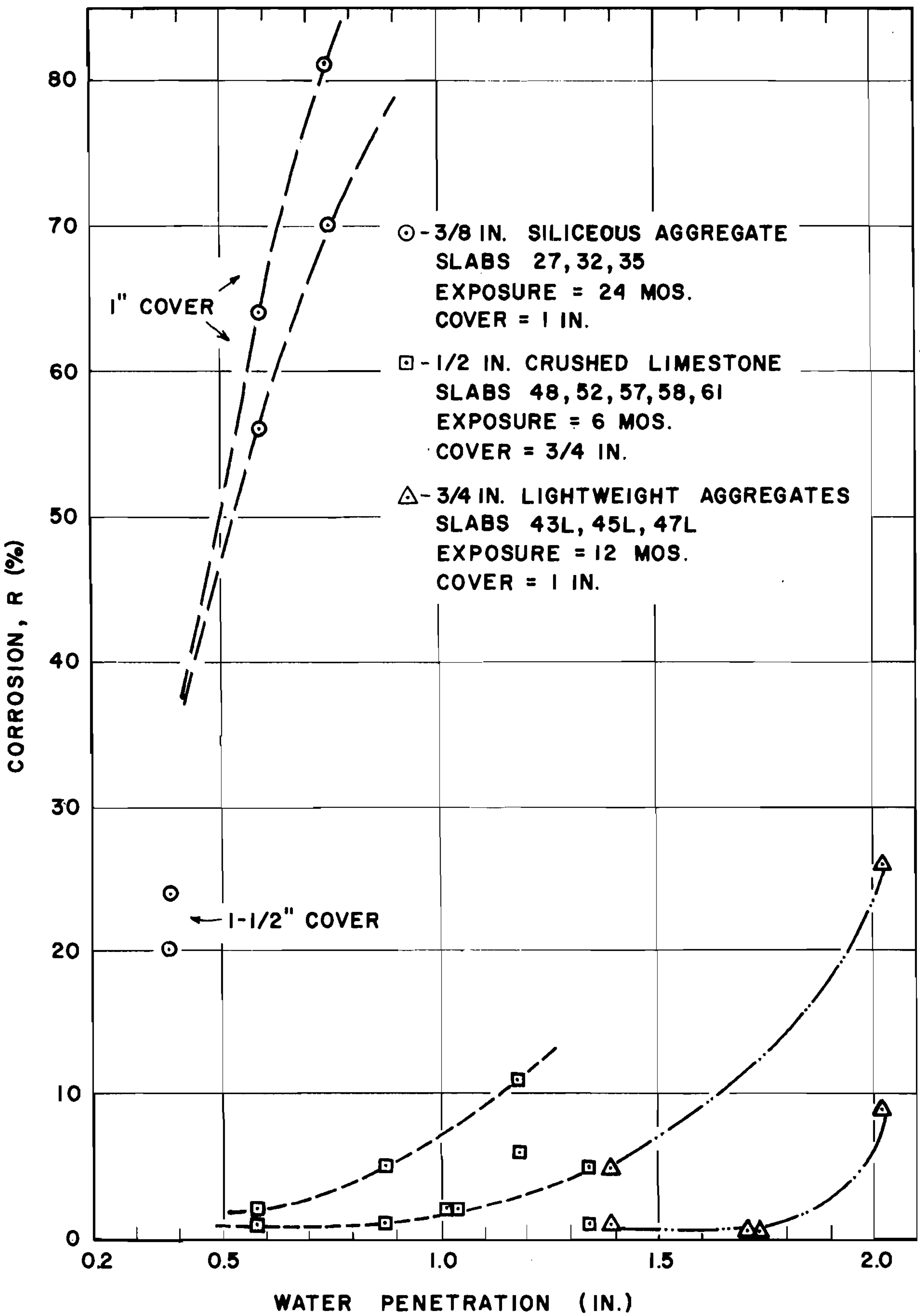


Fig. 3.3.7 The effect of water penetration upon the corrosion of selected concrete slabs.

reinforcement for 24 months while being subjected to the salt spray exposure.

In Fig. 3.3.7 are also shown corrosion data for the 1/2 in. crushed limestone and 3/4 in. lightweight aggregate concretes. Although these data are somewhat scattered, it seems that the general trend indicated is increased corrosion at the greater water penetration values for a given concrete type. Again, direct comparisons are difficult because of differences in age and cover.

The usefulness of the water penetration test as a concrete quality parameter affecting corrosion can be seen in Figs. 3.3.6 and 3.3.7. One might conclude by examining these data that in comparison to water-cement ratio, water penetration is the more reliable measure of corrosion resistance for a given type of concrete. This seems logical, since the water penetration test is performed on the actual concrete of the specimen and directly relates to the manner in which that concrete was cast, finished, cured, and aged. On the other hand, the water-cement ratio parameter can in no way account for the influence of these important field variables.

There are insufficient data in this research program to make a recommendation with respect to a maximum water-penetration value for concrete corrosion protection. It should be recognized that such specifications may necessarily be varied in certain cases for a few specific types of aggregates. For example, from Sec. 3.3.3 and Fig. 3.3.7, it can be reasoned that the lightweight concrete provided at least comparable corrosion protection to that of the crushed limestone in spite of the fact that the lightweight concrete exhibits the greater relative permeability. Apparently, there are additional, more subtle parameters which may influence the corrosion process when certain types of aggregates are used. One such parameter may be the influence of the aggregate upon the pH of the concrete. No specific effort was made to study this effect, but it is known that the pH of the solution made from crushing the lightweight aggregate used here and mixing it with distilled water is in the range of 8.5 to 9. It is, therefore, possible that the lightweight aggregate helps maintain a high pH in the concrete and thereby improves corrosion resistance. Certainly, additional research of such subtleties may prove of benefit in providing a clear understanding of the effect of aggregate type upon the degree of corrosion protection offered by a given concrete mix.

3.4 Placement of Steel and Concrete

3.4.1 Cover and Bar Size Effect. The effectiveness of the concrete cover in controlling corrosion of 3/8-in. prestress cables and No. 6 reinforcing bars is indicated in Fig. 3.4.1. Here, concrete slabs of 5.5 and 6.25 gal/sk were exposed for 24 months. Since the subject of prestress effects as related to corrosion is discussed in Sec. 3.5.3 only the nature of the cover influence is taken up in the current section.

From Fig. 3.4.1 it is clear that increasing the cover over the reinforcement provides greater corrosion protection to both prestress cables and reinforcing bars. The effect is quite pronounced and is approximately a linear relationship with cover for the 24 month exposure results shown. For the case of the 5.5 gal/sk mix, complete corrosion protection for 24 months resulted for the 2-in. cover over No. 6 bars. An additional discussion of the results of Fig. 3.4.1 is found in Sec. 3.5.3.

The influence of both cover and bar size on the corrosion of reinforcing bars of beam specimens is shown in Fig. 3.4.2. In this case the corrosion values on the unstressed bars are compared for concretes with 1-1/2 in. limestone aggregates and 6.25 gal/sk. From Fig. 3.4.2 it is again seen that increased cover results in greater corrosion protection. For example, considering No. 8 bars, 1 in. of cover resulted in approximately 95 percent corrosion at 24 months of exposure, while 3 in. of cover gave about 12 percent corrosion after 31 months of exposure. A similar trend is indicated for No. 6 bars although no 3 in. cover data were obtained in that case.

In comparing Figs. 3.4.1 and 3.4.2 it is of interest to note the degree of corrosion on the No. 6 bars at 2 in. of cover. It is apparent that water-cement ratios as high as 6.25 gal/sk provide good corrosion protection for the indicated exposure conditions and 2 in. covers.

An interesting result of this research not noted in the literature review was the effect of bar size on corrosion. The effect is clearly seen in Fig. 3.4.2 for covers of 1 in. and 2 in. In both cases decreasing bar sizes resulted in lower values of corrosion for the given exposure conditions.

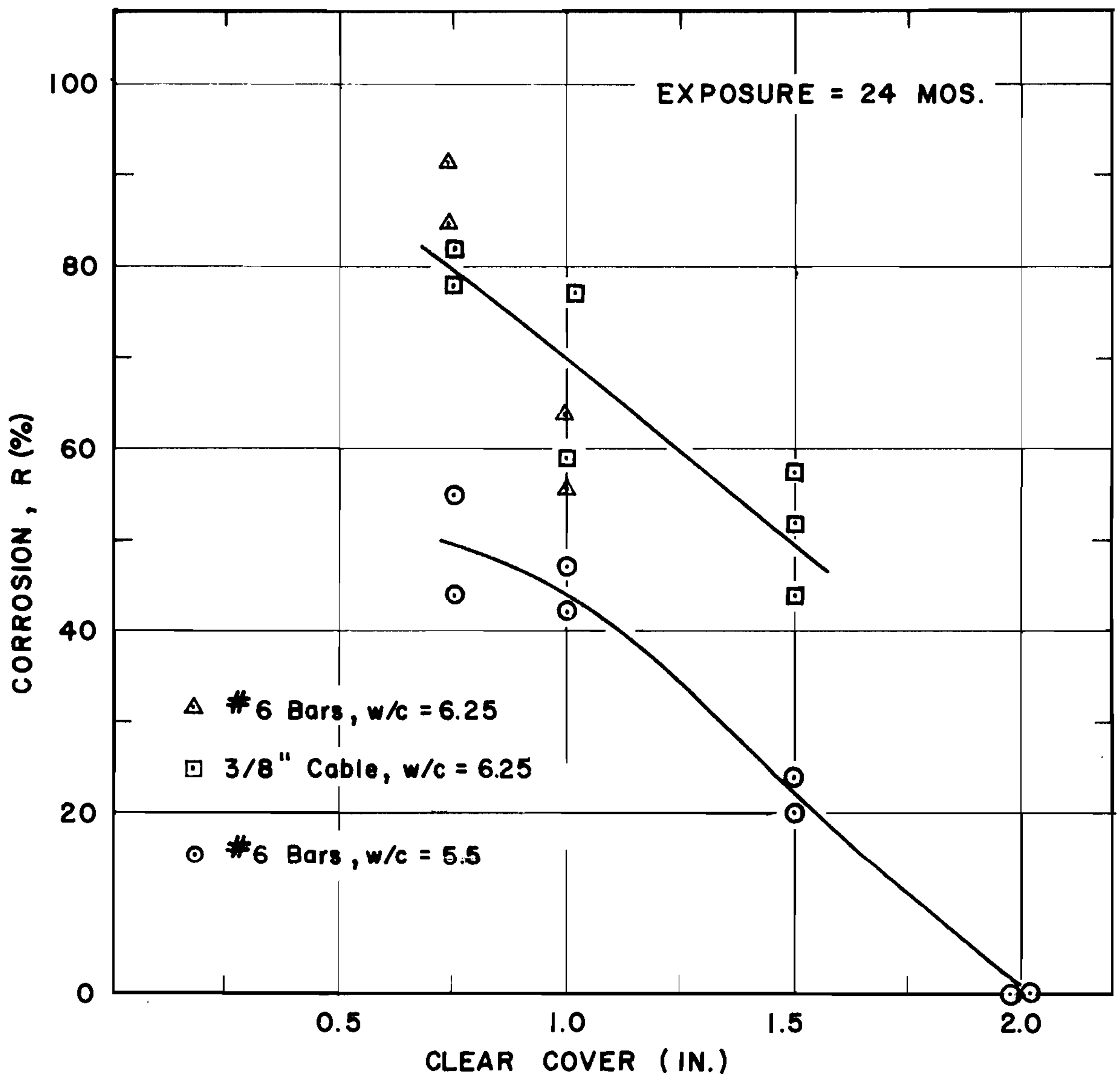


Fig. 3.4.1 Corrosion of reinforcing bars and prestress cables in slabs made of 3/8-in. siliceous aggregate.

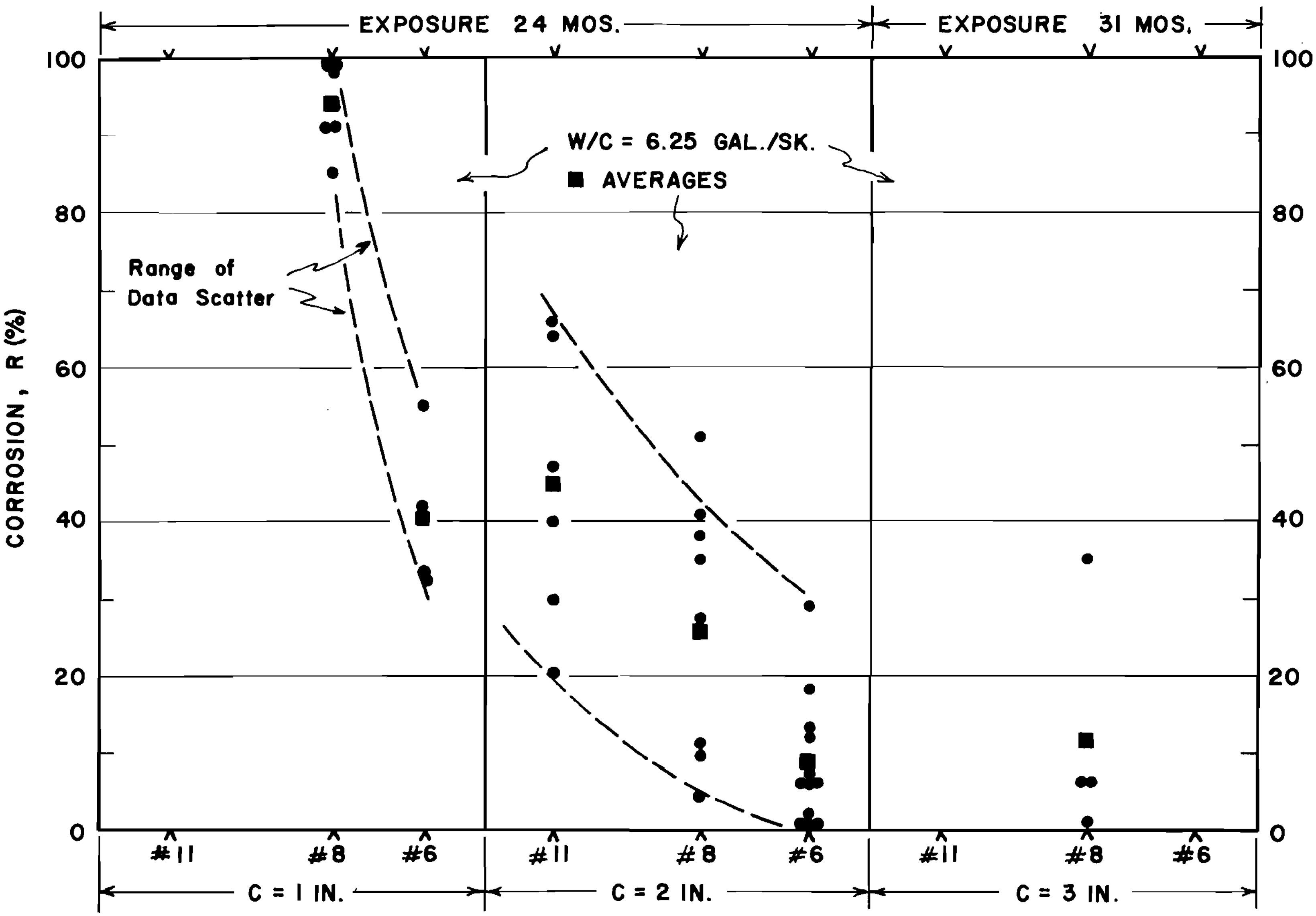


Fig. 3.4.2 Effect of clear cover and bar diameter upon corrosion of unstressed bars in beams of 1-1/2 in. crushed limestone aggregate concrete.

The bar size effect can be visually observed in Figs. 3.4.3 and 3.4.4 where cut sections of selected beams exposing the reinforcement are shown. Corrosion staining of the concrete and rusted areas oi the bars correspond to the dark spots in the photographs. Photography angles are given by the arrows shown on the beam cross section view of the identification plate for each bar shown. The reader may wish to compare the appearance of the bars with the corresponding rust percentages given for each case. Generally, the reverse side of each bar shown in both Figs. 3.4.3 and 3.4.4 exhibits less corrosion than that seen in these photographs.

At this point the reader should note that corrosion results reported thus far primarily pertain to unstressed reinforcement with the exception of brief mention of prestress cable as seen in Fig. 3.4.1. The photographs of Fig 3.4.3 and 3.4.4 also contain reference to stressed bars for visual comparison only. Effects of steel stress and flexural cracking upon corrosion are specifically discussed in sections 3.4.2, 3.4.3, and 3.5.1 through 3.5.3 of this report.

In Fig. 3.4.3a and b, note that for a cover of 1 in. the No. 8 bars are considerably more rusted than the No. 6's. A similar bar size effect is observed for No. 11, 8, and 6 bars of Fig. 3.4.4a, b, and c. The reader should also note that for each beam shown, those bars located at the top of the specimen (T) during exposure are generally more corroded in comparison to the bars located at the bottom of the beams.

That increasing concrete cover provides increased corrosion protection is perhaps an abvious result of any corrosion research study. The initiation of corrosion requires that certain specified elements of the corrosion reaction penetrate the concrete cover to the level of the reinforcing steel. Naturally, the time required for these elements to reach the reinforcing bars is dependent upon the thickness and permeability of the concrete cover, provided the cover is relatively free of cracks. However, if flexure or shrinkage cracks exists, much more rapid penetration of liquids will occur in the immediate vicinity of the cracks. The effects of cracking on corrosion are discussed in section 3.5.1.

After penetration has occurred and corrosion has been initiated, the influence of bar size becomes predominant in the course of the ensuing corrosion process. For the larger bars greater volumes of rust are produced. It follows that the expansive forces accompanying the formation of rust more readily produce splitting of a given concrete cover when the bars are large. Once the

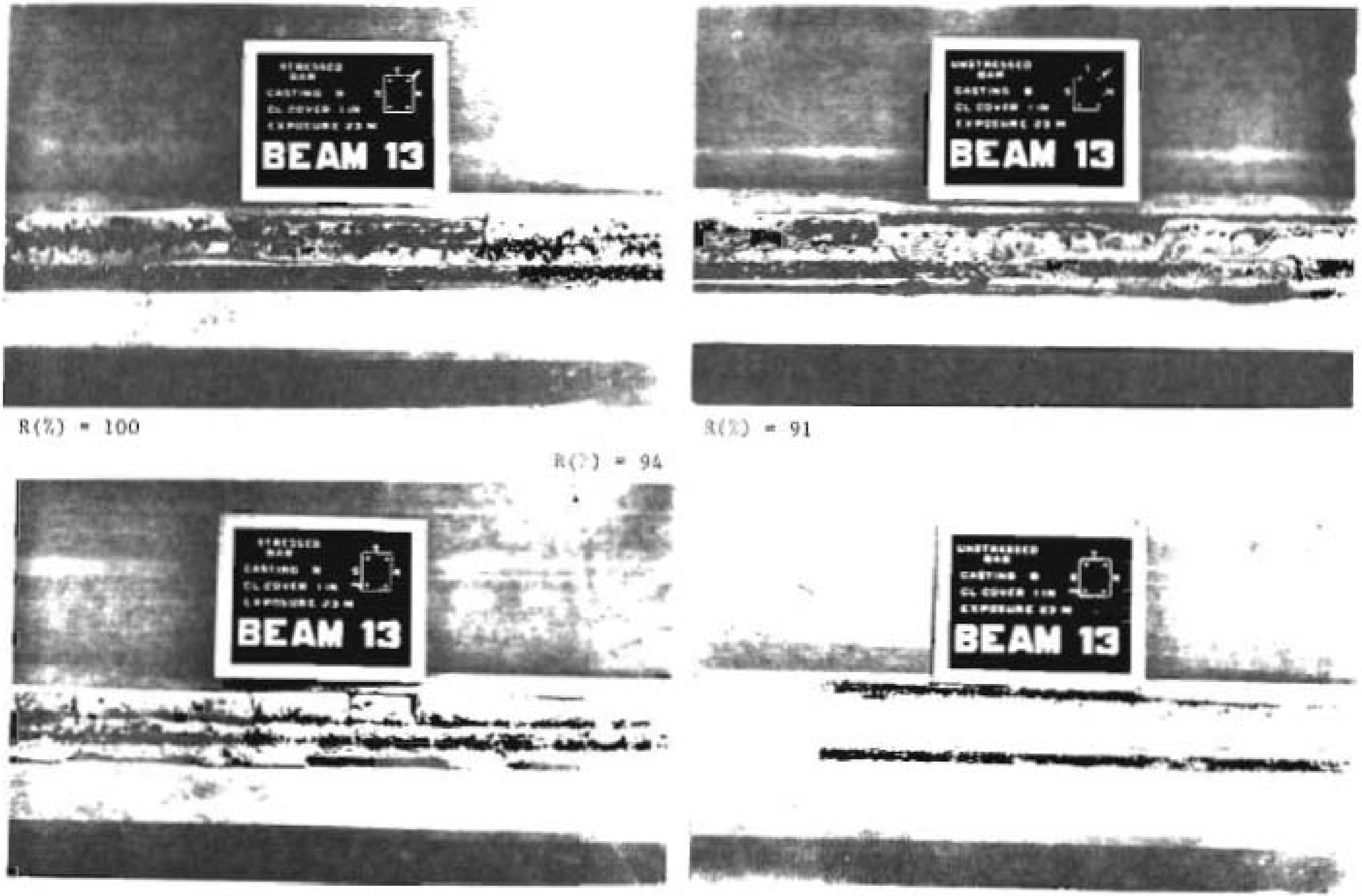


Fig. 3.4.3.a Corrosion of #8 top and #6 bottom bars with 1-in. cover in Beam 13.

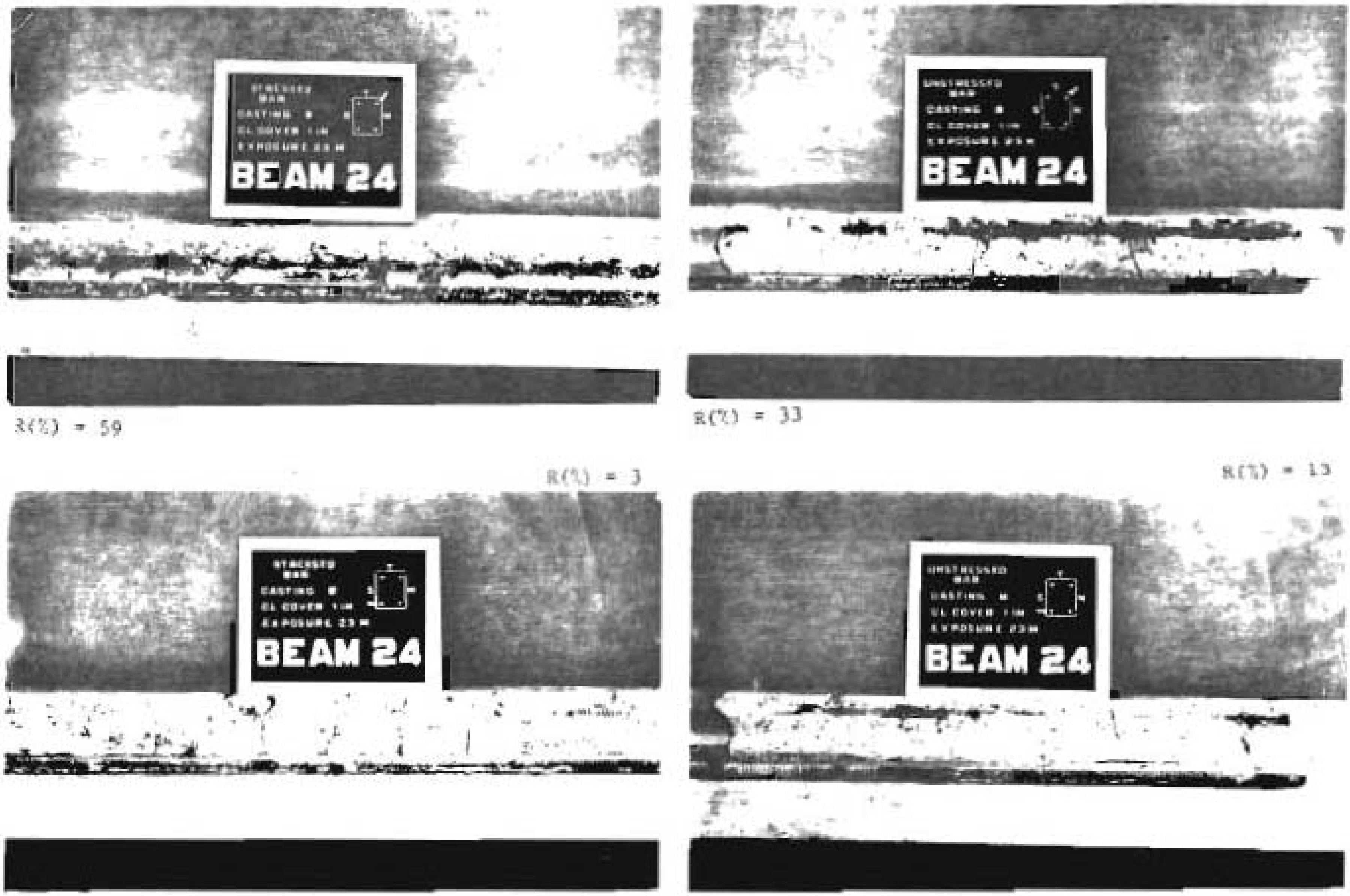


Fig. 3.4.3.b Corrosion of #8 top and #6 bottom bars with 1-in. cover in Beam 24.

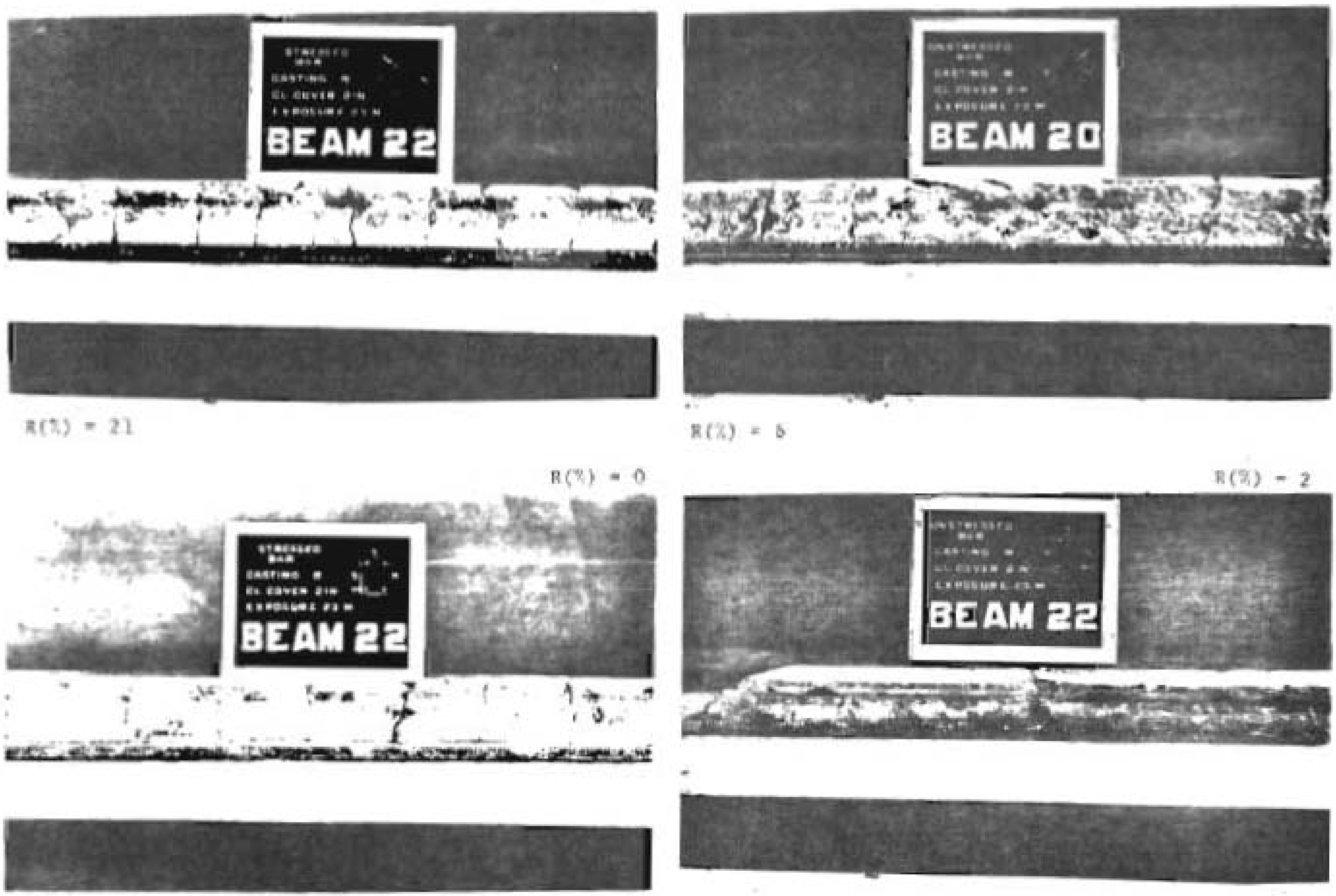
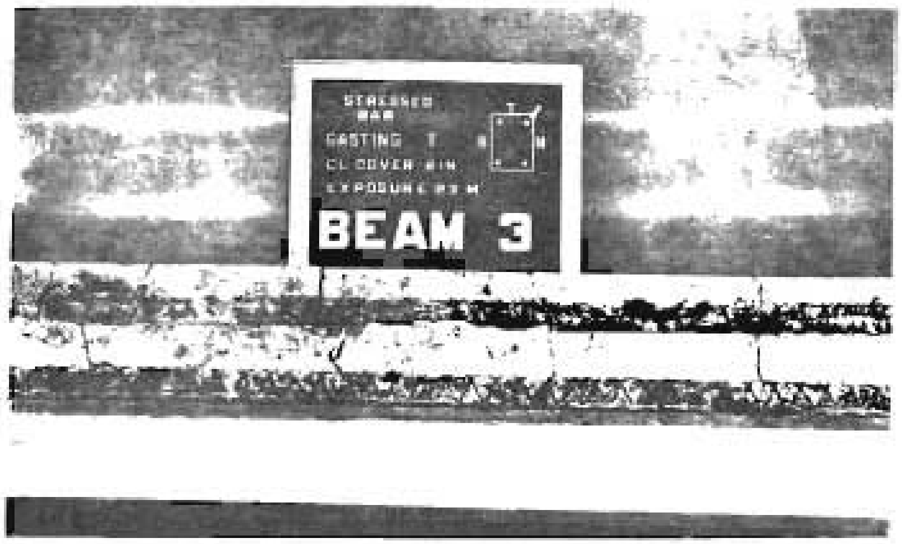


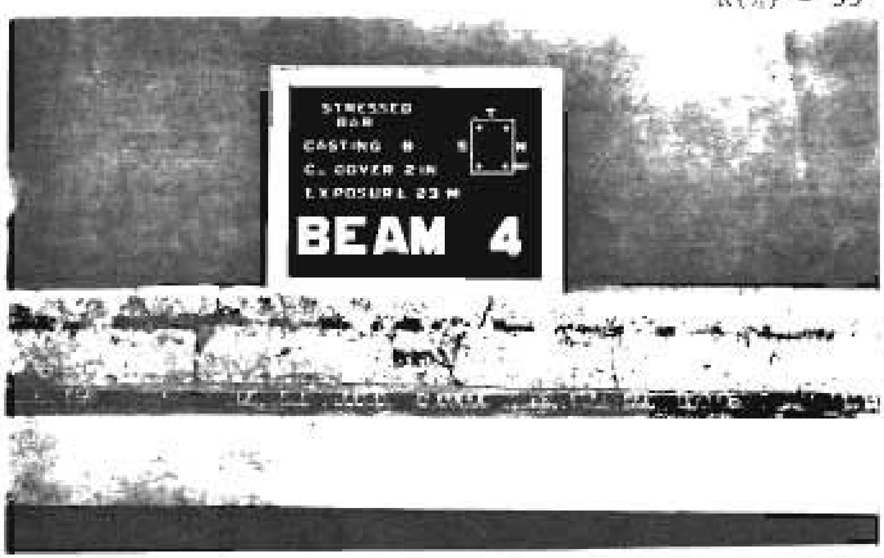
Fig. 3.4.3.c Corrosion of #6 bars with 2-in. cover in Beams 20 and 22.



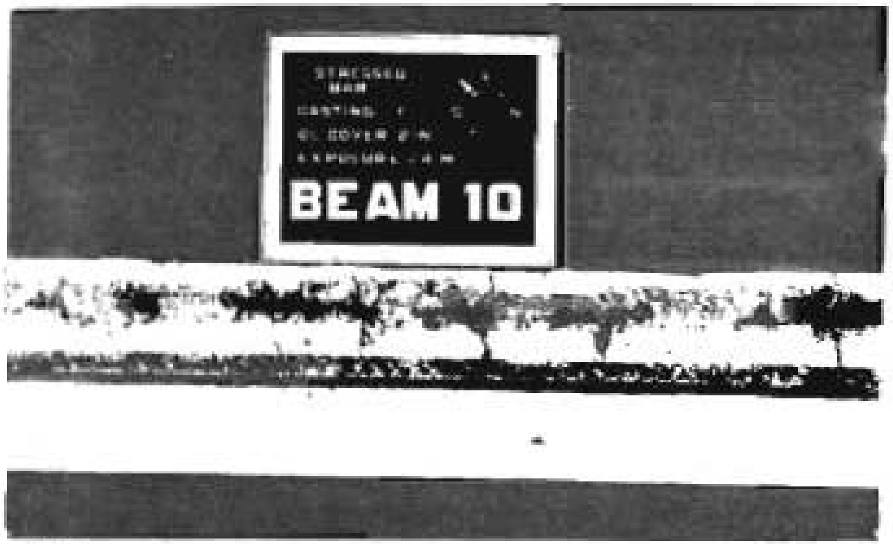
R(%) = 52

R(%) = 30

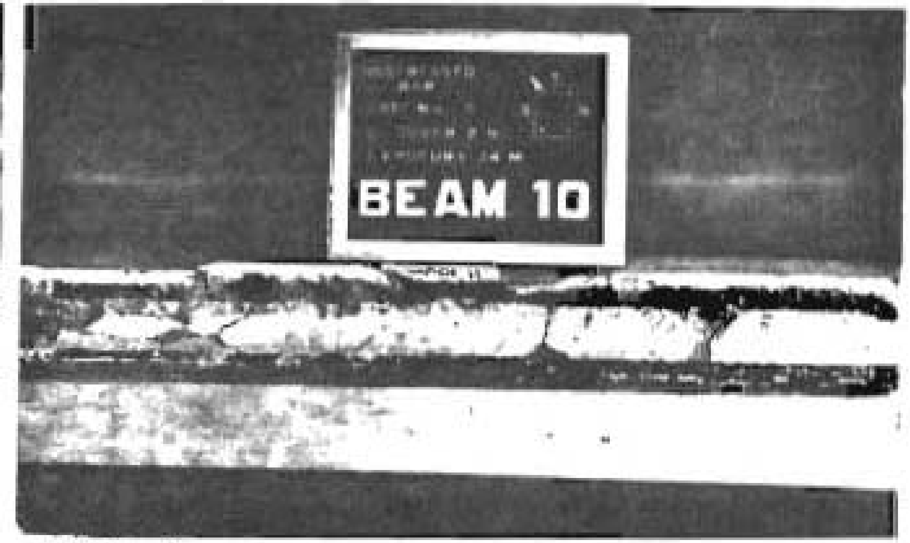
R(%) = 33



R(%) = 43

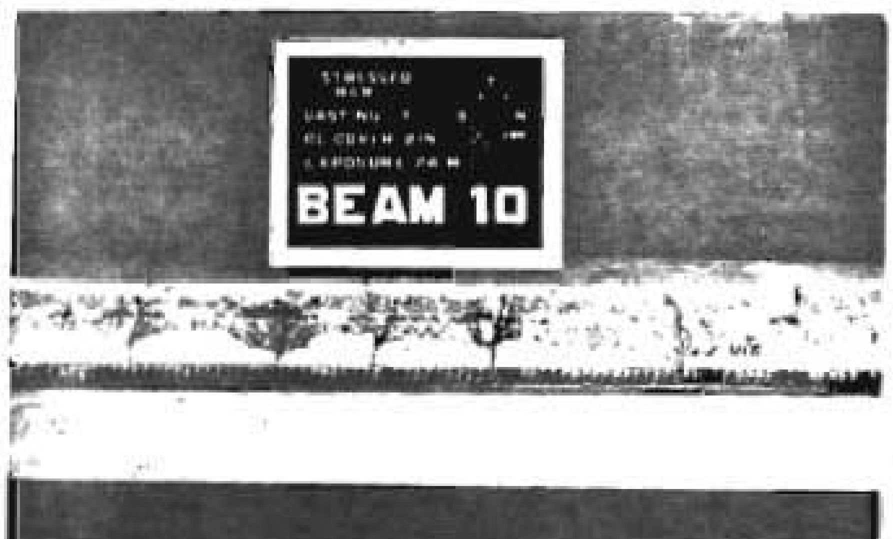


R(%) = 25



R(%) = 41

R(%) = 22



R(%) = 32

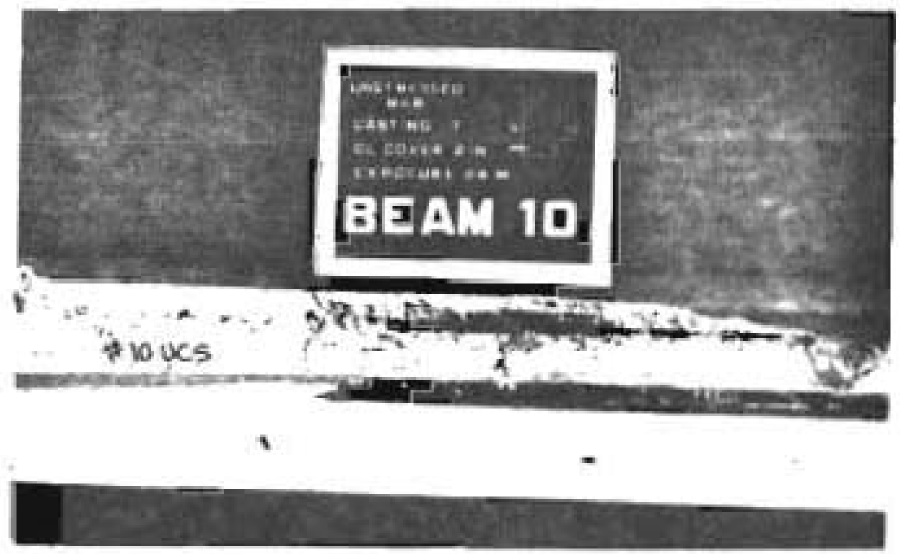


Fig. 3.4.4.b Corrosion of #8 top and #6 bottom bars with 2-in. cover in Beam 10.

cover is split longitudinally over a bar, corrosion is accelerated by the rapid access of the corrosion producing elements through the cracked cover to the level of the steel.

In considering both the role of cover and bar size in the corrosion process, it is perhaps apparent that even for large bars corrosion can be inhibited by the use of large concrete cover. In that case the time for penetration of the cover would be increased and the expansive forces required to split the cover during corrosion would be larger due to the increase in the area of the concrete being stressed. It is thus important that the interaction of both cover and bar size be considered singly in determining their combined effect in the corrosion process.

Such an effect was observed when a single parameter C/D (clear cover divided by bar diameter) was plotted vs. corrosion in beams and slabs as shown in Fig. 3.4.5. In Fig 3.4.5 concrete specimens of 6.25 gal/sk and exposures of 24 months are compared. Within the range of C/D ratios of 1.0 to 2.7, a definite trend to better corrosion protection at the higher values of C/D is clear. For a C/D of 1.0 all specimens exceeded 80 percent corrosion, while at a C/D of 2.7 no specimen exceeded 30 percent corrosion. It is also interesting to note in Fig. 3.4.5 that even for different bar sizes, similar C/D ratios result in similar ranges of corrosion. For example, No. 6 bars with 1 in. cover (C/D = 1.33) and No. 11 bars with 2 in. cover (C/D = 1.41) experience similar degrees of corrosion.

The results of Fig. 3.4.5 are of importance in the design of reinforced concrete structures subject to corrosion elements. It seems certain that a specification for concrete cover is more likely to produce a corrosion resistant structure if the bar size effect is also considered in the formulation of that specification. Unfortunately, the results of this research study are too limited for the selection of firm recommendations of C/D values to produce corrosion resistance structures. However, the authors do feel that a tentative selection of a C/D value of $\geq$ 3.0 should result in reasonably good corrosion protection at cover values not too different from current design practice.

3.4.2 Bar Spacing. The significant effect of the bar size on resulting corrosion, as discussed in the previous section, suggests the possible advantage of using smaller, more closely spaced bars to provide the total required

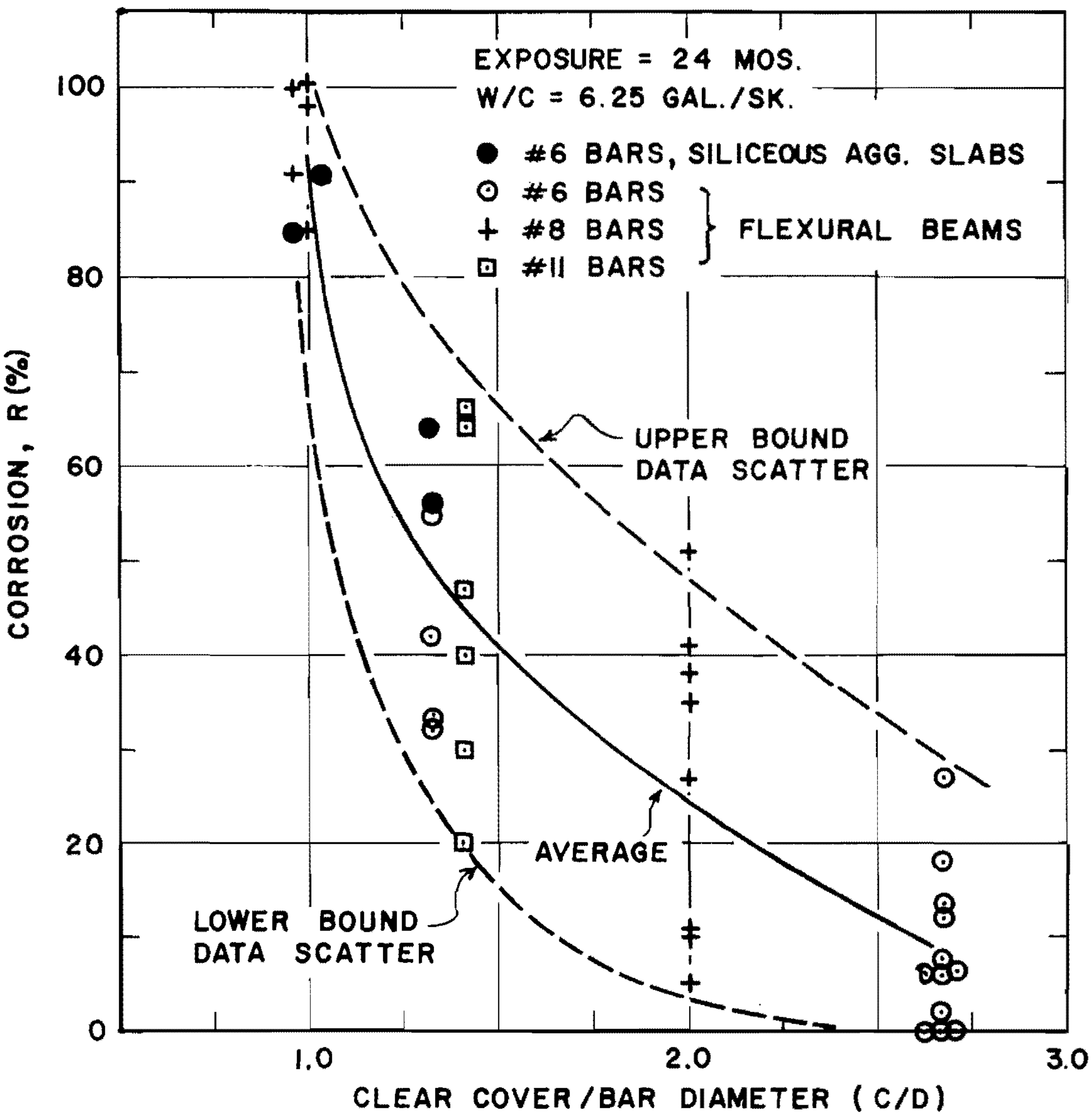


Fig. 3.4.5 Effect of C/D ratio upon corrosion of unstressed reinforcing bars in beams and slabs.

percentage of steel area. Late in the research program four beams with 1. in. cover were cast in order to explore this possibility, two with two No. 11 tension bars and two with four No. 8 tension bars, which resulted in the same steel percentage (beams 69, 70, 71, 72).

At the end of six months of exposure, the beams were unloaded and the bars were analyzed. Figure 3.4.6 shows the resulting data for the effect of bar spacing on corrosion and splitting of these four beams. It is seen that the average corrosion of the No. 8 bars is less than one-half that of the No. 11 bars. Also, the average percentage of longitudinal splitting on beams with No. 8 bars is about one-third that on beams with No. 11 bars. A detailed discussion of the method used to determine longitudinal cracking of the specimens is found in the Appendix section 4.5.2.

The above results help to substantiate the previous conclusion that the degree of corrosion deterioration is very closely associated with the C/D ratio. The beams with No. 8 bars had a C/D ratio of 1.0, while beams with No. 11 bars had a C/D ratio of 0.71. There was a large increase in corrosion deterioration as the C/D ratio decreased from 1.0 to 0.71. Since the clear cover and concrete mixes were identical, the resulting difference in corrosion was due to the bar size effect; the larger bars produced greater splitting forces which led to greater longitudinal splitting and, in turn, to more extensive corrosion.

With regard to bar spacing, another question may arise as to how closely the bars can be spaced. It is possible that during the early stages of corrosion very closely spaced bars may produce a weakened horizontal plane, especially when coupled with critical bond stresses. Under these circumstances severe spalling of the concrete at the level of the steel would very likely result. However, such spalling was not observed for beams 71 and 72, since the exposure time was relatively short (six months). Spalling would probably result for longer exposure times although the critical bar spacing to cover relationship would influence the nature of the resulting cracking. Selection of the preferred bar size in relation to spacing and cover, in order to minimize corrosion and spalling damage, requires additional research.

3.4.3 Position of Casting. Because the effectiveness of consolidation of the fresh concrete greatly influences the permeability of the hardened concrete, uniformity of consolidation from bottom to top of the form is highly

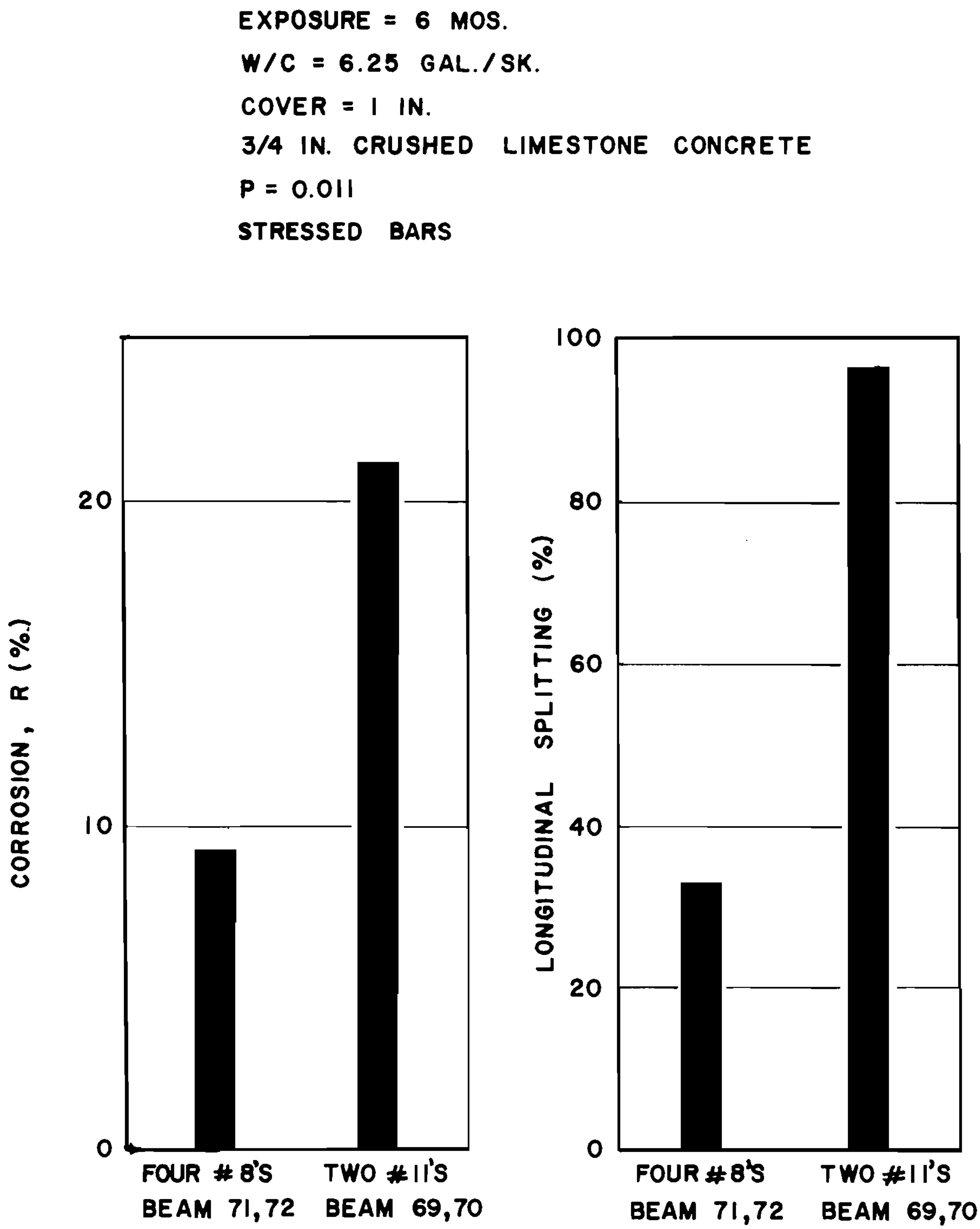


Fig. 3.4.6 Effect of bar spacing on corrosion and splitting of concrete beams of equivalent steel percentages.

desirable, especially with respect to corrosion protection. In the case of nonuniform consolidation, corrosion cells are likely to be initiated.

Achieving uniform consolidation with depth in a beam is difficult for several reasons. First, the greater fluid head on the concrete at the bottom of a beam results in better consolidation at that level in comparison to that of the top section. Second, bleed water rising to the surface of fresh concrete results in poorer quality hardened concrete at the upper zone of the member. The bleed water also tends to collect beneath the top bars of a structural element and produce nonhomogeneous concrete immediately adjacent to the bars. Corrosion cells are promoted under these conditions.

In order to determine if the corrosion results of this study were influenced by the effects of variation in consolidation, several specimens, identical except for position of casting, were compared. The results of the comparison are given in Fig. 3.4.7. Data identified as bottom or top cast refer to the location of the analyzed bars at the time of casting.

It can be seen in Fig. 3.4.7 that in every case, whether stressed or unstressed, bottom cast bars experienced less corrosion. Even though the data are limited in this comparison, it does seem likely that a variation in quality of the concrete with depth resulted and that this variation produced lower corrosion resistance for top cast bars, the difference observed being relatively small in some cases.

3.5 Exposure and Loading

3.5.1 Concrete Cracking. The locations and widths of initial flexural cracks along the bar in the constant moment zone of two flexural beams are shown in Fig. 3.5.1. The crack widths are expressed on a vertical scale, the length of the lines indicating the width of the cracks. Below are plotted the rust distribution profiles for the corresponding bars. The data shown represent in general the outer bounds of the range of crack patterns and rust profiles observed during this research program.

The upper plot of Fig. 3.5.1 corresponds to a beam with 1 in. cover, 3/4 in. crushed limestone concrete exposed for a duration of six months. The lower plot represents a beam with 2 in. cover, 3/4 in. lightweight aggregate, and exposed for a duration of twelve months. Even though these two beams have

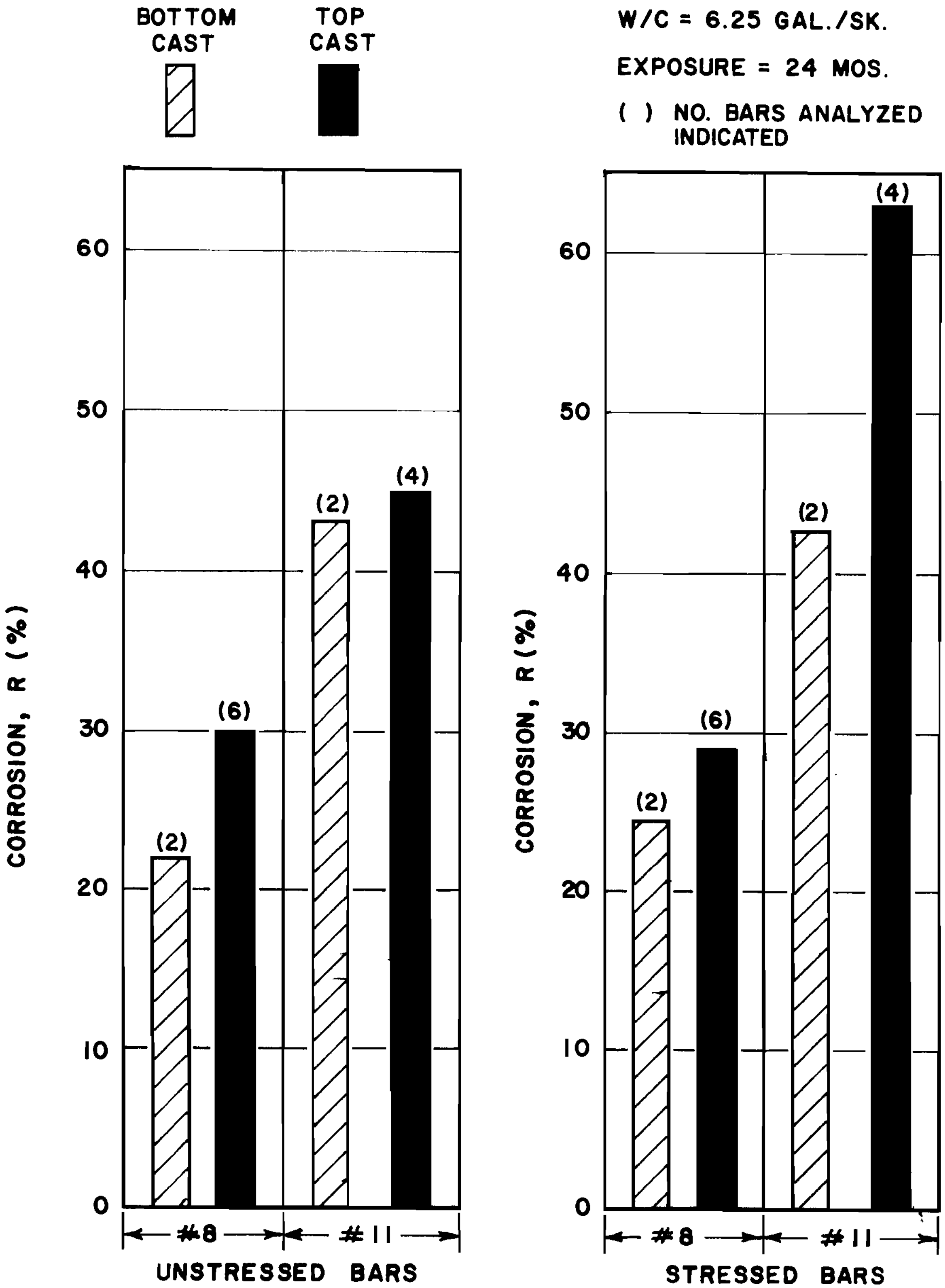


Fig. 3.4.7 Comparison of corrosion of bottom and top cast bars with 2-in. cover of 1-1/2 in. crushed limestone aggregate concrete.

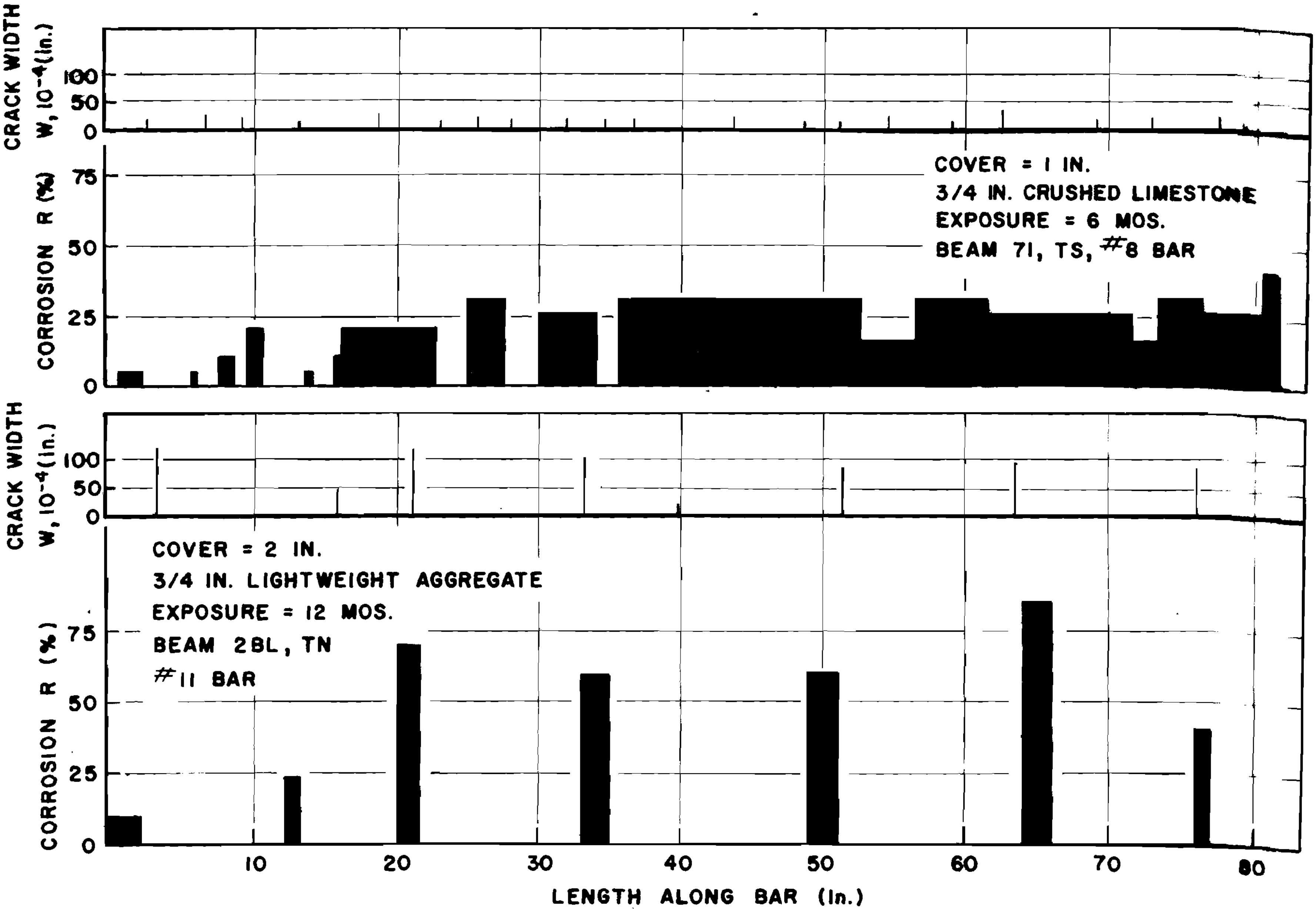


Fig. 3.5.1 Influence of initial width and location of flexural cracks upon corrosion of No. 8 and No. 11 bars in concretes of water-cement ratio = 6.25 gal./sk.

similar cross-sectional dimensions, the surface cracks are wider in the 2 in. cover beam which was stressed at 20 ksi compared to 30 ksi for the 1 in. cover beam. This is due to the crack widening effect which increases almost linearly with cover while the crack width at the bar is much less affected, being a small percentage of the surface crack width.[44]

The contrasting nature of the crack patterns and rust profiles of Fig. 3.5.1 are helpful in determining the relative influence of crack width and cover upon the resulting corrosion. It will be recalled from section 2.5.7 that crack widths of more than about 40×10^{-4} in. have been reported to induce serious corrosion conditions. However, from the data for Beam 71 it is apparent that significant and somewhat uniform corrosion had developed in a period of six months, although all crack widths were considerably below 40×10^{-4} in. In this case the relatively shallow cover, 1 in., is clearly insufficient to inhibit corrosion even when crack widths are minimized. On the other hand, the data shown for Beam 2BL with 2 in. cover definitely indicates that cracks can play an important role in the initiation of corrosion. In this beam most of the crack widths are considerably greater than 40×10^{-4} in. and in almost every case these cracks have promoted corrosion of the bars at the crack location while essentially no corrosion was observed between cracks. Apparently the 2 in. cover, when uncracked, was sufficient to protect the bars of this particular beam for a period of twelve months.

In comparing the data of Beams 71 and 2BL it is clear that crack widths and concrete cover are integrally related in their effect upon corrosion. The setting of crack width limits to minimize corrosion should also include corresponding cover requirements in order to be meaningful. Cover requirements referred to here include concrete quality as well as depth of cover.

When the test specimens were removed from the exposure site and bars analyzed, it was seen that usually there was not much difference between the corrosion of stressed and unstressed bars of the same beams. It was initially presumed that considerably more corrosion would occur on stressed bars which are accompanied by flexural concrete cracking. In order to compare the resulting corrosion in stressed and unstressed regions of all flexural beam specimens, corrosion percentages for stressed top bars versus unstressed top bars were plotted, as shown in Fig. 3.5.2. If the degrees of corrosion on both stressed

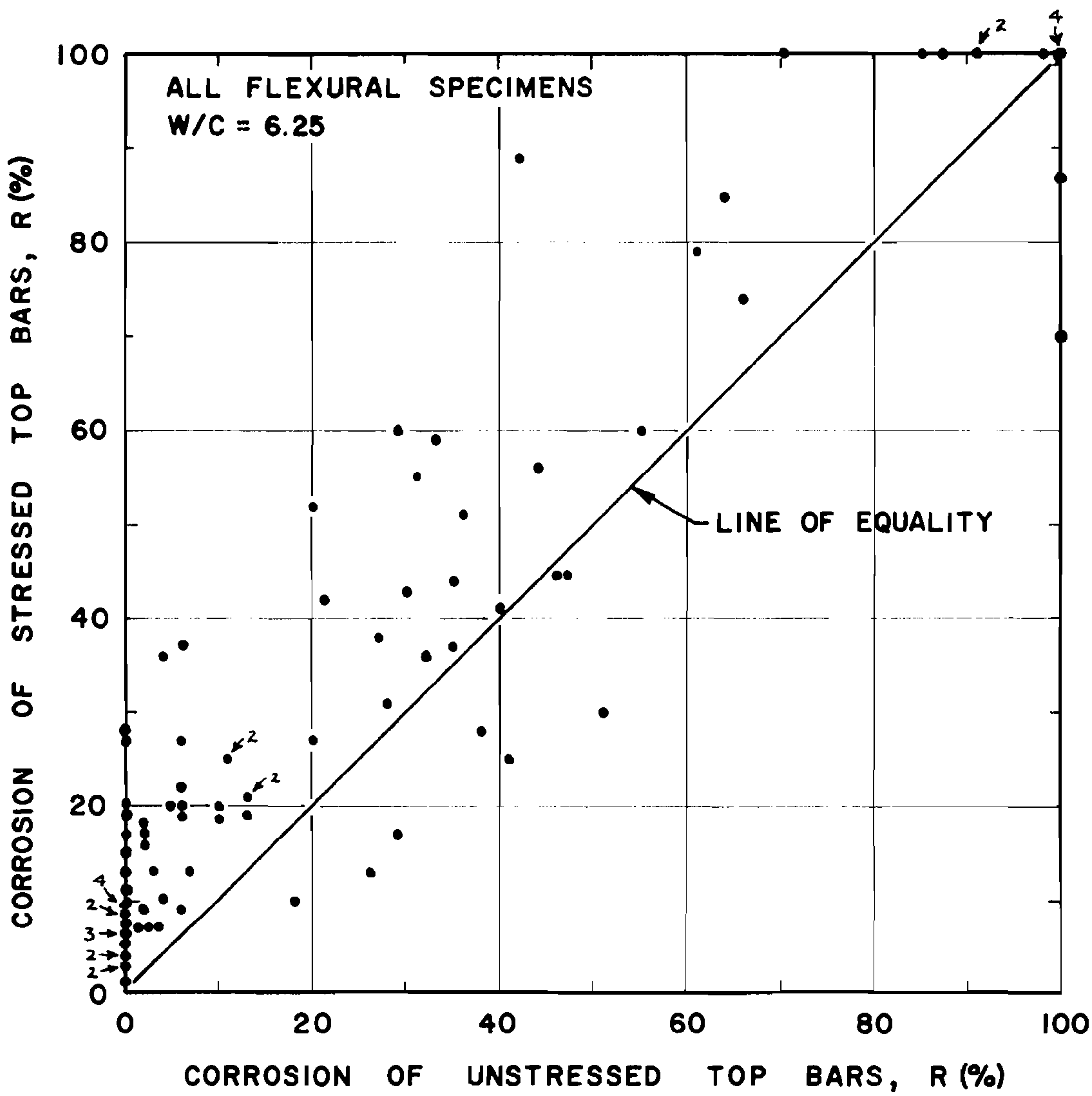


Fig. 3.5.2 Relative corrosion on stressed and unstressed top bars of all flexural beams.

and unstressed portions of a given bar were equal, the point would fall on the line of equality (45^{o} line). However, it can be seen that most data points indicate about 10 to 15 percent more corrosion on the stressed bars in comparison to unstressed bars. This difference apparently reflects the combined effect of the existence of cracks and steel stress on the degree of corrosion of bars in the constant moment zone.

It should be pointed out that the data of Fig. 3.5.2 do not emphasize the maximum possible severity of the influence of cracking and stress upon corrosion. This is because of the exposure timing of the bar analyses (all in excess of 6 months). The effect of the presence of cracks is most prominent at early ages and becomes comparatively less noticeable at later times due to the gradual penetration of chloride ions into the cover followed by uniform rusting of the entire bar. If all bars had been analyzed after a relatively short exposure time of, say, three months, then the corrosion differential between stressed and unstressed bars would probably be greater on a proportional scale. The reader is reminded, however, that the observed corrosion at this shorter exposure time is usually small and from a structural standpoint may be of less significance than the corrosion resulting after, say 6 months of exposure for either stressed or unstressed bars.

Even though flexural cracks lose their importance as corrosion promoters after a certain exposure duration, the occurrence and development of longitudinal cracks are of paramount importance. Formation of longitudinal cracks along the top of the reinforcement destroys the protective effect of the concrete cover and leads to severe, continued corrosion of the reinforcement.

In order to determine the relationship between corrosion and the amount of longitudinal splitting thus produced, corrosion versus splitting was plotted in Fig. 3.5.3 for all flexure specimens of this study. Although the data show considerable scatter, several important trends can be seen. First, note that in general severe longitudinal splitting tends to develop at relatively low levels of corrosion. For example, there are several cases of 90 to 100 percent splitting produced by less than 30 percent corrosion. Second, the data indicate that beams with shallow cover and low C/D ratios are most likely to exhibit severe splitting at relatively low levels of corrosion (see solid data points). On the other hand, beams characterized by high C/D ratios tend to be grouped near or above the line of equality in Fig. 3.5.3. This is especially

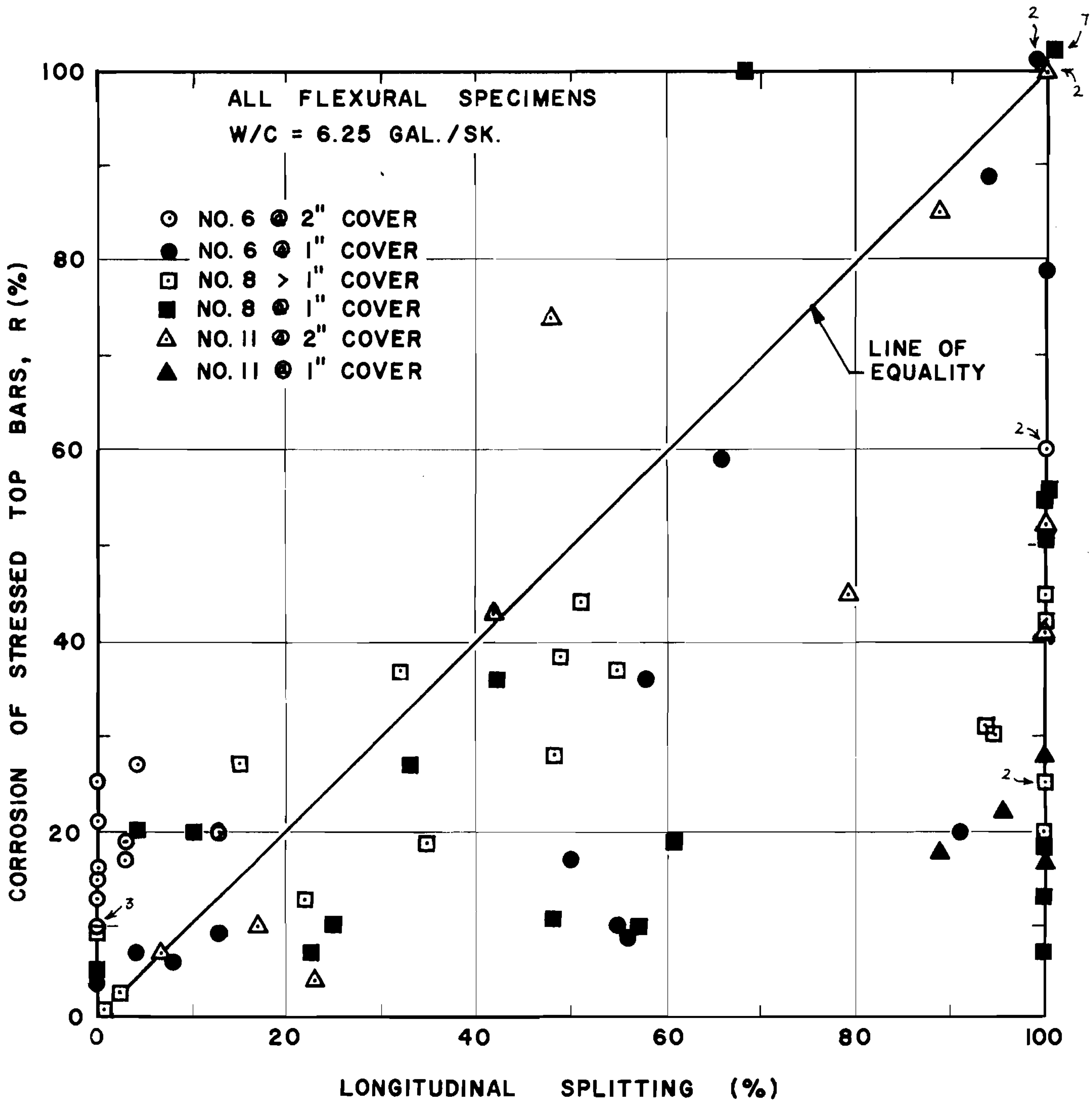


Fig. 3.5.3 Longitudinal splitting and corrosion of stressed bars in all flexural specimens.

noticeable for those beams showing very low values of longitudinal splitting. Therefore, it is apparent that beams with high C/D ratios are most resistant to splitting produced by corrosion of reinforcement.

3.5.2 Steel Stress. As has been pointed out in Section 2.5.8, the level of steel stress apparently has some effect on corrosion deterioration of concrete reinforcement, although the exact nature of the relationship is not definitely known. For this reason, some design engineers are reluctant to use high strength steel, which employs high working stresses.

To study the effect of the level of steel stress in this research program, comparable beam specimens were subjected to sustained steel stresses of 20, 30, and 36 ksi. The unstressed portions of flexural specimens and the unloaded slabs provided data for zero stress levels (excluding volume change effects).

Since stressing is frequently accompanied by flexural cracking in ordinary reinforced concrete structures, the individual effect of stressing or cracking cannot be presented separately. The influence of the existence or absence of stressing and flexural cracking is brought out in Fig. 3.5.2. The majority of data show that the combined effect of stressing and cracking is to increase the degree of corrosion by about 10 to 15 percent.

Figure 3.5.4 shows the effect of the level of steel stress upon corrosion of beams containing No. 6 and No. 8 bars at 2 in. clear cover, W/C ratio of 6.25 gal./sk., 1-1/2 in. crushed limestone concrete, and exposed for a duration of 24 months. Also shown in Fig. 3.5.4 are the variations for the average widths of initial flexural cracks at several levels of stress.

As would be expected, the crack widths for all the beams show similar values at each level of stress, with the larger stresses producing the larger crack widths. However, the indicated relationships for corrosion show considerable deviation for the two bar sizes. For the No. 6 bars of Fig. 3.5.4 a slight increase in corrosion results as the level of steel stress is increased. For example, the average corrosion at 0 and 36 ksi is 10 and 19 percent respectively, with an approximate linear relationship at intermediate points.

With respect to the No. 8 bars of Fig. 3.5.4, two points of difference with the corrosion behavior of the No. 6's is apparent. First, although the crack widths are similar, the No. 8 bars show greater surface corrosion than

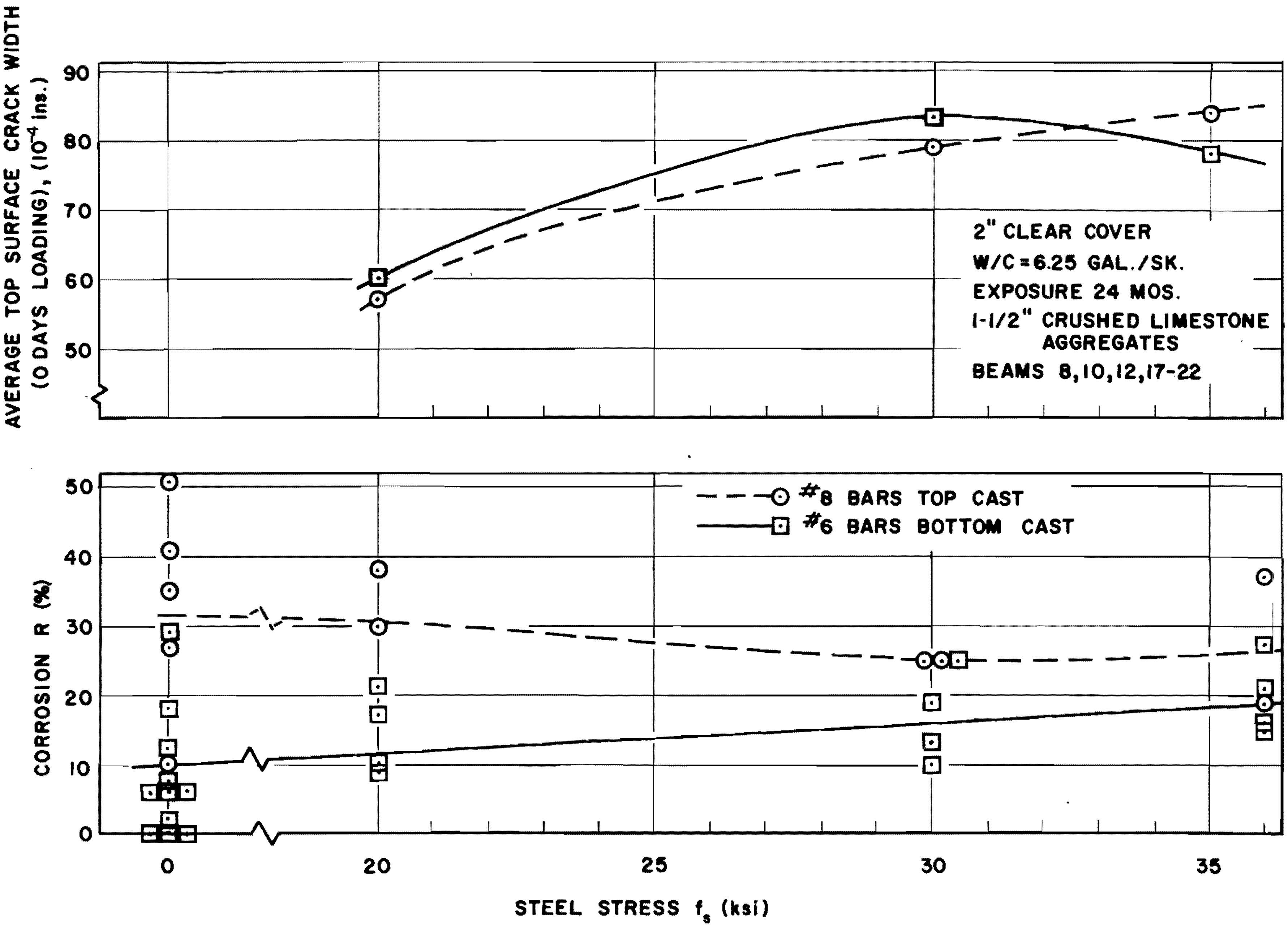


Fig. 3.5.4 Effect of level of steel stress and initial flexural crack width upon corrosion of reinforcement.

the No. 6's at all levels of stress. This, again, points to the significance of the bar size effect, as discussed in Section 3.4.1. As shown in Fig. 3.5.4 the bar size effect tends to diminish at the higher levels of stress where the influence of cracking may predominate. Second, for the No. 8 bars there is no significant corrosion-stress level relationship as the resulting average corrosion is approximately the same at all levels of stress.

When all corrosion data of Fig. 3.5.4 are considered, it is apparent that no significant trend is indicated for the levels of stress and the intermediate grade of steel investigated here. This is perhaps expected for these materials and conditions of loading. That is, the maximum stress level of 36 ksi used here is relatively low in comparison to the yield strength of many of the steels used in various concrete applications. For steels having yield points in excess of 60 ksi and stressed to values considerably above those used here (maximum of 36 ksi), one may find significant relationships between magnitude of stress and long term corrosion. Therefore, additional research is required at the higher stress levels especially since instances of corrosion and stress dependence have been previously noted in Section 2.5.8.

3.5.3 Prestressing. A total of five slab specimens having dimensions of approximately 2 in. x 10 in. x 7 ft. long were cast with 3/8 in. prestress cables. Specific dimensions of the specimens are given in Section 4.3.2 of the Appendix. Concretes of 6.25 and 7.0 gal./sk. and covers of 3/4 in., 1 in., and 1-1/2 in. were used. In all but one slab two cables were employed, the exception having three cables. The prestress loading equivalent to 1270 psi concrete stress was released after initial curing.

In an effort to determine the effect of prestressing upon corrosion, a comparison of the resulting corrosion on prestress cables and No. 6 bars in selected unstressed slabs is given in Fig. 3.4.1 of Section 3.4.1. It should be noted that a difference in the reinforcement diameter exists for this comparison, since the smallest bars used were No. 6 in size. The 6.25 gal./sk. prestressed specimens were compared, since more data were available for this mix. For the unstressed slabs of comparable age and covers, only two specimens of similar water-cement ratio were available for comparison.

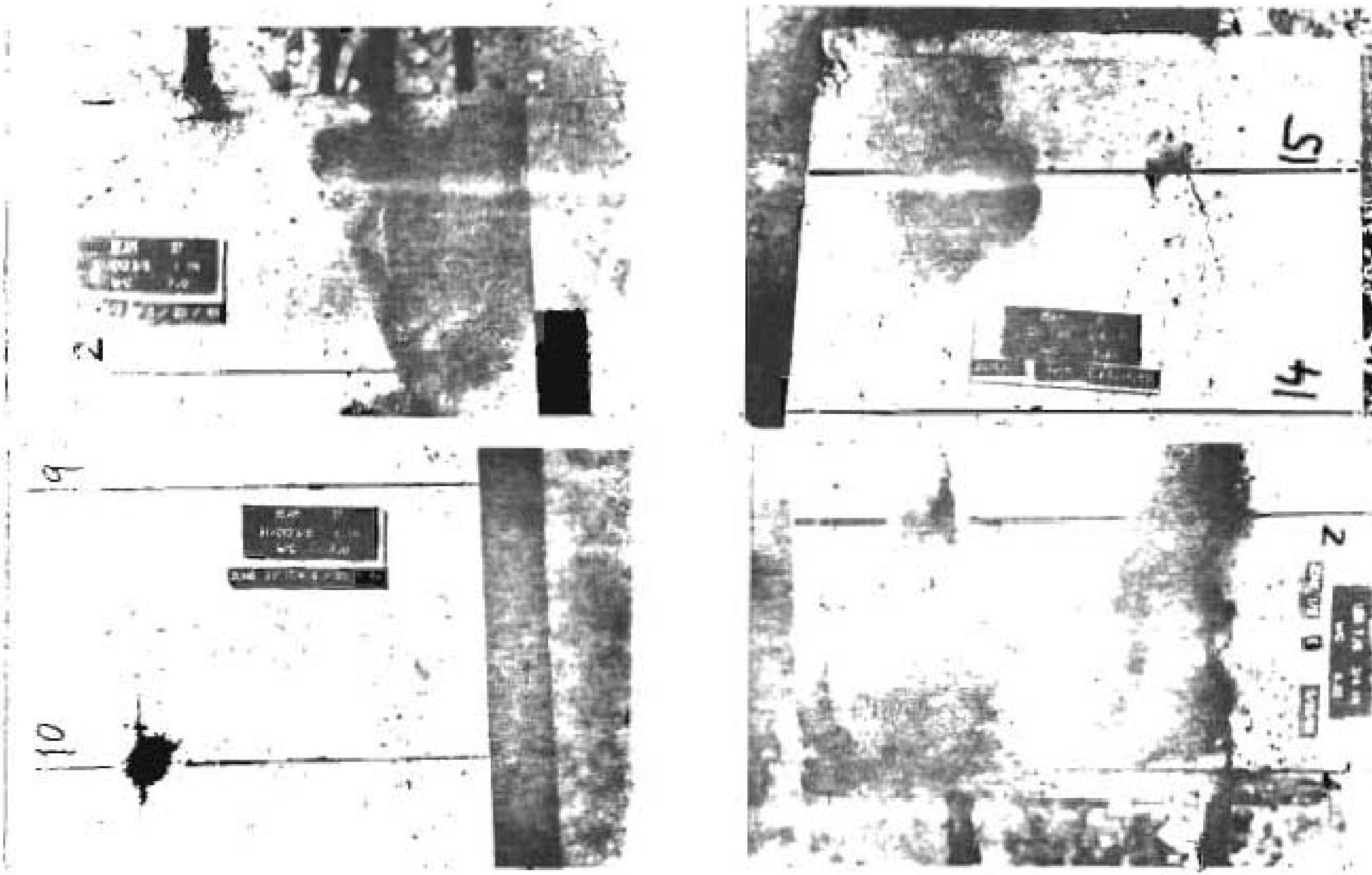
From Fig. 3.4.1 it can be seen that for the 6.25 gal./sk. mixes, the corrosion resulting for both types of reinforcement are similar for covers of

3/4 in. and 1 in. One might speculate that had No. 3 bars been used instead of the No. 6 bars, the resulting corrosion in the No. 3 bars would have been less, due to the bar size effect as previously discussed in Section 3.4.2. If that were the case, the corrosion of the prestressing cables would be generally higher than that of reinforcing bars of the same size. This is perhaps the expected comparison, since prestress cable is usually reported to be more susceptible to corrosion than reinforcing bars.

Also shown in Fig. 3.4.1 is the corrosion-cover relationship for No. 6 bars in 5.5 gal./sk. concrete of 24 month exposure. It can be seen that the general slope of this curve is similar to that of the prestress cable of the 6.25 gal./sk. concrete mix. Thus, the general effects of varying the cover as discussed in Section 3.4.2 are quite probably applicable to corrosion of the prestressing cable. Since no specimens having different cable sizes were studied, no specific projections can be made concerning possible cable size effects similar to those previously discussed for bar size effect.

Two photographs showing the development of surface rust spots and longitudinal splitting in typical prestressed specimens with 3/4 in. and 1 in. cover are given in Fig. 3.5.5. In all prestressed specimens rust spots and longitudinal splitting were first observed at the ends near the exposed, cutoff cables as shown. Although tensile stresses in the concrete can develop near the ends of the specimens as a result of the release of the prestress load at the cable cutoff, there was no evidence of cracking of the specimens until the forces developed by rust formation were of sufficient magnitude to initiate longitudinal splitting. For the 3/4 in. and 1 in. cover specimens, as shown in Fig. 3.5.5, this splitting was seen as early as four months of exposure and extended to approximately 55 and 70 percent respectively of the specimen length at the end of two years of exposure.

3.5.4 <u>Rates of Corrosion</u>. Although not intended as a study parameter, enough data were available in two instances to obtain information concerning the effect of exposure time on corrosion. These data are presented in Fig. 3.5.6, where corrosion values for No. 8 bars in the unloaded portions of beams made of 6.25 gal./sk. concrete and having covers of 1 in. and 2 in. are given. Data for the 1 in. cover specimens range from a maximum exposure of 31 months to a minimum of 6 months, where 12 reinforcing bars averaged



a. Cover = 1 in., W/C = 7.0 gal/sk

b. Cover = 3/4 in., W/C = 6.25 gal/sk

Fig. 3.5.5 Crack development as a result of corrosion of prestress cables in concrete slabs.

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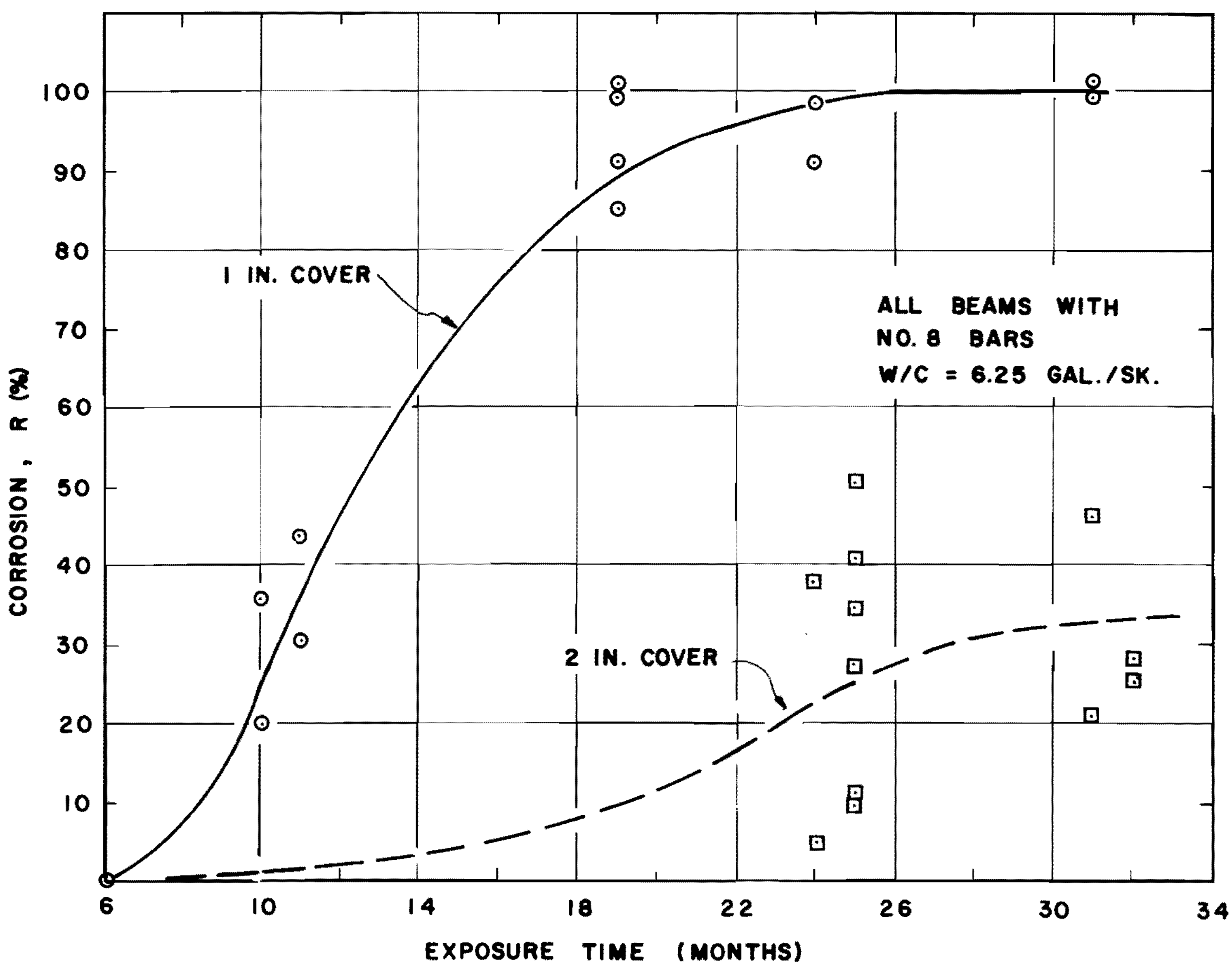


Fig. 3.5.6 Rates of corrosion for uncracked portion of loaded beams made with crushed limestone aggregates.

approximately 1 percent corrosion. No data were available at 6 months exposure for the 2-in. cover specimens, although it is valid to project the curve through the origin in view of the results of the 1-in. cover specimens.

From Fig. 3.5.6 is seen that there is a significant difference in the rates of corrosion for the two cover values shown. If it is assumed that the approximated relationships drawn through the data points are valid, then the maximum corrosion rate of the 1-in. cover specimens is found to be more than four times greater than that of the 2-in. specimens.

The curves of Fig. 3.5.6 also give some indication of the self-inhibition of corrosion for the different covers. For the 1-in. cover it is seen that the buildup of a rust layer around the reinforcing bar was not effective in inhibiting further corrosion. A lack of self-inhibition of corrosion with this shallow cover is expected, since relatively small stresses resulting from the rust buildup are sufficient to split the cover longitudinally. Once the cover is cracked at the bar, the concrete cover and rust layer are more readily penetrated to promote additional corrosion and splitting. For the 2-in. cover specimens, it would be expected that self-inhibition of the corrosion would be more effective since the stresses required to split the concrete are higher. That is, the rust layers on the bars would probably be more dense and thus contribute to inhibiting continued rusting. Although the data are somewhat scattered for the 2-in. cover specimens of Fig. 3.5.6, there is a general indication of a reduction in the rate of corrosion beyond 25 months exposure. However, additional data beyond 36 months would be required to confirm a decrease in corrosion rate for the 2-in. cover specimens.

APPENDIX

4.1 Electrochemical Nature of Corrosion of Steel in Concrete

Corrosion can be defined as the deterioration or destruction of a material by reaction with its environment. It is an electrochemical process, an example of which is a galvanic cell. For an electrochemical cell to function, three basic elements are necessary: anode, cathode, and electrolyte. An anode is an electron producing unit, while the cathode is the electron consuming unit. The electrolyte is a medium through which ionic flow can occur.

Typical reactions at the anode and cathode for the corrosion of iron are:

$$\text{Anodic reaction} \quad Fe \rightarrow Fe^{++} + 2e^{-} \text{ (oxidation)} \tag{4.1}$$

$$\text{Cathodic reaction} \quad 2H^{+} + 2e^{-} \rightarrow H_2 \text{ (reduction)} \tag{4.2}$$

$$\text{Depolarization reaction} \quad 2H^{+} + 2e + 1/2\ O_2 \rightarrow H_2O \tag{4.3}$$

At the anode, metallic iron is oxidized and electrons are generated (Eq. 4.1). Since the metal must remain at a state of electron equilibrium, an equal amount of electrons are consumed at the cathode to form hydrogen gas (Eq. 4.2). The hydrogen gas tends to remain near the bar surface and the reaction becomes self-inhibiting, as shown in Fig. 4.1.1. The cathode is then said to be polarized and no further reaction is possible unless the protective hydrogen film is removed (depolarized).

Hydrogen may be evolved as a gas, but this process is normally quite slow. More important is the breakdown of the hydrogen film by the depolarizing action of oxygen. In this case, oxygen acts to prevent the buildup of hydrogen gas by consuming the free electrons, as indicated by Eq. 4.3 (see Fig. 4.1.2). Once the hydrogen layer is broken, the corrosion reactions are free to continue.

Since sodium and chloride ions do not participate in the reaction, the total reaction can be expressed as the sum of Eq. 4.1 and Eq. 4.3. Making use of the reaction $H_2O \rightarrow H^{+} + OH^{-}$ gives the principal corrosion reaction,

$$Fe + H_2O + 1/2\ O_2 \rightarrow Fe(OH)_2 \tag{4.4}$$

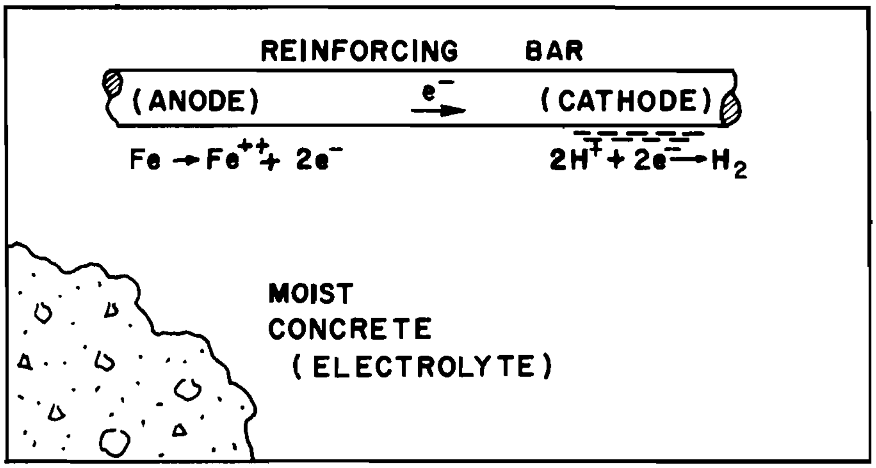


Fig. 4.1.1 Polarization of the cathode by film of hydrogen gas

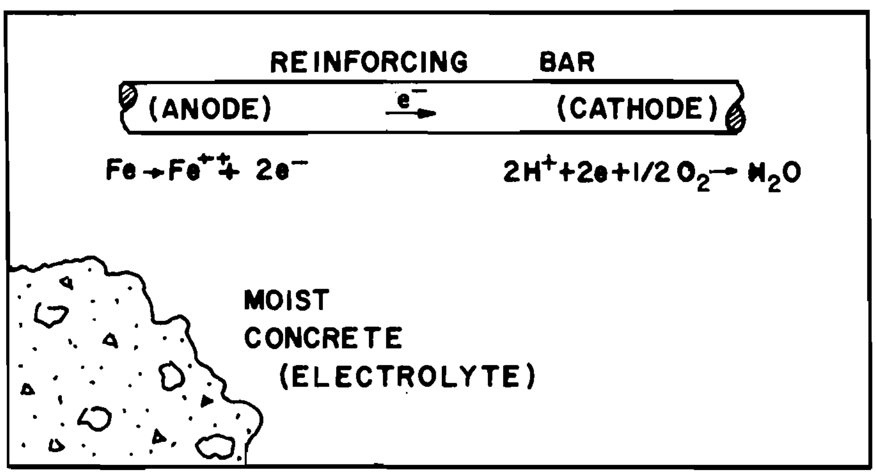


Fig. 4.1.2 Depolarization by the action of oxygen.

The compound precipitating is ferrous hydroxide, a form of rust with whitish color. However, in oxgenated solutions, ferrous hydroxide is further oxidized to ferric hydroxide.

$$Fe(OH)_2 + 1/2\ H_2O + 1/4\ O_2 \rightarrow Fe(OH)_3 \qquad (4.5)$$

The product finally formed is the familiar reddish-brown rust.

In a medium of perfect uniformity, corrosion is very unlikely to occur. However, reinforced concrete is by no means a homogeneous material and corrosion cells are set up when certain conditions exist. There are numerous reasons for the corrosion enhancing nonuniformity of concrete. Concrete may be honeycombed, porous, and unevenly wet and dry. Cracking causes differences in steel stress, differential aeration, and depositions of salt. There are always inherent nonuniformities in the steel itself due to initial locked-in residual stresses and the manufacturing processes. As a result, regions of lower potential become anodic and regions of higher potential become cathodic. Moist concrete acts as the electrolyte, the action of which is further accelerated if salt ions exist.

Corrosion in reinforced concrete can be studied under two general groups: cracked and uncracked concrete. The case of uncracked concrete will be discussed first.

Fresh uncracked concrete normally has ample resistance to corrosive attack. The concrete cover over the bar is very effective in inhibiting the penetration of corrosive agents to the level of the steel. It is obvious that the thicker and denser the concrete cover, the more effective it becomes in resisting corrosion. Also, fresh concrete has a very high pH value which usually inhibits corrosion reactions. The pH number is an index of the acidity or alkalinity of a medium. Numbers from 0 to 7 indicate acidity of a solution (in which corrosion is promoted), and numbers from 7 to 14 indicate solution alkalinity (in which corrosion is retarded). Fresh concrete has a high $Ca(OH)_2$ content which gives it a pH of about 13.

The last defense against corrosion is offered by the oxide film (mill scale) around the bar surface. This oxide film prevents corrosive agents from coming into direct contact with the bare metal. Thus mill scale provides localized corrosion protection.

However, as time passes the above conditions tend to alter. Water, salt, oxygen, carbon dioxide, and industrial gases (if present) slowly begin penetrating the concrete, the rate of which depends on the permeability of the concrete cover. Carbon dioxide, which penetrates into concrete through pores and cracks, reacts with calcium hydroxide and produces calcium carbonate (Eq. 4.6):

$$[\{CO_2 + H_2O\} \rightarrow H_2CO_3] + Ca(OH)_2 \rightarrow CaCO_3 + 2H_2O \qquad (4.6)$$

Thus, the pH value and consequently the protective quality of concrete are reduced. The general mechanism by which corrosion occurs in concrete is indicated in the drawing of Fig 4.1.3.

When the pH of concrete falls as low as 8, the probability of corrosion is high. Crystallizing salt and freeze-thaw effects set up internal forces that adversely affect the durability of the concrete cover. As a corrosive medium reaches the steel, it concentrates its attack at the flaws in the oxide film. More importantly, if salt is present, it will destroy the passivity of the oxide film on the steel and corrosion is thus promoted.

It is obvious that at large cracks in the concrete, the penetration phase of the above sequence will be considerably shorter and corrosion will rapidly begin on the steel below the cracks. In uncracked regions of the concrete the same sequence will take place as outlined, except at a much reduced rate. That is, corrosion initiates as soon as the corrosion promoting medium penetrates through the concrete to the level of the steel.

It should be kept in mind that the presence of salt is an important factor in the corrosion process. Salt ions destroy the passivity of steel, set up corrosion cells, and increase the conductivity of the electrolyte. Without salt ions, corrosion of steel in concrete may be inhibited for a long period of time. In that case, the corrosion rate is generally controlled by the processes of carbonation. If the concrete cover is relatively impermeable and thick, corrosion may not occur at all in uncracked areas. However, cracks do not lose their importance in this case because localized corrosion can occur under them.

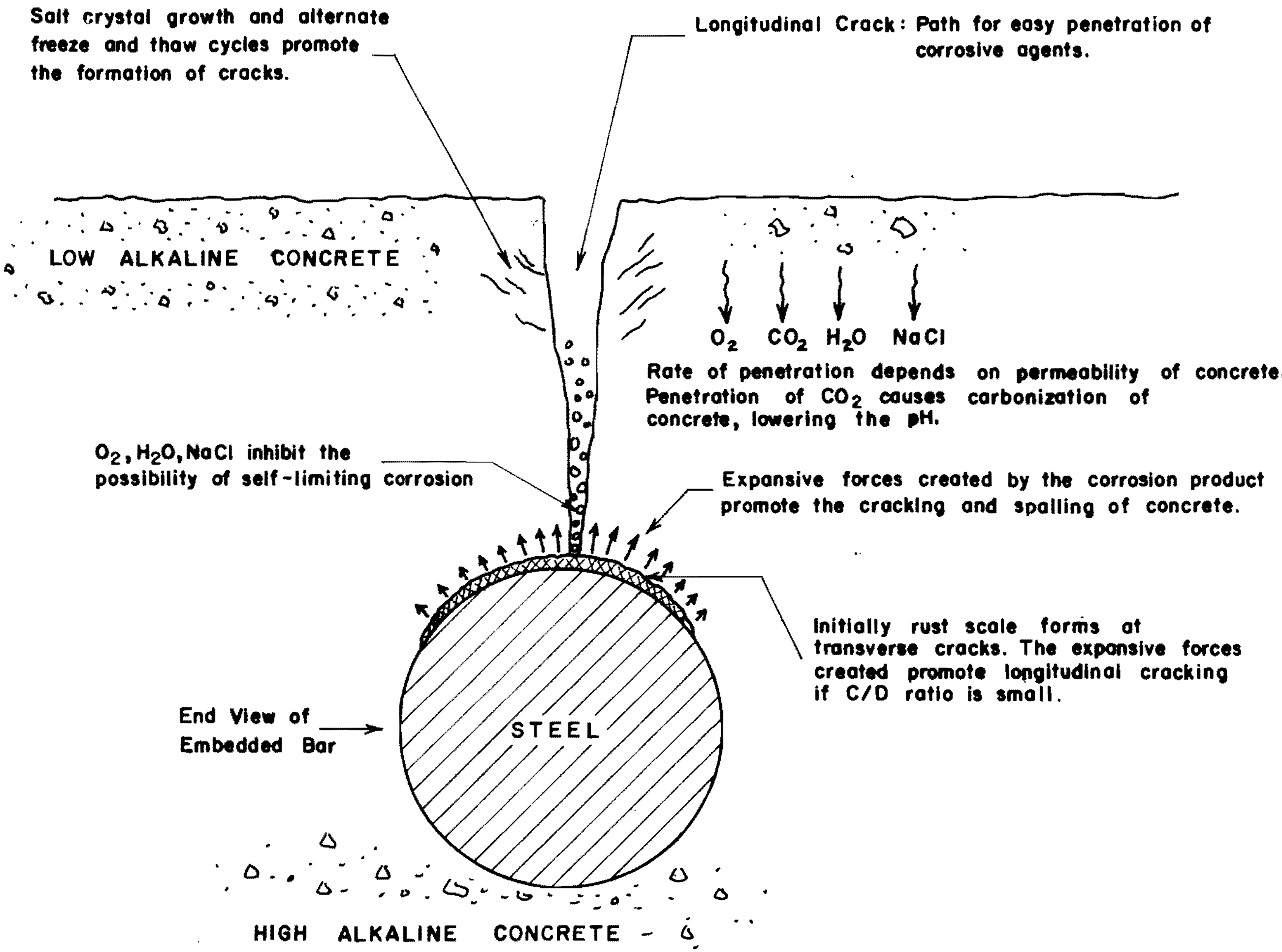


Fig. 4.1.3. General mechanism for the corrosion of reinforcing steel in concrete.

The corrosion products formed tend to have an inhibiting effect upon continued corrosive reactions. These products may seal off the base metal from oxygen and hydrogen diffusion and thus terminate the corrosion reactions. This process is known as self-limiting corrosion. The authors feel that a self-limiting corrosion can take place at high C/D ratios which seem to determine the occurrence and extent of longitudinal splitting along the bars. The longitudinal splitting is mainly due to the tensile forces created by the corrosion products which occupy about three times greater volume than the metal from which they are formed. If the concrete cover is not sufficient to resist such forces, longitudinal cracks develop through which oxygen and other external agents gain access to the steel. At this point, it is only a matter of time until the structure reaches a hazardous state of corrosion and must be repaired or replaced. Repeated loadings may also play a role in breaking the protective effect of the rust scale, but more research is needed to establish its importance.

4.2 Concrete Data

4.2.1 Materials

The normal weight concrete was made from two types of coarse aggregates in conjunction with Colorado River fine aggregate, crushed limestone and Colorado River coarse aggregate. A 3/4 in. expanded shale from Featherlite's Ranger plant was used to make the lightweight concrete. The physical properties of both aggregates are shown in Table 4.2.1.

In all specimens, Colorado River fine aggregate was used in conjunction with the coarse aggregates given in Table 4.2.1.

Type III cement (Alamo) was used in all types of concrete, except specimens Nos. 66, 67, and 68 which were made with Type V cement (El Toro). Ordinary tap water was used to make the concrete. Locally available ASTM 305 intermediate grade steel was used for both the longitudinal and transverse reinforcement.

4.2.2 Mix Design

Since concrete quality was one of the research variables, an effort was made to hold slump, air content, and concrete strength within practical ranges. Trial batches were used in order to determine the proper proportions of mix

TABLE 4.2.1

PHYSICAL PROPERTIES OF AGGREGATES

WEIGHT	TYPE	MAX. SIZE (in.)	SPECIFIC GRAVITY	STEVE ANALYSIS CUMULATIVE PERCENTAGE RETAINED ON	24 HOURS ABSORPTION (%)	SPECIMENS MADE
Normal	Crushed Limestone	1 1/2	---	----	---	1-26
		3/4	2.50	1" 3/4" 1/2" 3/8" #4 #8 08 11.9 65.8 87.3 96.8 99.1	2.90	69-76
		1/2	2.52	3/4" 1/2" 3/8" #4 #8 0 17.5 54.1 96.5 98.7	2.40	48-68
	Colorado River Gravel	3/8	---	----	---	27-41
	Fine Aggregate	---	2.60	#4 #8 #16 #30 #50 #100 0.5 8.2 27.2 69.6 92.8 97.9	1.00	-----
Light-weight	Ranger (Feather-lite)	3/4	---	----	4.20	42L-47L,14BL, 15BL,2BL,9BL, 23BL,25BL

ingredients for the required quality of concrete. To obtain the desired air content, Septair CRD-C-13, AASHO-M was used for specimen 1-47L, and Airsene 311-CRD-C-87, AASHO-M-194 was used for specimens 48-76. Table 4.2.2 shows the concrete mix properties for specimens 1-47L and Table 4.2.3 for specimens 48-76.

The design of the lightweight concrete mix for the desired W/C ratio was particularly difficult, due to the gradual water absorption of the lightweight aggregate from the mixing water. This problem was solved by establishing a time-absorption curve for oven-dry lightweight aggregate. Then, at the time of mixing, a stockpile sample was dried to determine the moisture content as a percent of the oven-dry weight. By assuming that the aggregate would continue to absorb water in the fresh concrete, the absorption curve could be used to compute a moisture correction for the assumed saturated surface dry state. The additional absorption time used in this study was 24 hours although it is apparent that a period of 2 to 6 hours would be more suitable. Thus, it was possible to compute a value for the water-cement ratio of the lightweight concrete.

4.2.3 Control Tests

The control cylinder tests are tabulated in Table 4.2.4 together with their curing conditions. For specimens 1 - 47, the cylinders were tested at the same day salt spraying was started. In order to get an idea of the effect of concrete strength on corrosion, it was decided to obtain a calculated strength at a given age. Therefore, all cylinder strengths were reduced to 28-day strengths by the use of Erzen's formula

$$f'_{c(28)} = \frac{f'_{c(t)}}{\left(\frac{12}{11 + \frac{28}{t}}\right)^2}$$

where $f'_{c(t)}$ is the strength of concrete at t days.[46]

4.3 Corrosion Specimens

4.3.1 Beams

The cross-sectional dimensions of the beams varied as well as the bar sizes. They can be considered under three main groups, as shown in Table 4.3.1.

(text continued p. 104)

TABLE 4.2.2

CONCRETE MIX PROPERTIES

SPECIMEN NO.	MIX. PROPORTIONS BY WEIGHT* (Water:cement:sand:gravel)	SLUMP (in.)	AIR (%)	CEMENT FACTOR (Sacks/or yd.)
1	0.54: 1 :3.14: 4.08	5	---	5.0
2	0.60: 1 :3.14: 4.08	5	---	5.0
2B	0.54: 1 :3.14: 4.08	4 1/2	4.6	---
2BL	0.55: 1 :2.32: 2.38	4 1/2	5.0	5.5
3	0.60 1 :3.65: 4.06	3	6.0	5.0
4	0.60: 1 :3.65: 4.06	3	6.0	5.0
5A	0.51: 1 :3.15: 4.08	3 1/2	5.5	5.0
6	0.49: 1 :3.15: 4.08	3 1/2	7.0	5.0
7A	0.52: 1 :3.15: 4.08	4 1/2	4.5	5.0
8	0.49: 1 :3.15: 4.08	3	6.25	5.0
9	0.53: 1 :3.15: 4.08	2 3/4	4.25	5.0
9BL	0.55: 1 :2.32: 2.38	4 1/2	5.0	5.5
10	0.53: 1 :3.15: 4.08	2 3/4	4.25	5.0
11	0.52: 1 :3.15: 4.08	3	5.0	5.0
12	0.52: 1 :3.15: 4.08	3	5.0	5.0
13	0.49: 1 :3.15: 4.08	3	6.0	5.0
14	0.49: 1 :3.15: 4.08	3	6.5	5.0
14B	0.49: 1 :3.15: 4.08	3	7.0	---
14BL	0.55: 1 :2.32: 2.38	4 1/2	4.5	5.5
15	0.49: 1 :3.15: 4.08	3	5.0	5.0
15B	0.49: 1 :3.15: 4.08	3	7.0	---

* Actual batch weights not adjusted to SSD condition by moisture corrections.

TABLE 4.2.2 cont'd.

SPECIMEN NO.	MIX. PROPORTIONS BY WEIGHT (Water:cement:sand:gravel)	SLUMP (in.)	AIR (%)	CEMENT FACTOR (Sacks/or yd.)
15BL	0.55: 1 :2.32: 2.38	4 1/2	4.5	5.5
16	0.49: 1 :3.15: 4.08	3	6.0	5.0
17	0.49: 1 :3.15: 4.08	3 1/2	6.0	5.0
18	0.49: 1 :3.15: 4.08	3 1/2	6.0	5.0
19	0.49: 1 :3.15: 4.08	3	5.0	5.0
20	0.49: 1 :3.15: 4.08	3	5.0	5.0
21	0.49: 1 :3.15: 4.08	3 1/4	6.5	5.0
22	0.49: 1 :3.15: 4.08	3 1/4	6.5	5.0
23	0.53: 1 :3.15: 4.08	3	6.0	5.0
23BL	0.55: 1 :2.32: 2.38	3 1/4	5.6	5.5
24	0.53: 1 :3.15: 4.08	3	6.0	5.0
25	0.49: 1 :3.15: 4.08	3 1/2	5.25	5.0
25BL	0.55: 1 :2.32: 2.38	3 1/4	5.6	5.5
26	0.49: 1 :3.15: 4.08	3 1/2	5.25	5.0
27	0.53: 1 :3.14: 2.79	4	7.0	6.0
28	0.53: 1 :3.14: 2.79	4	7.0	6.0
29	0.53: 1 :3.14: 2.79	4	7.0	6.0
30	0.53: 1 :3.14: 2.79	4	7.0	6.0
31	0.43: 1 :1.70: 2.06	3	6.0	10.0
32	0.43: 1 :1.70: 2.06	3	6.0	10.0
33	0.43: 1 :1.70: 2.06	3	4.0	10.0
34	0.43: 1 :1.70: 2.06	3	4.0	10.0
35	0.49: 1 :2.42: 2.50	4 1/2	4.5	8.0

TABLE 4.2.2 cont'd.

SPECIMEN NO.	MIX. PROPORTIONS BY WEIGHT (Water:cement:sand:gravel)	SLUMP (in.)	AIR (%)	CEMENT FACTOR (Sacks/or yd.)
36	0.49: 1 :2.42 :2.50	4 1/2	4.5	8.0
37	0.56: 1 :3.12 :2.78	2	2.0	---
38	0.56: 1 :3.12 :2.78	2	2.0	---
39	0.49: 1 :2.38 :2.46	2	6.0	---
40	0.49: 1 :2.37 :2.45	3	4.0	---
41	0.49: 1 :2.37 :2.45	3	4.0	---
42L	0.62: 1 :2.63 :1.9	4	---	5.5
43L	0.62: 1 :2.63 :1.9	4	---	5.5
44L	0.55: 1 :2.32 :2.38	4 1/2	5.0	5.5
45L	0.55: 1 :2.32 :2.38	4 1/2	5.0	5.5
46L	0.49: 1 :1.90 :1.47	5	5.0	7.0
47L	0.49: 1 :1.90 :1.47	5	5.0	7.0

TABLE 4.2.3
CONCRETE MIX PROPORTIONS*

CONCRETE PARAMETER	SPECIMENS 48-52	53	54-59	60-65	66	67	68	69-70	71-72	73-74	75-76
Percent Cement	12.7	12.6	13.7	11.2	13.9	9.8	11.0	10.2	10.0	10.0	9.9
Percent Water	19.6	19.6	18.3	19.5	18.3	19.2	19.3	17.7	17.4	17.4	17.2
Percent Fine Aggregate	34.7	34.7	33.1	34.9	33.0	35.5	34.9	32.6	33.1	33.7	33.4
Percent Course Aggregate	30.0	30.2	30.7	30.1	30.5	30.5	30.0	34.5	34.5	34.8	35.0
Percent Air	3.0	3.0	4.1	4.3	4.3	5.0	4.8	5.5	5.1	4.1	4.5
C.F. (sk/cy)	7.2	7.1	7.8	6.3	7.8	6.5	6.2	5.8	5.6	5.6	5.6
Slump (in.)	2.3	2.3	3.1	6.4	2.8	6.3	4.5	7.0	6.0	6.3	5.5
W/C (gal/sack)	5.5	4.75	4.75	6.25	4.25	5.5	6.25	6.25	6.25	6.25	6.25

* Percent volume proportions based on SSD conditions. Batch weights adjusted by aggregate moisture determinations.

TABLE 4.2.4

COMPRESSIVE STRENGTHS OF CONCRETE MIXES

SPECIMEN NO.	CURING CONDITIONS	CYLINDER TEST AGE (Days)	f_c^* (Psi)	f'_c (28 days) (Psi)
1	3 days moist, then dry until tested	88	3850	3430*
2	" "	88	3730	3320*
3	" "	130	5250	4590*
4	" "	81	5790	5180*
5A	" "	96	5030	4460*
6	" "	86	4370	3900*
7A	" "	47	5780	5400*
8	" "	72	4080	3670*
9	" "	103	5090	4490*
10	" "	72	5080	4580*
11	" "	70	5620	5070*
12	" "	79	6260	5600*
13	" "	79	5610	5020*
14	" "	69	5620	5080*
15	" "	74	5850	5260*
16	" "	74	5790	5200*
17	" "	83	4700	4200*
18	" "	83	4780	4200*
19	" "	77	4450	3990*
20	" "	72	4330	3900*
21	" "	73	4250	3820*
22	" "	73	4150	3740*
23	" "	70	5670	5120*

*Calculated by Erzen's formula; all others were 28-day tests.

TABLE 4.2.4 cont'd.

SPECIMEN NO.	CURING CONDITIONS	CYLINDER TEST AGE (Days)	f'_c (Psi)	f'_c (28 days) (Psi)
24	3 days moist, then dry until tested	71	5530	4990*
25	" "	63	4370	3970*
26	" "	63	4350	3950*
2LB	" "	7	4560	5190*
9LB	" "	7	4560	5190*
14LB	" "	7	4330	4460*
15LB	" "	7	4330	4460*
23LB	" "	7	4550	4670*
25LB	" "	7	4550	4670*
2B	" "	175	4650	4020*
14B	" "	153	4760	4460*
15B	" "	153	4760	4460*
27	" "	43	3760	3550*
28	" "	43	3760	3550*
29	" "	43	3760	3550*
30	" "	43	3760	3550*
31	" "	37	5570	5350*
32	" "	37	5570	5350*
33	" "	37	5570	5350*
34	" "	37	5570	5350
35	" "	33	3770	3680*
36	" "	33	3770	3680*

*Calculated by Erzen's formula; all others were 28-day tests.

TABLE 4.2.4 cont'd.

SPECIMEN NO.	CURING CONDITIONS	CYLINDER TEST AGE (Days)	f'_c (Psi)	f'_c (28 days) (Psi)
37	3 days moist, then dry until tested		3960	
38	" "		3960	
39	" "	41	4430	4200*
40	" "	35	4180	4050*
41	" "	35	4180	4050*
42L	" "	7	3660	5720*
43L	" "	7	3660	5720*
44L	" "	7	4560	5190*
45L	" "	7	4560	5190*
46L	" "	7	4850	7580*
47L	" "	7	4850	7580*
48	7 days moist, then dry	21	6530	
	All moist	28		6920*
49	7 days moist, then dry	21	6530	
	All moist	28		6920
50	7 days moist, then dry	21	6530	
	All moist	28		6920
51	7 days moist, then dry	21	6530	
	All moist	28		6920
52	7 days moist, then dry	21	6530	
	All moist	28		6220

*Calculated by Erzen's formula; all others were 28-day tests.

TABLE 4.2.4 cont'd.

SPECIMEN NO.	CURING CONDITIONS	CYLINDER TEST AGE (days)	f'_c (Psi)	$f'_{c(28\ days)}$ (Psi)
53	7 days moist, then dry	21	4580	
	All moist	28		6080
54	7 days moist, then dry			
	All moist	28		6300
55	7 days moist, then dry	21	6150	
	All moist	28		6300
56	7 days moist, then dry	21	6150	
	All moist	28		6300
57	7 days moist, then dry	21	6150	
	All moist	28		6300
58	7 days moist, then dry	21	6150	
	All moist	28		6300
59	7 days moist, then dry	21	6150	
	All moist	28		6300
60	7 days moist, then dry	21	4830	
	All moist	28		4540
61	7 days moist, then dry	21	3830	
	All moist	28		4540
62	7 days moist, then dry	21	3830	
	All moist	28		4540
63	7 days moist, then dry	21	3830	
	All moist	28		4540
64	7 days moist, then dry	21	4830	
	All moist	28		4540

TABLE 4.2.4 cont'd.

SPECIMEN NO.	CURING CONDITIONS	CYLINDER TEST AGE (Days)	f'_c (Psi)	f'_c(28 days) (Psi)
65	7 days moist, then dry	21	4830	
	All moist	28		4540
66	7 days moist, then dry	21	4530	
	All moist	28		5330
67	7 days moist, then dry	21	3160	
	All moist	28		4150
68	7 days moist, then dry	21	3860	
	All moist	28		4500
69	7 days moist, then dry	22	4140	
	All moist	28		4090
70	7 days moist, then dry	22	4140	
	All moist	28		4090
71	7 days moist, then dry	21	4211	
	All moist	28		4490
72	7 days moist, then dry	21	4200	
	All moist	28		4490
73	7 days moist, then dry	21	4250	
	All moist	28		3830
74	7 days moist, then dry	21	4250	
	All moist	28		3830
75	7 days moist, then dry	21	4710	
	All moist	28		4700
76	7 days moist, then dry	21	4710	
	All moist	28		4700

TABLE 4.3.1

PHYSICAL CHARACTERISTICS OF SPECIMEN TYPES

GROUP	DIMENSIONS bxh (in.)	CLEAR COVER (in.)	MAIN BAR SIZE	STIRRUP BAR SIZE	COMP. BAR SIZE	SPECIMEN DESIGNATION
I	12x24	2	#11	#3	#6	1,2,3,4,2B,2BL
	12x23	1				69,70
	11x16	3				5A,6
II	11x15	2	#8	#2	#6	7A,8,9,10,11,12
	11x14	1				13,14,15,16, 71*,72*,73,74
	12x8 1/4	2				17,18,19,20,21,22
III	12x7 1/4	1	#6	#2	#6	23,24,25,26 ,75,76
	11x14	1				

*Beams 71 and 72 have 4 #8 bars.

Figure 4.3.1 shows a sketch of a typical beam and Fig. 4.3.2 shows a photograph of typical beams under load.

The flexural specimens were loaded so as to subject the unstirruped middle portion of the beams to a constant moment, tension always being at the top. At both ends, outside the load points, there were cantilever portions, one reinforced and the other plain concrete (see Fig. 4.3.1).

The beams were loaded to sustain different levels of steel stress. During the initial loading, an overstress was applied for about 10 minutes before the desired sustained stress to induce the effects of a possible overload on ordinary bridges. Table 4.3.2 shows the physical makeup of all the specimens. The variation in the casting position is indicated by bottom cast (B), or top cast (T). Bottom cast means the tension steel was cast at the bottom of the form and top cast means tension bars at the top of the form when concrete was poured.

4.3.2 Slabs

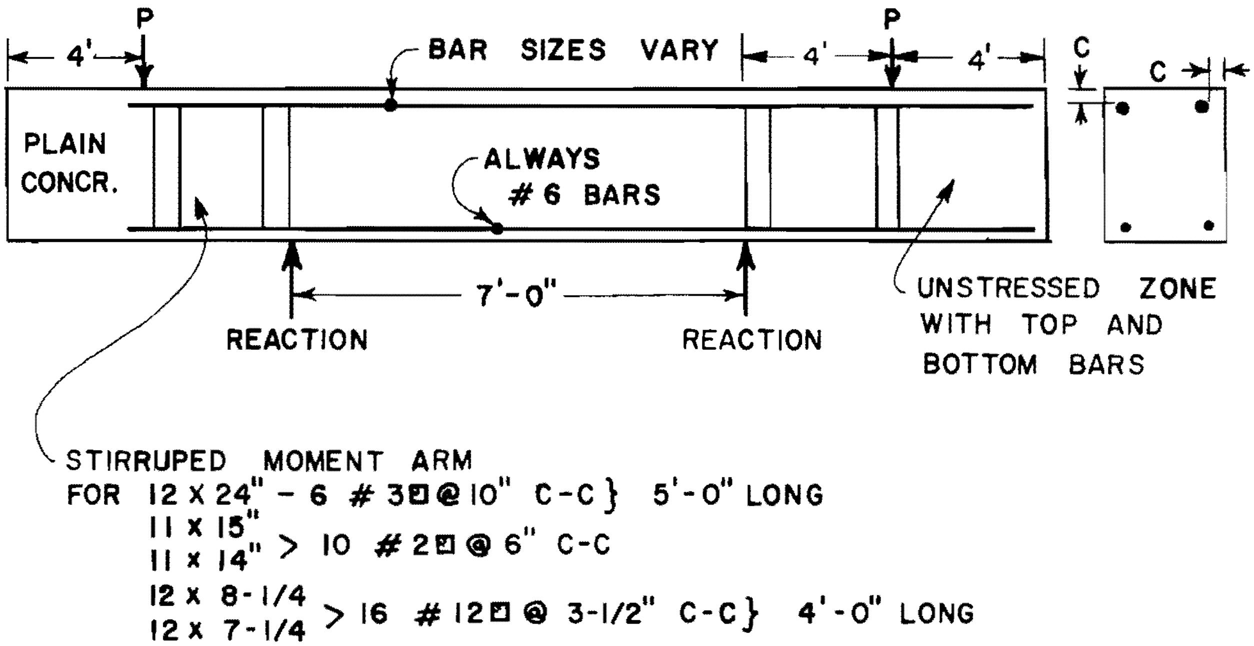
All the reinforced concrete slabs were 7'0" long and had 6 x 12 in. cross sections, but the dimensions of prestressed slabs varied according to their clear cover. All the prestressed slabs were subjected to an average concrete prestress of 1.27 ksi (see Fig. 4.3.3a and b).

During corrosion spraying the slabs were simply placed on concrete pedestals with their main bars on top. Table 4.3.2 also includes all the physical data of the unloaded slabs.

4.4 Salt Spraying

Salt containing 99 percent of NaCl was dissolved in ordinary tap water to make a 3 percent solution by weight to simulate sea water concentration. All the specimens were thoroughly sprayed once every day (weekends included) except on those days when spraying was prevented by rainfall. The spraying procedure consisted of two passes, one after another each time uniformly wetting all the surfaces of the specimens from one end of the exposure site to the other. Figure 4.4.1a-b shows the arrangement of the exposure site and all specimen properties.

(text continued p. 104)



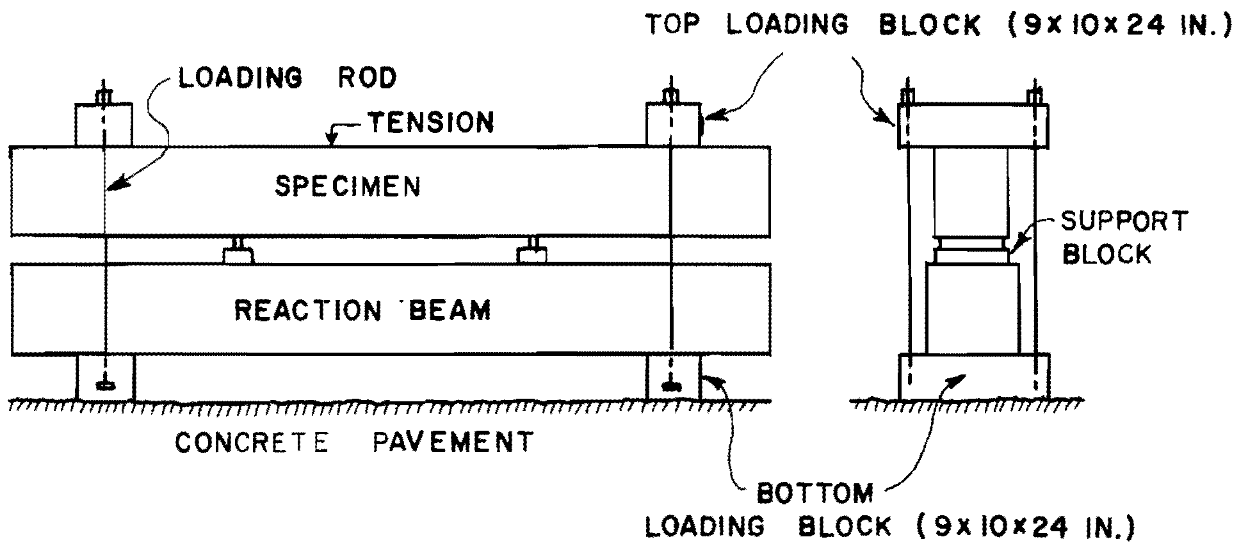


Fig. 4.3.1. Reinforcing bar and loading details for beam specimens.

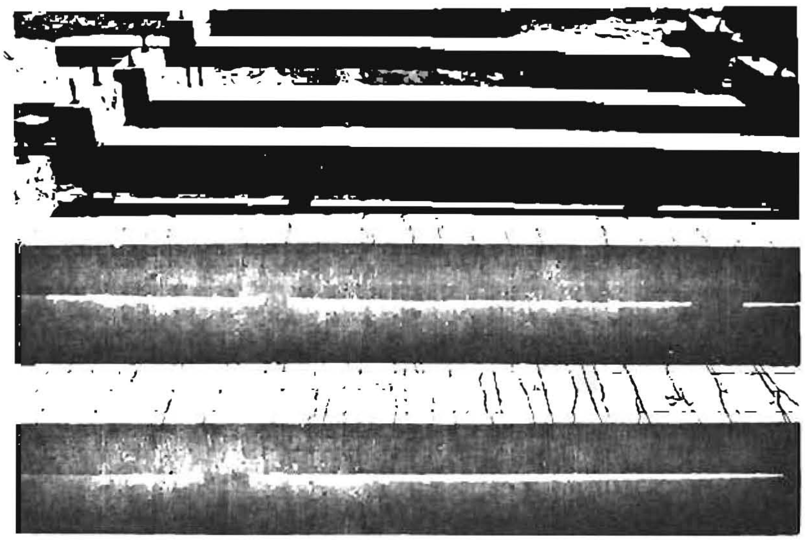


Fig. 4.3.2 Flexural specimens under load.

TABLE 4.3.2 PROPERTIES OF INDIVIDUAL SPECIMENS

SPECIMEN NO.	DIMENSIONS b x h (in.^{2})	CLEAR COVER (in.)	REINFORCEMENT	STEEL STRESS (ksi)	W/C (gal./sack)	POSITION OF CASTING
1	12 x 24	2	2#11 (2#6)	30(30)	6.25	B
2	12 x 24	2	2#11 (2#6)	30(30)	6.25	T
2B	12 x 24	2	2#11 (2#6)	30(30)	6.25	T
2BL	12 X 14	2	2#11 (2#6)	20(24)	6.25	T
3	12 x 24	2	2#11 (2#6)	30(30)	6.25	T
4	12 x 24	2	2#11 (2#6)	30(30)	6.25	B
5A	11 x 16	3	2#8 (2#6)	30(37.5)	6.25	B
6	11 x 16	3	2#8 (2#6)	20(25)	6.25	B
7A	11 x 15	2	2#8 (2#6)	20(25)	6.25	B
8	11 x 15	2	2#8 (2#6)	20(25)	6.25	T
9	11 x 15	2	2#8 (2#6)	30(37.5)	6.25	B
9BL	11 x 15	2	2#8 (2#6)	30(37.5)	6.25	B
10	11 x 15	2	2#8 (2#6)	30(37.5)	6.25	T
11	11 x 15	2	2#8 (2#6)	36(40)	6.25	B
12	11 x 15	2	2#8 (2#6)	36(40)	6.25	T
13	11 x 14	1	2#8 (2#6)	20(25)	6.25	B

TABLE 4.3.2 cont'd.

SPECIMEN NO.	DIMENSIONS b X h(in.^{2})	CLEAR COVER	REINFORCEMENT	STEEL STRESS (ksi)	W/C (gal./sack)	POSITION OF CASTING
14	11 x 14	1	2#8 (2#6)	20(25)	6.25	B
14B	11 x 14	1	2#8 (2#6)	20(25)	6.25	B
14BL	11 x 14	1	2#8 (2#6)	20(25)	6.25	B
15	11 x 14	1	2#8 (2#6)	30(37.5)	6.25	B
15B	11 x 14	1	2#8 (2#6)	30(37.5)	6.25	B
15BL	11 x 14	1	2#8 (2#6)	30(37.5)	6.25	B
16	11 x 14	1	2#8 (2#6)	30(37.5)	6.25	B
17	12 x 8 1/4	2	2#6 (2#6)	36(40)	6.25	B
18	12 x 8 1/4	2	2#6 (2#6)	36(40)	6.25	B
19	12 x 8 1/4	2	2#6 (2#6)	30(37.5)	6.25	B
20	12 x 8 1/4	2	2#6 (2#6)	30(37.5)	6.25	B
21	12 x 8 1/4	2	2#6 (2#6)	20(25)	6.25	B
22	12 x 8 1/4	2	2#6 (2#6)	20(25)	6.25	B
23	12 x 7 1/4	1	2#6 (2#6)	30(37.5)	6.25	B
23BL	12 x 7 1/4	1	2#6 (2#6)	30(37.5)	6.25	B
24	12 x 7 1/4	1	2#6 (2#6)	30(37.5)	6.25	B
25	12 x 7 1/4	1	2#6 (2#6)	20(25)	6.25	B

TABLE 4.3.2 cont'd.

SPECIMEN NO.	DIMENSIONS b x h(in.^{2})	CLEAR COVER	REINFORCEMENT	STEEL STRESS (ksi)	W/C (gal./sack)	POSITION OF CASTING
25BL	12 x 7 1/4	1	2#6 (2#6)	20(25)	6.25	B
26	12 x 7 1/4	1	2#6 (2#6)	20(25)	6.25	B
27	12 x 6	1	2#6 (2#4)	0	7.0	B
28	12 x 6	1 1/2	2#6 (2#4)	0	7.0	B
29	12 x 6	1	2#6 (2#4)	0	7.0	B
30	12 x 6	3/4	2#6 (2#4)	0	7.0	B
31	12 x 6	2	2#6 (2#4)	0	5.5	B
32	12 x 6	1 1/2	2#6 (2#4)	0	5.5	B
33	12 x 6	1	2#6 (2#4)	0	5.5	B
34	12 x 6	3/4	2#6 (2#4)	0	5.5	B
35	12 x 6	1	2#6 (2#4)	0	6.25	B
36	12 x 6	3/4	2#6 (2#4)	0	6.25	B
37	9 1/2 x 2 3/8	1	2 x 3/8" (P)*	f_c=1.27 ksi	7.0	-
38	12 x 1 7/8	3/4	2 x 3/8" (P)	f_c=1.27 ksi	7.0	-
39	9 1/2 x 2 3/8	1	2 x 3/8" (P)	f_c=1.27 ksi	6.25	-
40	10 x 3 3/8	1 1/2	3 x 3/8" (P)	f_c=1.27 ksi	6.25	-
41	12 x 1 7/8	3/4	2 x 3/8" (P)	f_c=1.27 ksi	6.25	-

TABLE 4.3.2 cont'd.

SPECIMEN NO.	DIMENSIONS b x h(in.^{2})	CLEAR COVER	REINFORCEMENT	STEEL STRESS (ksi)	W/C (gal./sack)	POSITION OF CASTING
42L	12 x 6	2	2#6 (2#4)	0	7.0	B
43L	12 x 6	1	2#6 (2#4)	0	7.0	B
44L	12 x 6	2	2#6 (2#4)	0	6.25	B
45L	12 x 6	1	2#6 (2#4)	0	6.25	B
46L	12 x 6	2	2#6 (2#4)	0	5.5	B
47L	12 x 6	1	2#6 (2#4)	0	5.5	B
48	12 x 6	3/4	2#6 (2#4)	0	5.5	B
49	12 x 6	1 1/2	2#6 (2#4)	0	5.5	B
50	12 x 6	3/4	2#11 (2#4)	0	5.5	B
51	12 x 6	1 1/2	2#11 (2#4)	0	5.5	B
52	12 x 6	3/4	2#8 (2#4)	0	5.5	B
53	12 x 6	1 1/2	2#8 (2#4)	0	5.5	B
54	12 x 6	1 1/2	2#11 (2#4)	0	4.75	B
55	12 x 6	3/4	2#11 (2#4)	0	4.75	B
56	12 x 6	1 1/2	2#8 (2#4)	0	4.75	B
57	12 x 6	3/4	2#8 (2#4)	0	4.75	B
58	12 x 6	3/4	2#6 (2#4)	0	4.75	B
59	12 x 6	1 1/2	2#6 (2#4)	0	4.75	B

TABLE 4.3.2 cont'd.

SPECIMEN NO.	DIMENSIONS b x h(in.^{2})	CLEAR COVER	REINFORCEMENT	STEEL STRESS (ksi)	W/C (gal./sack)	POSITION OF CASTING
60	12 x 6	3/4	2#6 (2#4)	0	6.25	B
61	12 x 6	1 1/2	2#6 (2#4)	0	6.25	B
62	12 x 6	1 1/2	2#8 (2#4)	0	6.25	B
63	12 x 6	3/4	2#8 (2#4)	0	6.25	B
64	12 x 6	1 1/2	2#11 (2#4)	0	6.25	B
65	12 x 6	3/4	2#11 (2#4)	0	6.25	B
66	12 x 6	3/4	2#6 (2#4)	0	4.75	B
67	12 x 6	3/4	2#8 (2#4)	0	7.0	B
68	12 x 6	3/4	2#11 (2#4)	0	6.25	B
69	12 x 23	1	2#11 (2#6)	30 (30)	6.25	B
70	12 x 23	1	2#11 (2#6)	30 (30)	6.25	B
71	12 x 23	1	4#8 (2#6)	30 (30)	6.25	B
72	12 x 23	1	4#8 (2#6)	30 (30)	6.25	B
73	11 x 14	1	2#8 (2#6)	30 (30)	6.25	B
74	11 x 14	1	2#8 (2#6)	36 (40)	6.25	B
75	11 x 14	1	2#6 (2#6)	36 (40)	6.25	B
76	11 x 14	1	2#6 (2#6)	36 (40)	6.25	B

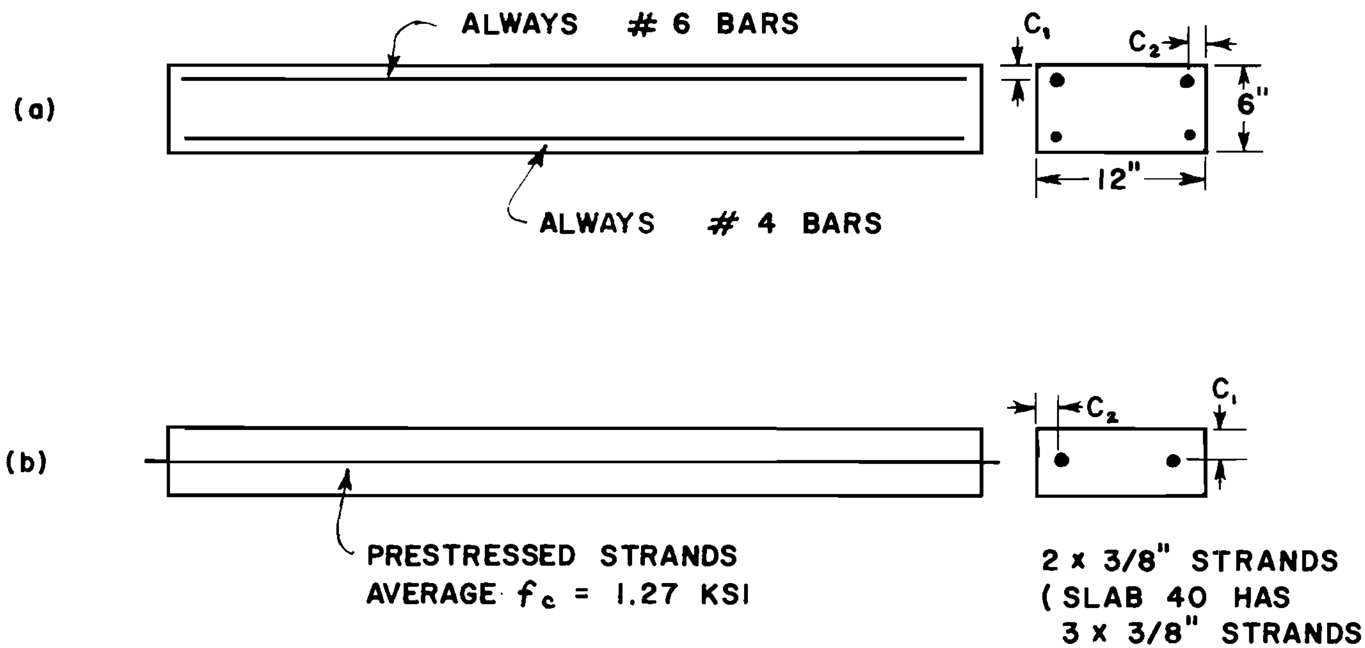


Fig. 4.3.3 (a) Details of unloaded slabs (#27 - #36)

(b) Details of prestressed slabs (#37 - #41)

Slab No.	Bar size no.	W/C ratio (gal./sk.)	Clear cover (in.)	f'_c (psi)
27	6	7.0	2	3550
28	6	7.0	1 1/2	3550
29	6	7.0	1	3550
30	6	7.0	3/4	3550
31	6	5.5	2	5350
32	6	5.5	1 1/2	5350
33	6	5.5	1	5350
34	6	5.5	3/4	5350
35	6	6.25	1	3680
36	6	6.25	3/4	3680
37	3/8"*	7.0	1	3960
38	3/8"*	7.0	3/4	3960
39	3/8"*	6.25	1	4200
40	3/8"*	6.25	1 1/2	4050
41	3/8"*	6.25	3/4	4050
42L	6	7.0	2	3660
43L	6	7.0	1	3660
44L	6	6.25	2	4960
45L	6	6.25	1	4960
46L	6	5.5	2	4850
47L	6	5.5	1	4850

*prestress strands

Beam No.	Beam cross section (width x depth)	Clear cover (in.)	Steel stress (ksi)	Casting position (bottom or top)	Bar size no.	f'_c (psi)
2	12x24	2	30(30)	T	11	3910
4	12x24	2	30(30)	B	11	5180
6	11x16	3	20(25)	B	8	3900
7A	11x15	2	20(25)	B	8	5400
12	11x15	2	36(40)	T	8	5600
15	11x14	1	30(37.5)	B	8	4000
14	11x14	1	20(25)	B	8	4000
16	11x14	1	30(37.5)	B	8	5200
18	12x8 1/4	2	36(40)	B	6	4270
20	12x8 1/4	2	30(37.5)	B	6	3900
22	12x8 1/4	2	20(25)	B	6	3740
24	12x7 1/4	1	30(37.5)	B	6	4990
26	12x7 1/4	1	20(25)	B	6	3950
15BL	11x14	1	30(37.5)	B	8	4630
14BL	11x14	1	20(25)	B	8	4630
2BL	12x24	2	30(36)	T	11	4960

Note: All slabs are 12"x6" in cross-section. "B" indicates a lag in loading time of about 20 months. "L" indicates a lightweight beam.

Beam No.	Beam cross section (width x depth)	Clear cover (in.)	Steel stress (ksi)	Casting position (bottom or top)	Bar size no.	f'_c (psi)
1	12x24	2	30(30)	T	11	3430
3	12x24	2	30(30)	T	11	4590
5A	11x16	3	30(37.5)	B	8	4460
8	11x15	2	20(25)	T	8	3670
10	11x15	2	30(37.5)	T	8	4580
9	11x15	2	30(37.5)	B	8	4490
11	11x15	2	36(40)	B	8	5070
13	11x14	1	20(25)	B	8	5020
17	12x8 1/4	2	36(40)	B	6	4200
19	12x8 1/4	2	30(37.5)	B	6	3990
21	12x8 1/4	2	20(25)	B	6	3820
23	12x7 1/4	1	30(37.5)	B	6	5120
25	12x7 1/4	1	20(25)	B	6	3970
23BL	12x7 1/4	1	30(37.5)	B	6	4880
25BL	12x7 1/4	1	20(25)	B	6	4880
9BL	11x15	2	30(37.5)	B	8	4960

All concrete strengths of normal weight beams are reduced to 28 days by Erzen's formula.

All beams have a nominal W/C ratio of 6.25 gal./sack.

Fig. 4.4.1.a. The exposure site and the properties of specimens during the first eighteen months of the research program.

	Slab No.	Bar size no.	W/C ratio (gal./sk.)	Clear cover (in.)	f'_c (psi)
	53	8	5.5	1 1/2	6100
TV	67	8	7.0	3/4	4180
TV	68	11	6.25	3/4	4520
	63	8	6.25	3/4	4540
	65	11	6.25	3/4	4590
TV	66	6	4.75	3/4	5650
	61	6	6.25	1 1/2	4540
	62	8	6.25	1 1/2	4540
	64	11	6.25	1 1/2	4540
	58	6	4.75	3/4	6300
	55	11	4.75	3/4	6300
	60	6	6.25	3/4	4540
	56	8	4.75	1 1/2	6300
	54	11	4.75	1 1/2	6300
	57	8	4.75	3/4	6300
	50	11	5.5	3/4	6900
	51	11	5.5	1 1/2	6900
	59	6	4.75	1 1/2	6300
	52	8	5.5	3/4	6900
	49	6	5.5	1 1/2	6900
	48	6	5.5	3/4	6900

Beam No.	Beam cross section (width x depth)	Clear cover (in.)	Steel stress (ksi)	Casting position (bottom or top)	Bar size no.	f'_c (psi)
2B	12x24	2	30(30)	T	11	4650
69	12x23	1	30(30)	B	11	4100
6	11x16	3	20(25)	B	8	4370
73	11x14	1	30(30)	B	8	3890
74	11x14	1	36(40)	B	8	3890
15B	11x14	1	30(37.5)	B	8	4630
14B	11x14	1	20(25)	B	8	4630
16	11x14	1	30(37.5)	B	8	5787
15BL	11x14	1	30(37.5)	B	8	4630
14BL	11x14	1	20(25)	B	8	4630
2BL	12x24	2	30(36)	T	11	4960

Beam No.	Beam cross section (width x depth)	Clear cover (in.)	Steel stress (ksi)	Casting position (bottom or top)	Bar size no.	f'_c (psi)
72	12x23	1	30(30)	B	8	4500
71	12x23	1	30(30)	B	8	4500
5A	11x16	3	30(37.5)	B	8	5030
70	12x23	1	30(30)	B	11	4100
75	11x14	1	36(40)	B	6	4710
9	11x15	2	30(37.5)	B	8	5090
11	11x15	2	36(40)	B	8	5617
76	11x14	1	36(40)	B	6	4710
23BL	12x7 1/4	1	30(37.5)	B	6	4880
25BL	12x7 1/4	1	20(25)	B	6	4880
9BL	11x14	2	30(37.5)	B	8	4850

Fig. 4.4.1.b. The exposure site and the properties of specimens after eighteen months to the end of the research program.

4.5 Concrete Surface Observations

4.5.1 Transverse Cracking

Grid lines 6 in. apart were drawn longitudinally on the loaded and unloaded test portions of all beams. At a distance "clear cover + 1/2 in." from top corners of the beams, longitudinal lines were also drawn (see Fig. 4.3.2). Flexural cracks were carefully mapped using the grid lines as references. A microscope with a magnification of 60 and reading to 10^{-4} in. was used to read crack widths. The widths of the flexural cracks were read at their crossing points with the longitudinal grid lines, that is, directly over the bars. Table 4.5.1 gives the average crack widths at the indicated age after initial loading.

4.5.2 Longitudinal Cracking

As a result of bar corrosion, longitudinal splitting sometimes developed along the top tension bars (see Fig. 4.5.1a-b). At the time the specimens were removed from the exposure bed for analysis, the length of each longitudinal crack was measured. The total sum of longitudinal cracks at one tension bar, both on top and side of the beam, was divided by the length of the test section under consideration in order to obtain the percentage of longitudinal cracking. Table 4.5.2a-b shows the percentages of longitudinal cracking for all specimens at the time they were removed for analysis.

4.6 Rust Observations

4.6.1 Surface Rusting

The initial development and the enlargement of rust spots on concrete surfaces were observed and photographed (see Fig. 4.5.1a-b). The appearance of rust and size of cracks were usually closely associated with depth of cover, amount of longitudinal splitting, and the bar size. All the photographs of the rust spots are on file in the Civil Engineering Department of The University of Texas at Austin.

4.6.2 Evaluation of Rust on Reinforcement

Care was taken to knock the bars loose from the sawed prism of concrete with minimum damage to the surrounding concrete. The degree of corrosion

(text continued p. 117)

TABLE 4.5.1

AVERAGE CRACK WIDTHS OF LOADED SPECIMENS
(All readings are 10^{-4} in. units)

BEAM 1

Read At (Days)	0	135	366	469	544
NT*	43	52	62	78	92
NS*	43	35	52	75	66
ST*	38	45	54	58	103
SS*	32	32	38	66	48

BEAM 2

Read At (Days)	0	131	389	465	540	574
NT*	55	86	141	109	151	147
NS*	58	53	119	84	76	63
ST*	67	100	86	98	124	121
SS*	58	60	151	61	80	91

	BEAM 2B				BEAM 2BL		
Read At (Days)	0	129	245	352	0	131	352
NT*	79	85	121	93	76	135	52
NS*	67	56	61	125	44	93	119
ST*	77	83	124	126	69	133	112
SS*	65	56	74	83	42	74	61

BEAM 3

Read At (Days)	0	113	346	417	491	689
NT*	66	84	189	120	110	120
NS*	61	58	84	67	57	119
ST*	92	103	218	151	148	132
SS*	70	87	90	81	71	91

*NT = At north and top of the beam
*NS = At north and side of the beam
*ST = At south and top of the beam
*SS = At south and side of the beam

TABLE 4.5.1 cont'd.

BEAM 4

Read At (Days)	0	151	359	457	537	760
NT*	46	71	61	72	73	126
NS*	55	61	51	66	51	50
ST*	48	91	58	61	70	83
SS*	43	44	34	47	47	53

BEAM 5A

Read At (Days)	0	101	344	441	522	917
NT*	83	90	80	91	102	169
NS*	62	61	57	65	48	80
ST*	81	87	68	69	66	126
SS*	55	58	52	47	43	98

BEAM 6

Read At (Days)	0	122	341	438	519	1038
NT*	74	78	102	91	93	188
NS*	56	66	44	49	49	110
ST*	84	86	80	114	108	172
SS*	52	52	53	47	52	103

BEAM 7A

Read At (Days)	0	132	343	438	521	718
NT*	41	43	38	46	42	61
NS*	34	37	31	47	33	36
ST*	55	53	73	60	51	94
SS*	46	47	50	50	47	121

BEAM 8

Read At (Days)	0	157	361	457	540	771
NT*	63	66	133	72	69	170
NS*	44	40	47	46	41	81
ST*	52	77	88	110	68	143
SS*	44	47	44	45	41	49

	BEAM 9						BEAM 9BL		
Read At (Days)	0	115	319	444	529	923	0	131	352
NT*	56	67	55	60	50	232	75	116	133
NS*	45	42	42	43	41	120	45	99	112
ST*	54	55	62	74	68	263	69	122	107
SS*	43	35	41	39	41	261	39	78	96

TABLE 4.5.1 cont'd.

BEAM 10

Read At (Days)	0	137	342	467	551	755
NT*	76	83	109	88	133	122
NS*	46	54	52	51	52	68
ST*	80	78	96	127	110	
SS*	45	44	42	42	40	82

BEAM 11

Read At (Days)	0	146	349	485	575	930
NT*	62	83	70	76	100	115
NS*	55	65	58	55	58	71
ST*	55	65	77	83	73	110
SS*	45	60	63	66	52	72

BEAM 12

Read At (Days)	0	125	307	520	610	830
NT*	83	101	90	115	122	155
NS*	70	63	51	59	51	65
ST*	86	116	99	145	149	115
SS*	50	70	50	62	52	69

BEAM 13

Read At (Days)	0	119	323	394	504
NT*	22	31	26	30	28
NS*	18	30	26	24	24
ST*	28	29	36	32	31
SS*	20	24	27	31	20

BEAM 14

Read At (Days)	0	112	321	390	499
NT*	22	32	39	36	45
NS*	23	33	30	25	20
ST*	20	33	29	33	29
SS*	14	24	16	23	19

	BEAM 14B				BEAM 14BL		
Read At (Days)	0	62	179	308	0	133	268
NT*	36	36	32	40	18	26	25
NS*	19	15	28	36	14	14	17
ST*	27	34	26	61	25	34	32
SS*	19	20	19	46	16	21	24

TABLE 4.5.1 cont'd.

BEAM 15

Read At (Days)	0	119	328	396	497	550
NT*	26	42	45	44	49	40
NS*	23	30	30	23	29	30
ST*	37	55	65	74	63	45
SS*	30	36	38	32	37	32

	BEAM 15B			BEAM 15BL		
Read At (Days)	0	62	179	0	133	268
NT*	30	24	33	25	24	108
NS*	20	15	26	19	25	93
ST*	41	36	45	18	20	103
SS*	30	29	31	13	20	63

BEAM 16

Read At (Days)	0	118	325	412	552
NT*	35	51	32	32	39
NS*	27	30	25	24	30
ST*	32	44	62	46	52
SS*	24	33	25	29	25

BEAM 17

Read At (Days)	0	119	315	406	496	710
NT*	94	108	113	90	100	156
NS*	77	90	41	50	54	69
ST*	55	97	97	95	96	137
SS*	73	72	44	40	42	62

BEAM 18

Read At (Days)	0	92	290	384	467	704
NT*	81	98	102	80	92	97
NS*	56	62	80	49	51	56
ST*	84	101	90	115	118	124
SS*	56	58	35	37	34	46

BEAM 19

Read At (Days)	0	96	291	378	467	681
NT*	79	101	126	69	84	81
NS*	59	65	62	52	57	45
ST*	73	83	93	92	75	110
SS*	50	58	57	54	48	55

TABLE 4.5.1 cont'd.

BEAM 20

Read At (Days)	0	92	289	379	466	674
NT*	90	109	70	89	81	126
NS*	69	74	43	49	54	109
ST*	92	108	77	99	114	105
SS*	56	68	55	73	52	65

BEAM 21

Read At (Days)	0	96	194	282	371	585
NT*	68	75	76	50	55	61
NS*	47	54	39	32	29	34
ST*	57	61	40	52	41	104
SS*	30	39	27	34	29	36

BEAM 22

Read At (Days)	0	92	289	372	462	674
NT*	61	63	44	52	48	94
NS*	46	45	21	30	32	42
ST*	54	72	45	57	46	80
SS*	45	46	21	35	32	46

BEAM 23

Read At (Days)	0	96	289	360	463	648	779
NT*	54	56	39	38	40	17	42
NS*	48	50	39	36	38	15	19
ST*	47	51	28	43	48	13	42
SS*	37	44	28	24	27	10	29

BEAM 23BL

Read At (Days)	0	136	306
NT*	17	41	48
NS*	15	19	25
ST*	13	42	49
SS*	10	29	36

BEAM 24

Read At (Days)	0	92	283	359	461	674
NT*	56	52	36	41	37	82
NS*	53	50	31	34	36	
ST*	52	51	38	39	41	78
SS*	45	45	36	18	33	28

TABLE 4.5.1 cont'd.

BEAM 25

Read At (Days)	0	96	282	359	460	648	779	896
NT*	46	48	47	35	35	30	33	118
NS*	37	37	54	31	29	16	25	93
ST*	41	41	38	29	43	33	30	49
SS*	38	40	29	26	32	19	14	32

BEAM 25BL

Read At (Days)	0	75	234
NT*	31	33	30
NS*	67	25	24
ST*	33	29	30
SS*	19	14	21

BEAM 26

Read At (Days)	0	92	283	357	460	675
NT*	44	48	23	40	32	55
NS*	42	44	28	38	28	44
ST*	28	34	42	28	28	25
SS*	27	31	32	25	21	26

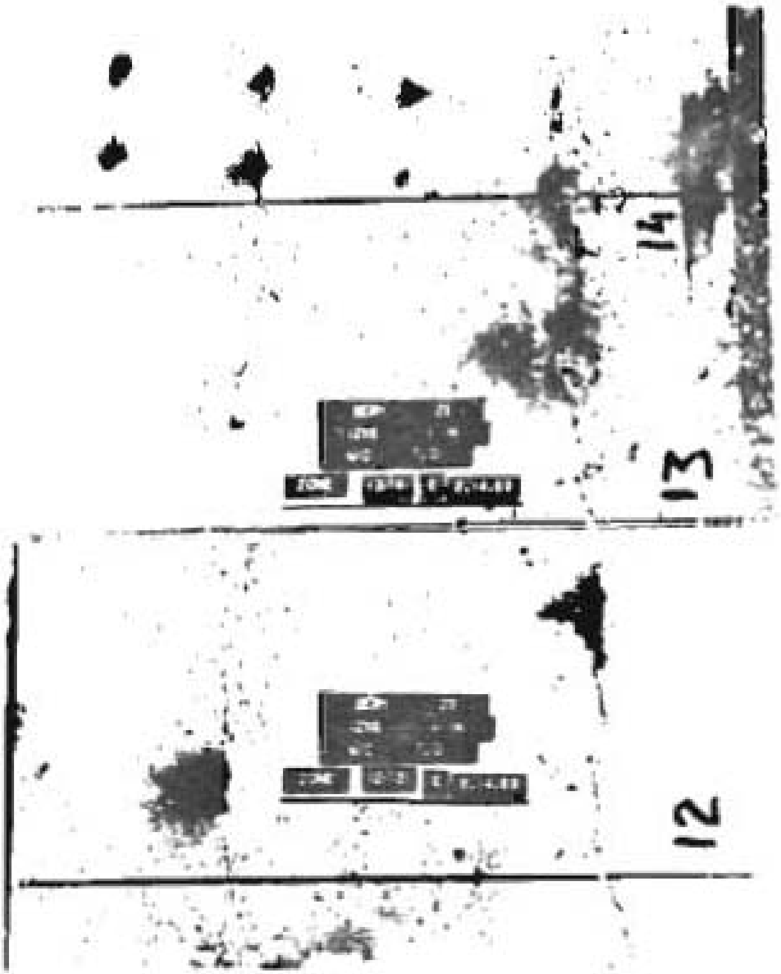
	BEAM 69		BEAM 70	
Read At (Days)	0	149	0	149
NT*	27	45	21	52
NS*	25	36	21	46
ST*	24	40	30	34
SS*	24	22	29	41

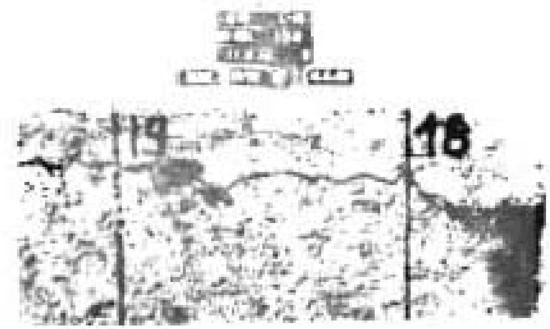
	BEAM 71		BEAM 72	
Read At (Days)	0	148	0	148
NT*	30	34	26	30
NS*	21	29	24	29
ST*	18	29	23	29
SS*	20	18	20	18

	BEAM 73		BEAM 74	
Read At (Days)	0	146	0	154
NT*	38	31	83	46
NS*	33	23	52	44
ST*	37	46	46	77
SS*	28	35	60	45

TABLE 4.5.1 cont'd.

	BEAM 75		BEAM 76	
Read At (Days)	0	148	0	148
NT*	32	54	26	32
NS*	35	32	26	23
ST*	36	44	43	29
SS*	31	56	39	35





(side view)

(top view)

a. Longitudinal cracking in unloaded slab

b. Longitudinal cracking in loaded beam

Fig. 4.5.1 Crack development as a result of corrosion of reinforcing bars in loaded and unloaded specimens.

TABLE 4.5.2a

LONGITUDINAL CRACKING OF CORROSION SPECIMENS
(Loaded Beams)

SPECIMEN NUMBER	NORTH BAR PERCENTAGE (%)		SOUTH BAR PERCENTAGE (%)		EXPOSURE (Months)
	STRESSED	UNSTRESSED	STRESSED	UNSTRESSED	
1	100	44	100	100	34
2	89	7	48	11	19
2B	23	0	17	0	12
2BL	7	0	3	0	12
3	100	9	42	42	23
4	100	0	79	60	24
5A	0	0	15	0	31
6	32	0	51	0	32
7A	48	100	100	8	25
8	95	99	49	86	25
9	100	48	100	100	31
9BL	1	0	3	0	12
10	100	100	100	100	25
11	94	27	22	12	32
12	35	0	55	98	24
13	68	9	100	0	24
14	100	48	100	45	19
14B	100	0	100	0	11
14BL	10	0	4	0	9
15	100	47	100	100	19
15B	33	0	100	0	10
15BL	100	23	42	0	9
16	100	100	100	100	31
17	13	9	0	9	24
18	4	0	0	0	24
19	0	0	0	0	24
20	3	0	0	6	24
21	3	50	0	0	24
22	0	10	0	4	24
23	100	69	100	24	29
23BL	4	0	56	0	9
24	66	36	58	36	24
25	100	100	100	87	29
25BL	0	0	13	0	9
26	100	71	94	98	24
69	100	0	96	0	6
70	88.6	0	100	0	6

TABLE 4.5.2a cont'd.

LONGITUDINAL CRACKING OF CORROSION SPECIMENS
(Loaded Beams)

SPECIMEN NUMBER	NORTH BAR PERCENTAGE (%)		SOUTH BAR PERCENTAGE (%)		EXPOSURE (Months)
	STRESSED	UNSTRESSED	STRESSED	UNSTRESSED	
71	48	0	61	0	6
72	23	0	0	0	6
73	57	0	100	0	6
74	25	0	100	0	6
75	55	0	50	0	6
76	91	0	8	0	6

TABLE 4.5.2b
LONGITUDINAL CRACKING OF CORROSION SPECIMENS
(Unloaded Slabs)

SPECIMEN NUMBER	NORTH BAR PERCENTAGE (%)	SOUTH BAR PERCENTAGE (%)	EXPOSURE (Months)
27	45	89	24
28	96	91	24
29	100	99	24
30	100	100	24
31	Data not taken	Data not taken	24
32	39	41	24
33	100	94	24
34	100	100	24
35	100	100	24
36	100	100	24
37	93	49	24
38	Data not taken	Data not taken	24
39	41	38	24
40	59	33	24
41	30	73	24
42L	42	64	12M
43L	44	50	12M
44L	0	30	12M
45L	27	14	12M
46L	14	4	12M
47L	0	0	12M
48	0	0	6M
49	0	0	6M
50	0	0	6M
51	0	8	6M
52	0	0	6M
53	0	0	6M
54	0	0	6M
55	16	44	6M
56	0	0	6M
57	0	0	6M
58	0	16	6M
59	0	0	6M
60	20	8	6M
61	0	5.4	6M
62	0	5	6M
63	61	63	6M
64	7	0	6M

TABLE 4.5.2b cont'd.
LONGITUDINAL CRACKING OF CORROSION SPECIMENS
(Unloaded Slabs)

SPECIMEN NUMBER	NORTH BAR PERCENTAGE (%)	SOUTH BAR PERCENTAGE (%)	EXPOSURE (Months)
65	91	99	6M
66	0	4	6M
67	94	100	6M
68	100	91	6M

on the bar surfaces was visually observed at 1/2 in. intervals along the bar length and estimated as a percentage of the bar surface. An average percentage of corrosion is calculated for each bar by finding the area under the corrosion distribution curve and dividing it by the length of the bar (see Fig. 3.5.1). This resulted in a weighted average for the degree of surface corrosion. Table 4.6.1, a-b, summarizes the results of such calculations for all the analyzed bars.

4.7 Concrete Water Penetration Test

Since it has been widely reported that permeability is a very significant concrete property related to corrosion resistance, a simple test method was sought to provide a measure of the relative permeabilities of the concretes of this study. The more common methods used for determining permeability were ruled out in this program due to the requirements of machined pressure chambers and leak-proof seals

In one of the few published research reports dealing with concrete permeability a description of a new approach for measuring permeability was given.[45] In that study it was concluded that a measure of the penetration of water into a concrete specimen provided an accurate and fast determination of permeability of concrete. Unfortunately, the method described required sealed pressure chambers, as a significant pressure was used to force the water into the concrete specimen. The writers therefore decided to adopt the idea of water penetration measurement and modify the procedure to eliminate the use of high pressure.

The basic procedure developed for the water penetration test is to oven dry a concrete specimen and then coat all but the horizontal surfaces with a fast curing epoxy. No specific requirements are placed on specimen shape, except that the vertical dimension should be constant for all specimens to be compared Both prisms and cylinder shapes have been successfully used. After the epoxy coating has cured, the specimen is placed into a vat of water to a depth of 5 in. above the horizontal, uncoated, bottom surface.

(text continued p. 122)

TABLE 4.6.1a

WEIGHTED AVERAGE SURFACE CORROSION ON BARS
(Loaded Beams)

BEAM NO.	CLEAR COVER (in.)	BAR SIZE #	TS* %	TN* %	CS* %	CN* %	UTS* %	UTN* %	UCS* %	UCN* %	EXPOSURE MONTH
1	2	11	100	100	27	68	87	70	55	92	34M
2	2	11	74	85	17	41	66	64	0	0	19M
2B	2	11	10	4	5	0	0	0	0	0	12M
2BL	2	11	3	7	0	0	0	0	0	0	12M
3	2	11	43	52	8	6	30	20	0	16	24M
4	2	11	45	41	15	33	47	40	0	43	24M
5A	3	8	27	9	36	16	0	6	21	67	31M
6	3	8	44	37	29	23	35	6	27	34	32M
7A	2	8	20	28	18	13	5	38	28	14	24M
8	2	8	38	30	5	29	27	51	30	20	24M
9	2	8	45	42	37	38	46	21	80	5	31M
9BL	2	8	3	1	0	0	0	0	0	0	12M
10	2	8	25	25	20	22	41	11	32	0	24M
11	2	8	13	31	22	38	26	28	23	59	32M
12	2	8	37	19	28	14	35	10	36	15	24M
13	1	8	100	100	94	70	98	91	52	100	24M
14	1	8	100	100	90	74	91	85	20	66	19M
14B	1	8	55	56	41	32	31	44	43	24	11M
14BL	1	8	20	20	7	3	10	10	0	40	9M
15	1	8	100	100	76	59	100	100	44	69	19M
15B	1	8	51	27	12	10	36	20	25	15	10M
15BL	1	8	36	19	5	0	4	13	0	20	9M
16	1	8	100	100	98	92	100	100	97	99	31M
17	2	6	16	20	3	0	2	6	29	19	24M
18	2	6	15	27	4	1	0	6	40	26	24M
19	2	6	25	10	1	0	12	0	46	0	24M
20	2	6	13	19	0	0	7	6	10	43	24M
21	2	6	10	17	0	0	18	29	5	12	24M
22	2	6	21	9	0	0	13	0	27	2	24M
23	1	6	79	60	1	4	61	29	28	30	29M
23BL	1	6	9	7	0	0	0	1	0	0	9M
24	1	6	36	59	3	2	32	33	13	12	24M
25	1	6	100	100	27	68	87	70	55	92	29M
25BL	1	6	9	4	0	0	2	0	0	0	9M
26	1	6	89	60	43	30	42	55	54	50	24M
69	1	11	22	17	3	0	6	2	0	0	6M

TABLE 4.6.1a cont'd.

BEAM NO.	CLEAR COVER (in.)	BAR SIZE #	TS* %	TN* %	CS* %	CN* %	UTS* %	UTN* %	UCS* %	UCN* %	EXPOSURE MONTH
70	1	11	28	18	2	1	0	2	0	0	6M
71	1	8	19/8**	11/13**	0	1	0/0**	0/0**	0	4	6M
72	1	8	5/5**	7/5**	1	2	0/0**	2/0**	2	1	6M
73	1	8	13	10	2	0	3	4	4	0	6M
74	1	8	7	10	7	1	3	0	5	3	6M
75	1	6	17	10	2	0	0	0	0	1	6M
76	1	6	6	20	0	0	0	0	0	0	6M

*TS = South top bar
*TN = North top bar
*UTS = Unstressed South top bar
*UTN = Unstressed North top bar
*CS = South bottom bar
*CN = North bottom bar
*UCS = Unstressed South bottom bar
*UCN = Unstressed North bottom bar

**Beams 71 and 72 have 4 #8 bars. The values indicated are for the bars in the middle.

TABLE 4.6.1b

WEIGHTED AVERAGE SURFACE CORROSION ON BARS
(Unloaded Slabs)

SLAB NO.	CLEAR COVER (in.)	BAR SIZE #	TS* %	TN* %	CS* %	CN* %	EXPOSURE MONTH
27	1	6	70	81	7	13	24M
28	1 1/2	6	100	97	76	36	24M
29	1	6	100	100	53	37	24M
30	3/4	6	99	100	92	61	24M
31	2	6	0	0	0	0	24M
32	1 1/2	6	20	24	2	4	24M
33	1	6	47	42	0	0	24M
34	3/4	6	55	44	8	14	24M
35	1	6	64	56	40	65	24M
36	3/4	6	85	91	29	50	24M
37	1	3/8"⊛	82	72	--	--	24M
38	3/4	3/8"⊛	80	92	--	--	24M
39	1	3/8"⊛	59	77	--	--	24M
40	1 1/2	3/8"⊛	58	52	44**	--	24M
41	3/4	3/8"⊛	82	78	--	--	24M
42L	2	6	9	15	--	--	12M
43L	1	6	27	9	--	--	12M
44L	2	6	0	12	--	--	12M
45L	1	6	5	1	--	--	12M
46L	2	6	2	6	--	--	12M
47L	1	6	0	0	--	--	6M
48	3/4	6	2	1	--	--	6M
49	1 1/2	6	0	0	--	--	6M
50	3/4	11	12	1	--	--	6M
51	1 1/2	11	0	0	--	--	6M
52	3/4	8	5	1	--	--	6M
53	1 1/2	8	0	0	--	--	6M
54	1 1/2	11	0	0	--	--	6M
55	3/4	11	10	3	--	--	6M
56	1 1/2	8	0	0	--	--	6M
57	3/4	8	5	1	--	--	6M
58	3/4	6	2	2	--	--	6M
59	1 1/2	6	0	0	--	--	6M
60	1 1/2	6	0	0	--	--	6M
61	3/4	6	11	6	--	--	6M
62	1 1/2	8	0	0	--	--	6M

TABLE 4.6.1b cont'd.

SLAB NO.	CLEAR COVER (in.)	BAR SIZE #	TS* %	TN* %	CS* %	CN* %	EXPOSURE MONTH
63	3/4	8	16	18	--	--	6M
64	1 1/2	11	0	0	--	--	6M
65	3/4	11	24	23	--	--	6M
66	3/4	6	3	7	--	--	6M
67	3/4	8	28	37	--	--	6M
68	3/4	11	36	39	--	--	6M

*TS = South top bar
*TN = North top bar

*CS = South bottom bar
*CN = North bottom bar

⊕ Prestressed strands

** Slab 40 has three prestressed strands. Last percentage belongs to the middle strand.

This provides a slight hydrostatic pressure of 0.18 lb/in.^{2} at the bottom surface as the water penetrates vertically, upward into the concrete. A photograph of both prism and core type specimens submerged during the penetration test is shown in Fig 4.7.1a. After a specified soak period, the specimens are removed from the water vat and broken open (Fig. 4.7.1b). The depth of penetration is then marked as shown in the split cores of Fig. 4.7.1c.

The initial development of the test required that an effective soak interval be determined. Therefore, a pilot study was made in which prism specimens were cut from a single unreinforced slab and tested at soak intervals of 2, 6, and 24 hours. Concretes made with 3/8 in. siliceous aggregates and having water-cement ratios of 5.5 and 6.75 gal/sk were used. After 24 months' exposure, prism specimens were cut from the slabs for permeability testing. The results of this pilot study are given in the depth of penetration vs. time plot of Fig. 4.7.2.

From the results of Fig. 4.7.2 it was decided to use a constant soak period of 24 hours for all subsequent penetration tests. This time interval was used primarily because the pilot study indicated that for shorter soak periods the difference between penetration depths for different quality concretes was small.

As a result of the pilot study a test procedure was selected and followed for the penetration tests reported for this research program.* That procedure is summarized step by step as follows:

1. After a thorough washing, oven dry the specimens to a constant weight at a temperature of 210-220°F.
2. Remove the specimens from the oven and seal all vertical surfaces with epoxy.

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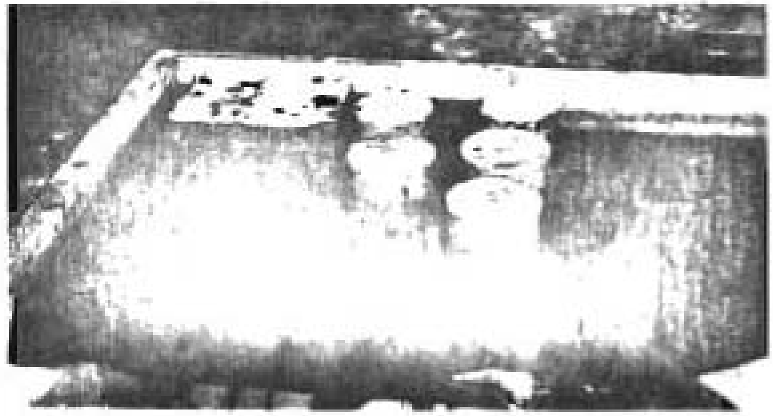
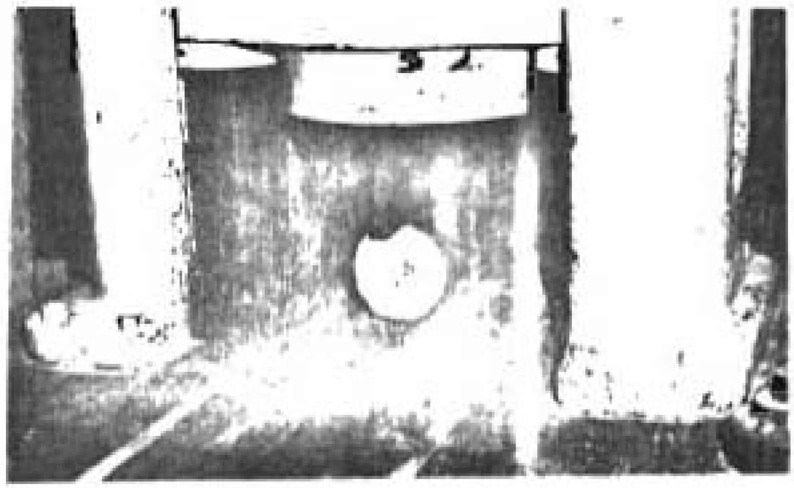
*One exception to the procedure was required due to the limitation of oven facilities. A few specimens were oven dried at 125°F for a minimum of 14 days.

a. Submerged specimens



b. Splitting of specimen



c. Water penetration marking

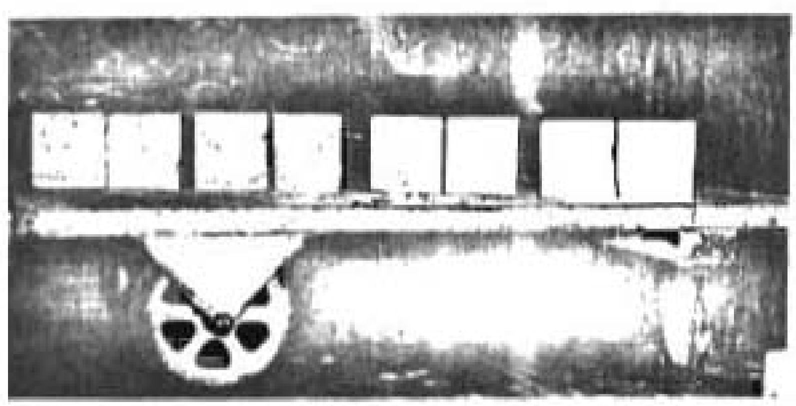


Fig. 4.7.1 Typical specimens in sequence of water penetration test.

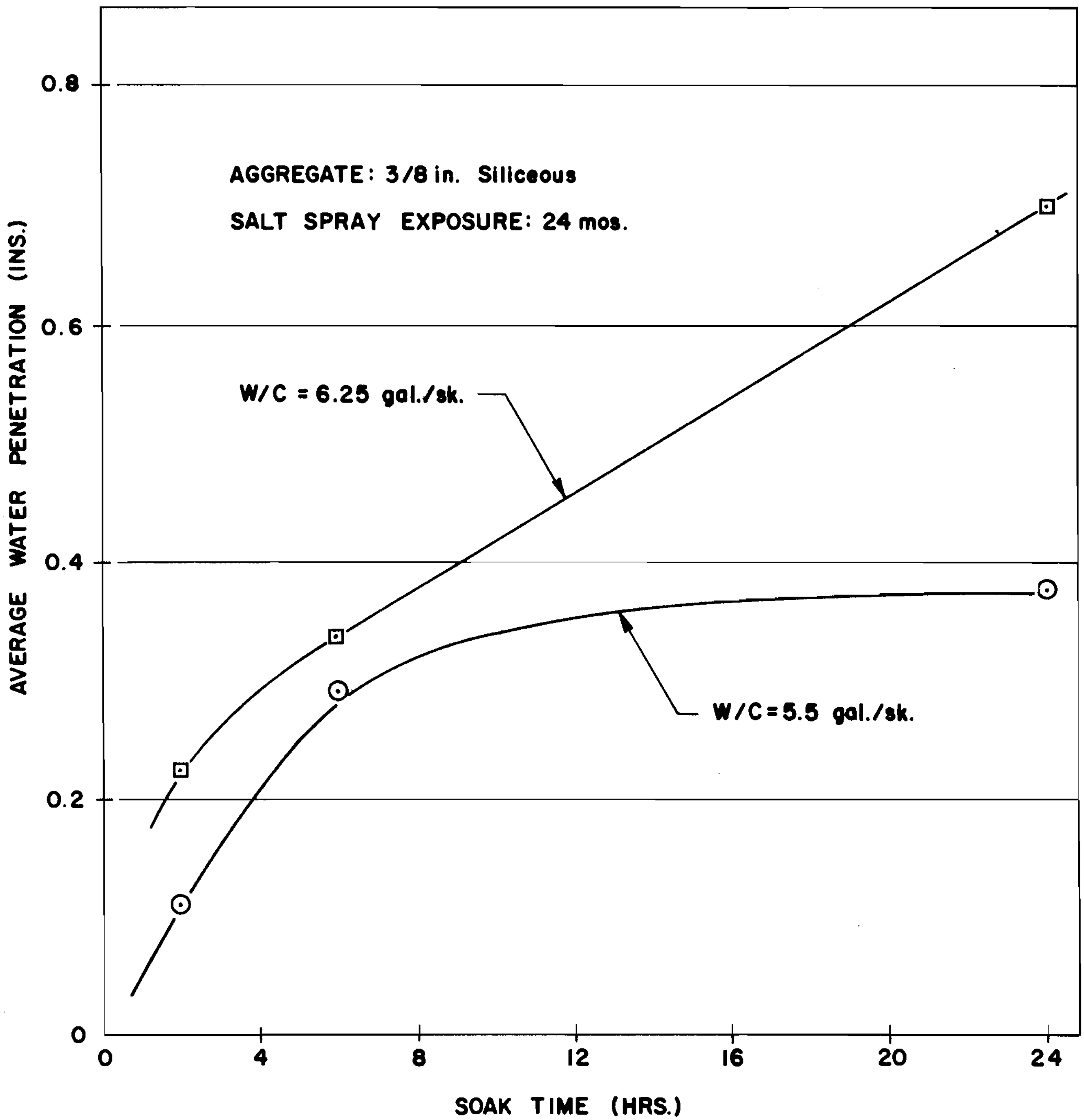


Fig. 4.7.2 Water penetration into concrete at various soak intervals.

3. Place the specimens back into the oven for an additional 24 hours and then remove and cover with polyethylene sheeting until cooled to room temperature.

4. Place the specimens into a water vat so that no air bubbles are trapped beneath the bottom surface. The specimen should be supported by point or line contact and not be placed directly against the bottom.

5. Adjust the water level as needed to provide a constant head of 5 in. above the bottom surface of the specimen

6. Remove the specimen from the water vat after 24 hours and immediately split it open at the center along a vertical plane.

7. Mark the depth of penetration along the split cross section.

8. Measure and record the penetration depth at 1/2 in. intervals between the outside edges of the specimen.

9. Compute the average depth of water penetration by summing the 1/2 in. interval measurements and dividing by the number of measurements.

4.8 Survey of Concrete Specifications Related to Corrosion

Concrete specifications of the Texas Highway Department (latest editions), AREA (1968), AASHO (Interim Specifications, 1970), and ACI (318-63 & 71) are reviewed and those parts related to corrosion are tabulated in Table 4.7.1.

It is observed that the required clear covers for the main bars vary from 1-1/2 in. to 4 in. depending on the exposure conditions and the specification being considered.

The ACI Building Code Requirements make a distinction between covers on bars equal or smaller than #5 and bars greater than #5. The authors feel that this is meant to reflect the danger of much greater percentage of weight reduction due to corrosion of smaller bars and does not imply the special effect of bar diameter as found in this research.

TABLE 4.7.1

SUMMARY OF CONCRETE SPECIFICATIONS RELATED TO CORROSION OF EMBEDDED REINFORCEMENT

SPECIFYING AGENCY	W/C (gal/sack)	C. F. (sacks/cu.yd.)	SLUMP (in.)	COVER (minimum) (in.)
Texas Highway Department				
1. Concrete is in contact with soil	7	5.0	3-4	--
2. Concrete is exposed to sea water	7	6.0	5-6	--
3. General structural concrete	7	5.0 5.5*~7.0*	3	--
4. Prestressed concrete	5.75	5.5-7.0 6.0*~8.0*	2-3	--
AASHO				
1. Concrete is in contact with soil	--	4.5~5.5	2-3	3
2. Concrete is exposed to sea water	--	6.0	3-5	4 (at corners) 3 (for precast concrete piles)
3. General structural concrete	--	6.0	3-5	2 1 1/2 (top of slab)
4. Prestressed concrete	--	7.0	3-5	1 1/2 (top of slab when de-icers are used)
AREA				
1. Concrete is in contact with soil	5-6	--	--	3
2. Concrete is exposed to sea water	5	6.0~7.0	--	3 4 (at corners)
3. General structural concrete	6	--	--	--
4. Prestressed concrete	--	--	--	--

TABLE 4.7.1 cont'd.

SPECIFYING AGENCY	w/c (gal/sack)	C.F. (sacks/cu. yd.)	SLUMP (in.)	COVER (minimum) (in.)
ACI (318-63)				
1. Concrete is in contact with soil	6	--	$2 \approx 5$	3
2. Concrete is exposed to sea water	5	--	$3 \approx 6$	--
3. General structural concrete	6	--	$3 \approx 6$	1 1/2 (when bar D $\leq$ No. 5, exposed) 2 (when bar D $>$ No. 5, exposed)
4. Prestressed concrete	--	--	$3 \approx 6$	1 1/2 (top of slab) 2 (when de-icers are used)
ACI (318-71) Concrete Cover Requirements Only				
Minimum cover requirements given in section 7.14 are specified for various bar sizes, exposure, and concrete type. Selected maximums and minimums only are given here.				
1. Cast in place concrete (non-prestressed)				
a. cast against and permanently exposed to earth.				3 in. (largest given)
b. shells and folded plates, No. 5 and smaller				1/2 in. (smallest given)
2. Precast concrete (under plant control)				
a. all members except walls, No. 14- No. 18 bars				2 in. (largest given)
b. shells and folded plates, No. 5 and smaller				3/8 in. (smallest given)
3. Prestressed concrete members (prestressed and nonprestressed concrete)				
a. cast against and permanently exposed to earth				3 in. (largest given)
b. shells and folded plates, No. 5 and smaller				3/8 in. (smallest given)
4. When severe corrosion exposure conditions exist, specified minimum cover shall be suitably increased.				

*For lightweight concrete

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