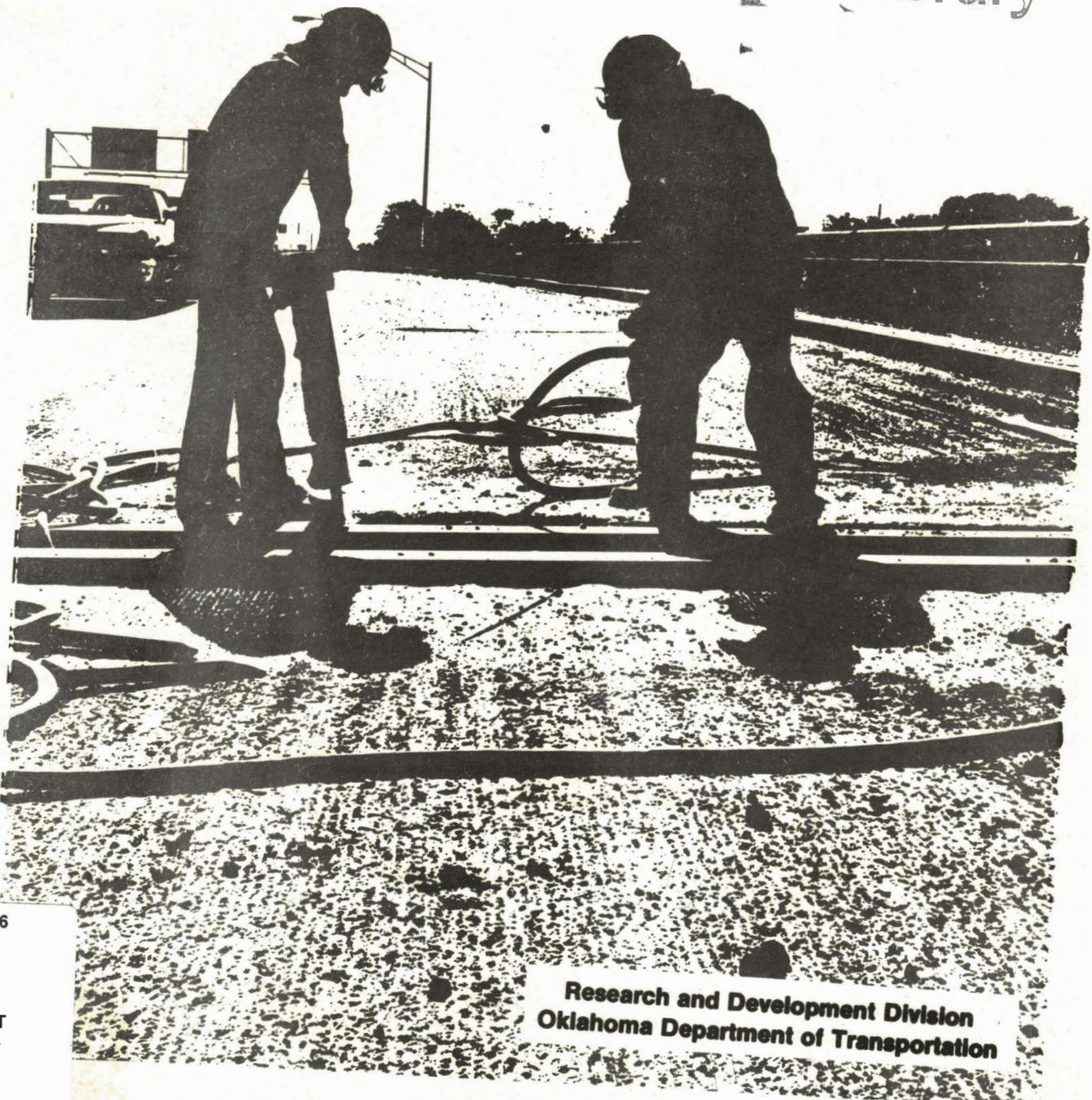




BRIDGE DECK REHABILITATION

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BRIDGE DECK REHABILITATION: METHODS AND MATERIALS

PART I

By

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16. Abstract The bridge deck deterioration processes of freeze-thaw scale, popout, and corrosion induced spalling are defined and the popular theories accounting for their mechanism are explained. Included is a basic discussion of corrosion . Oklahoma's method for surveying structures for deterioration is presented as well as the results of a statewide survey of structures using this method. Results of an attempt to correlate chloride concentration in concrete at the level of the upper rebar mat against half-cell potential is also included.			
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INTRODUCTION

In recent years the problem of deteriorating bridge decks has grown at an alarming rate. Various estimates have been made concerning the number of bridges in advanced states of deterioration, but the number most commonly used is 100,000 bridges nationwide. These 2,000 per state problems have stimulated interest in the areas of concrete and reinforcing steel deterioration. This renewed interest has produced a large amount of new research in this area, as well as a review of concrete research of the past 20 to 30 years. Part one of this three part report presents a synopsis of the results of these research efforts with a emphasis on reporting the theories that attempt to explain the mechanism of the various major deterioration types. A second bias will be to emphasize the deterioration types that have been found most prevalent and therefore most pertinent to Oklahoma. These forms of deterioration are: popouts and pitting - generally a problem associated with the coarse aggregate portion of concrete; freeze/thaw scale (salt scale) - a problem area associated with the concrete paste; and spalling-a potholing of bridge decks generally attributed to salt induced corrosion of the reinforcing steel. By and large these three deterioration types account for most of the bridge deck problems in Oklahoma.

The methodology and results of the bridge deck testing that was started in the early days of this project is presented as well as the changes in this methodology that have resulted in the bridge deck testing procedure now in use in Oklahoma.

DISCUSSION

The Structure of concrete

Prior to a discussion of the various deterioration types and theories a brief review of the make-up of concrete is in order.

Concrete is made up of coarse aggregate, fine aggregate, and a binding agent of hydrated cement called the gel. The combination of the gel and the fine aggregate is referred to as cement paste.

The gel is comprised of the hydrated products of the raw cement. The preponderance of the bulk and strength of the gel is contributed by two products; tricalcium silicate hydrate and beta dicalcium silicate hydrate. These two products make up approximately 75 percent of the concrete gel. The remainder of the gel is calcium hydroxide which is created during hydration, and the hydration products of tricalcium aluminate and tetracalcium aluminato ferrite and various other constituents of very minor proportions. The gel, commonly referred to as tobermorite gel because of its similarity to the natural mineral tobermorite, is the binding agent of concrete. The qualities of this gel along with those of the coarse and fine aggregate describe the strength, dimensional stability, and resistance to deterioration of the concrete of which they are a part.

The gel is composed of clusters of the above hydration products that often contain bits of unhydrated cement, and very small air voids called gel pores. These gel pores as we shall see, play a key role in the freeze-thaw deterioration of concrete. Gel pores along with three other types of air voids, are instrumental in describing the degree of resistance that a concrete possesses toward deterioration. Since these voids play such an important role in the durability of concrete they will be described individually by type.

Gel pores - These are pores within the colloidal gel. They are quite small, 15 to 20 Angstroms in diameter (approximately eight times the diameter of a water molecule). They are generally filled with water; however, the water within these gel pores behaves quite differently from free water because of the very large surface charges associated with such small cavities. As an example it has been estimated that water in these gel pores will not freeze until the temperature falls to -108.4°F (-78°C). These gel pores comprise 25 to 28 percent of the bulk volume of the gel.

Capillary cavities - Voids that are created when water in excess of the amount required for hydration in the fresh concrete evaporates. The higher the water-

cement ratio in the fresh concrete, the greater will be the percentage of these cavities in the hardened paste. They are thought to be at least partially interconnected in concretes of the lowest water/cement ratios with the degree of interconnection rising rapidly with higher water/cement ratios. The size of these cavities, although quite variable, averages about 5,000 Angstroms in diameter. This size is large enough to permit any water trapped within to freeze at temperatures commonly encountered in nature, although the freezing point may be depressed from the normal 32°F (0°C).

Entrained air voids - Spherical voids or bubbles intentionally introduced into the plastic concrete in order to increase the concrete's resistance to weathering. They typically comprise approximately five percent of the total concrete volume and range in diameter from a few microns to a few millimeters with an average size of about .004 in. (.1 mm). The smallest entrained air voids are 40,000 times the diameter of gel pores. These voids contain some water, but are rarely filled because capillary action preferentially fills the smaller capillary cavities.

Entrapped air - An unavoidable part of all concrete, these air voids, as the name implies, are trapped within the mix during batching and placing and are not removed during consolidation. They are generally irregularly shaped and range in size from barely visible to the unaided eye to one-half inch or more.

The preceding is a cursory discussion of the structure of concrete. Its intent is to acquaint the reader sufficiently with concrete in order that he may better understand the following discussion of concrete deterioration.

This section will present a discussion of the three major deterioration types and describe their individual physical characteristics as well as the popular theories concerning the different deterioration processes.

Freeze-thaw or salt scale - Freeze-thaw scale is the disintegration of the paste portion of concrete through the action of freezing and thawing. Freeze thaw scale is generally categorized into four degrees of severity. (1) Light scale - Loss of

surface mortar to a depth of one-quarter inch. Light scale is pictured in Figures 1 and 2. (2) Medium scale - Loss of surface mortar one-quarter to one-half inch in depth with some loss of mortar between coarse aggregate. See Figure 3. (3) Heavy scale - Loss of surface mortar and mortar surrounding aggregate particles one-half to one inch in depth, so that aggregate is clearly exposed and stands out from the concrete. (4) Severe scale - loss of surface mortar, mortar surrounding aggregate and aggregate particles to a depth of greater than 1 inch.

There are several hypothesized mechanisms to explain the freeze-thaw damage of concrete, but three are the most plausible and have the greatest experimental support. It is very likely that no single action accounts for all freeze-thaw damage but instead a combination of the following mechanisms and perhaps others not yet understood may be responsible.

1. Hydraulic Pressure Theory - The most popular explanation for freeze-thaw scale is the hydraulic pressure mechanism first postulated by Powers.¹ In this theory Powers hypothesized that hydraulic pressure could be created in the water contained in a capillary void upon freezing, and that these pressures would be great enough to exceed the tensile strength of the surrounding cement paste. Powers' contention was that as ice crystals began to form in a capillary pore the increased volume of the ice over the volume of the water from which the ice was formed (approximately nine percent) would create a hydraulic pressure in the surrounding liquid water. The basis for this hypothesis is the fact that all the water in a capillary would not freeze at the same time or temperature. This contention is undoubtedly true because differential salt concentration (amplified by the formation of ice) and the thermodynamic factors due to variations in the capillary diameter would undoubtedly create differential freezing points within a capillary. It must also be remembered that even should the above factors be non-existent, freezing takes a finite length of time after the first ice crystals are formed. Therefore, in any freezing situation, frozen water and unfrozen water will exist in the same cavity for the length of time required for the ice crystals to fill the void



FIGURE 1

Section of Bridge Deck with Light Freeze-Thaw Scale



FIGURE 2

Close up of Figure 1

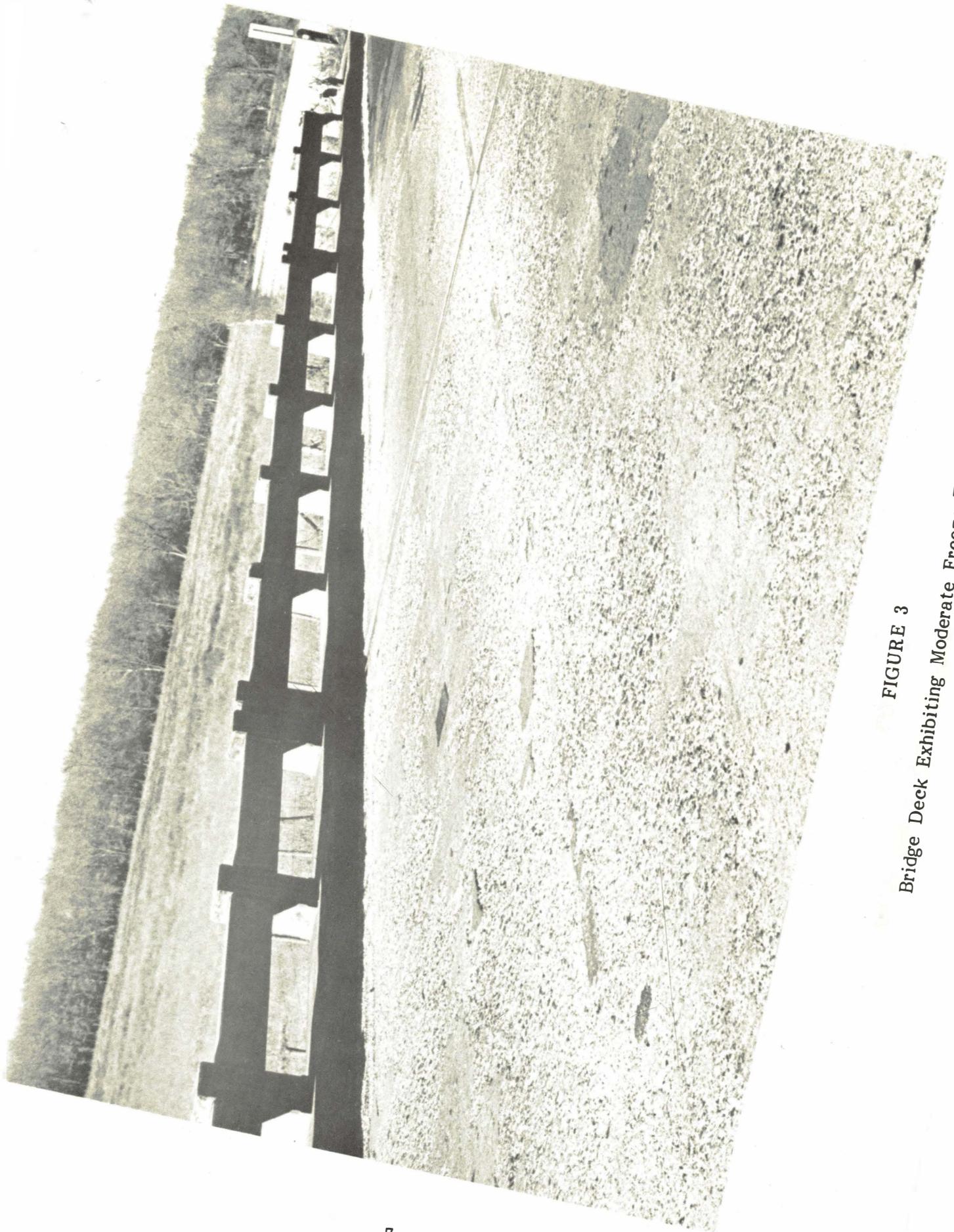


FIGURE 3
Bridge Deck Exhibiting Moderate Freeze-Thaw Scale

and/or displace excess fluid. When an ice crystal forms in a capillary, unfrozen water is forced away from the area because of the increased volume and if the capillary is over a certain critical saturation (which is a function of the amount of ice formed) hydraulic pressures will be created against the walls of these capillaries. These pressures can be quite large; in fact, ice freezing at 13.5°F (-10.3°C) can create hydraulic pressures of 10,000 psi.¹ Of course this force is much greater than the tensile strength of concrete. The term "critical saturation" is defined as the maximum water content that an individual capillary void can attain before hydraulic pressures are generated. This critical saturation is 91.7 percent by volume, since if all the water within the capillary froze, the capillary would then be just full.

In the event that one or any number of capillary voids attain a saturation greater than the critical saturation, then as freezing progresses, the unfrozen water will be forced ahead of the advancing ice crystal. This water encounters the gel wall surrounding the capillary void and hydraulic pressures are created with magnitudes that are functions of the impedance to flow through the gel matrix and the rate of advancement of the ice crystal. The impedance to flow of the onrushing water is a function of the permeability of the gel and how far through the gel this water must travel before it encounters a point of relief. This point of relief can be either the boundary of the concrete or a void that is either empty or only partially filled with water. Air entrainment voids fulfill this pressure relieving role admirably.

2. Capillary ice growth theory - The second theory proposed to account for the destructive action of freezing on concrete is the so called capillary ice growth theory.² Suppose that a sample of concrete has reached a temperature at which the water in the capillary voids freezes. The water in the gel pores is also at this same temperature, but because of the smallness of the gel pores the water trapped within them will not freeze. This situation of frozen and unfrozen water at the same temperature leads to a thermodynamic imbalance between the water

contained in the two voids: the unfrozen water possessing higher free energy than the frozen water. This difference in free energy creates forces that enable the gel water to migrate to the capillary where it freezes thus causing the ice crystal to grow. It has been estimated that for saturated gel pores at $23^{\circ}\text{F}(-5^{\circ}\text{C})$ the hydraulic pressure created between the capillary ice and the solid gel would be approximately 1200 psi.³ See Figure 4.

The same thermodynamic imbalance exists between frozen water contained in air entrainment voids and gel water as exists between water frozen in capillaries and gel water. Therefore the gel water would tend to migrate toward entrained air voids as well as capillary voids, however as previously pointed out, air entrainment voids are usually not completely filled with water and therefore can act as relief points.

In addition to the tendency to fill capillary voids that this mechanism accounts for is the very fact that the thermodynamic pressure gradient exists in opposition to the hydraulic pressure created from freezing within the capillary.

3. Osmotic Pressure Theory - The third theory concerns hydraulic forces created by osmotic pressure.

Let us assume that prior to freezing, water in capillary and gel pores contain an equal percentage of dissolved salts. These salts can be $\text{Ca}(\text{OH})_2$, NaOH , KOH , or NaCl and CaCl_2 or combinations of any or all of the preceding. When freezing begins in the capillaries the portion of the water not frozen attains a higher concentration of the above salts. This higher concentration then imbalances the concentration equilibrium that existed between capillary water and that in the surrounding gel pores. This imbalance leads to the creation of osmotic forces attempting to drive the pore water (lower concentration) to the capillary water (higher concentration). This pressure by itself would be considerable but it must be remembered that it, like the thermodynamic pressures, are in the opposite direction to the capillary hydraulic pressures and is therefore additive. See Figure 4.

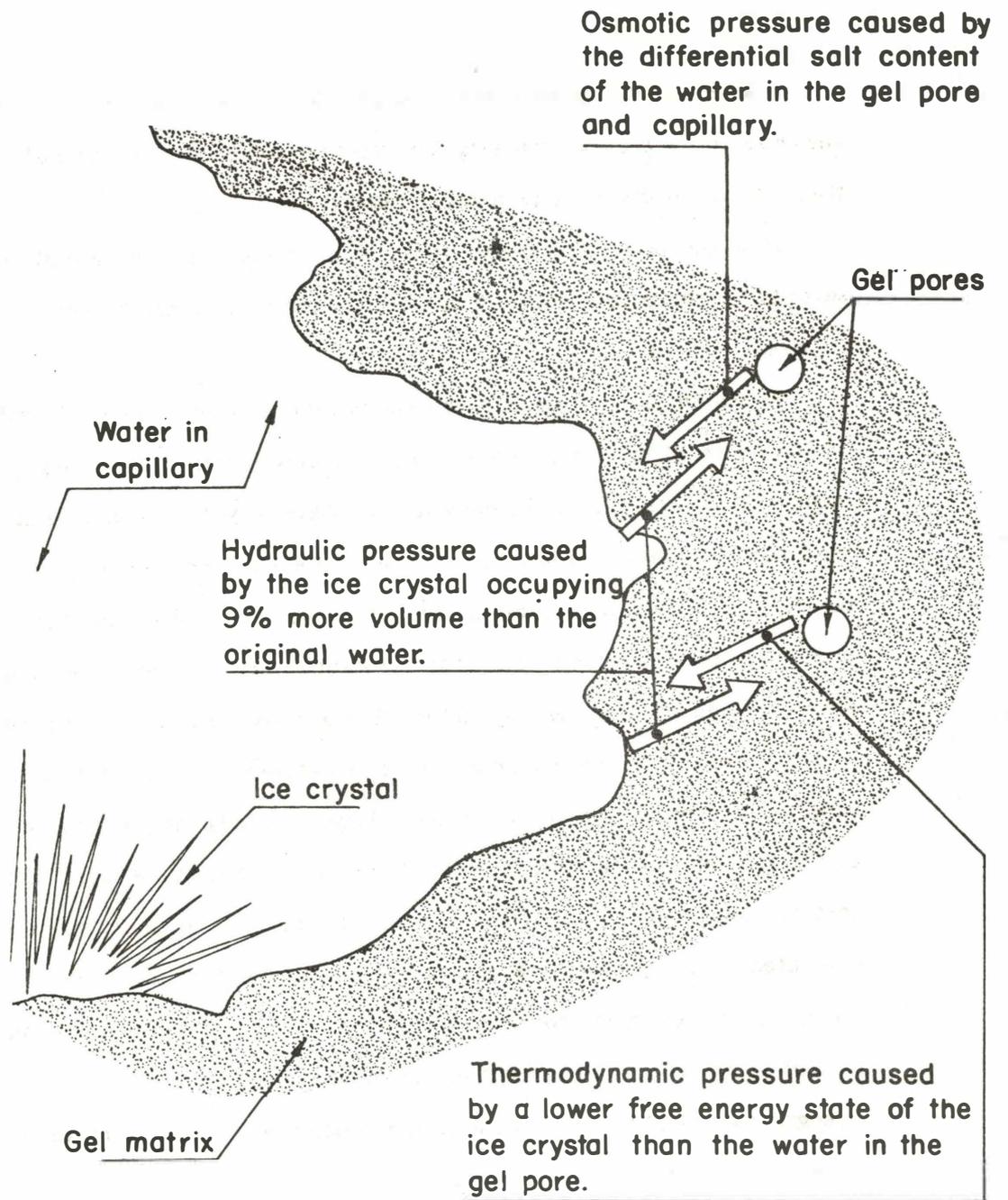


FIGURE 4

Theory Proposed to Account for Freeze-Thaw Scale

The role that air entrainment voids play in relieving osmotic pressure is analogous to the pressure relieving function they provide in the previously discussed theory of thermodynamically induced pressure.

It is the osmotic pressure theory that accounts for the greatly increased severity of freeze-thaw scale encountered in concrete to which deicing chemicals have been applied.

As stated before, freeze-thaw scale is very probably a result of combinations of the above phenomena, with perhaps unknown factors also taking part. As mentioned in each hypothesis, entrained air voids played a major role in relieving the hydraulic pressures created within the concrete matrix. Properly air entrained concrete can eliminate freeze-thaw scale in all but the most severe exposures. The primary factors that contribute to the protection afforded concrete by entrained air is the void size and spacing factor. The bubbles must be of sufficient size to permit the capillary cavities and gel pores to preferentially available water through capillary action; but they must not be so large as to adversely effect the strdnngth of the concrete. Their spacing factor is also quite critical. Klieger⁴ determined that hydraulic pressures sufficient to exceed the strength of the concrete would be generated if the path of the forced water was greater than about 0.010 inch. Entrained air reduces the compressive and flexural strengths of concrete an amount equivalent to double the percent of air entrained (generally about a ten percent strength reduction) but may increase the resistance of such concrete to freeze-thaw scale as much as 1,000 percent.

AGGREGATES

Aggregates Exhibiting Frost Susceptibility

Aggregates exhibit a freeze-thaw susceptibility similar to that of concrete paste. The primary difference between the freeze-thaw problems of concrete and that of aggregate is the result of variation in the size, number, and distribution of pore voids between the two.

The major evidence of frost susceptible aggregate in Oklahoma is in the form of popouts and surface pitting. This is not to say that frost susceptible aggregates have not caused structural disintegration of concrete in Oklahoma, but, because these aggregates were usually cast in unair-entrained concrete, their role in this disintegration was masked. Researchers in other states have cited examples of bulk disintegration of concrete traced directly to the frost susceptibility of the aggregate. Fortunately the aggregate used thus far in Oklahoma has not exhibited this malady.

Popouts can be identified by their characteristic conical shape. They result from the fracture of aggregate particles near the surface of a structure resulting from the action of the freezing of water trapped within the aggregate pores. A fragment of concrete is fractured from the surface as a result of the internal fracture of the aggregate. The resulting fractured particle is conical in shape and contains a piece of the fractured aggregate at the apex. The remainder of the offending aggregate remains in the concrete. In Oklahoma the offending aggregate particles are generally one of the two types, a chert of relatively low specific gravity or a shale particle.

Figure 5 illustrates the physical mechanism of a popout. Figure 6 pictures a bridge deck with light concentrations of popouts.

Certain shale particles, particularly the softer less dense shales, may not have enough strength to create a popout but instead may simply crush within themselves during freezing and when near the surface create an irregular void similar to that created by a mud ball. This phenomena is referred to as pitting. Figures 7 and 8 depict light pitting of a bridge deck.

(The creation of hydraulic pressures ahead of an advancing ice front is the primary mechanism promulgated to account for freezing and thawing deterioration of aggregate.⁵⁾ Some aggregates have the ability to establish osmotic and thermodynamic imbalances between the water contained within the aggregate and

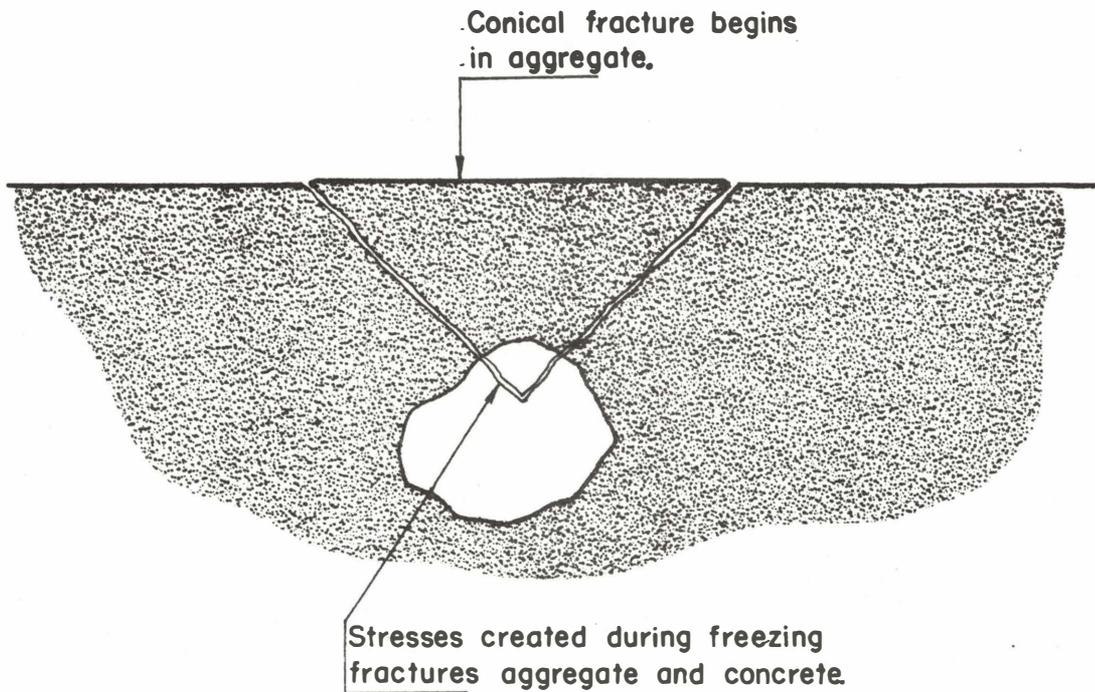


FIGURE 5
Formation of a Popout

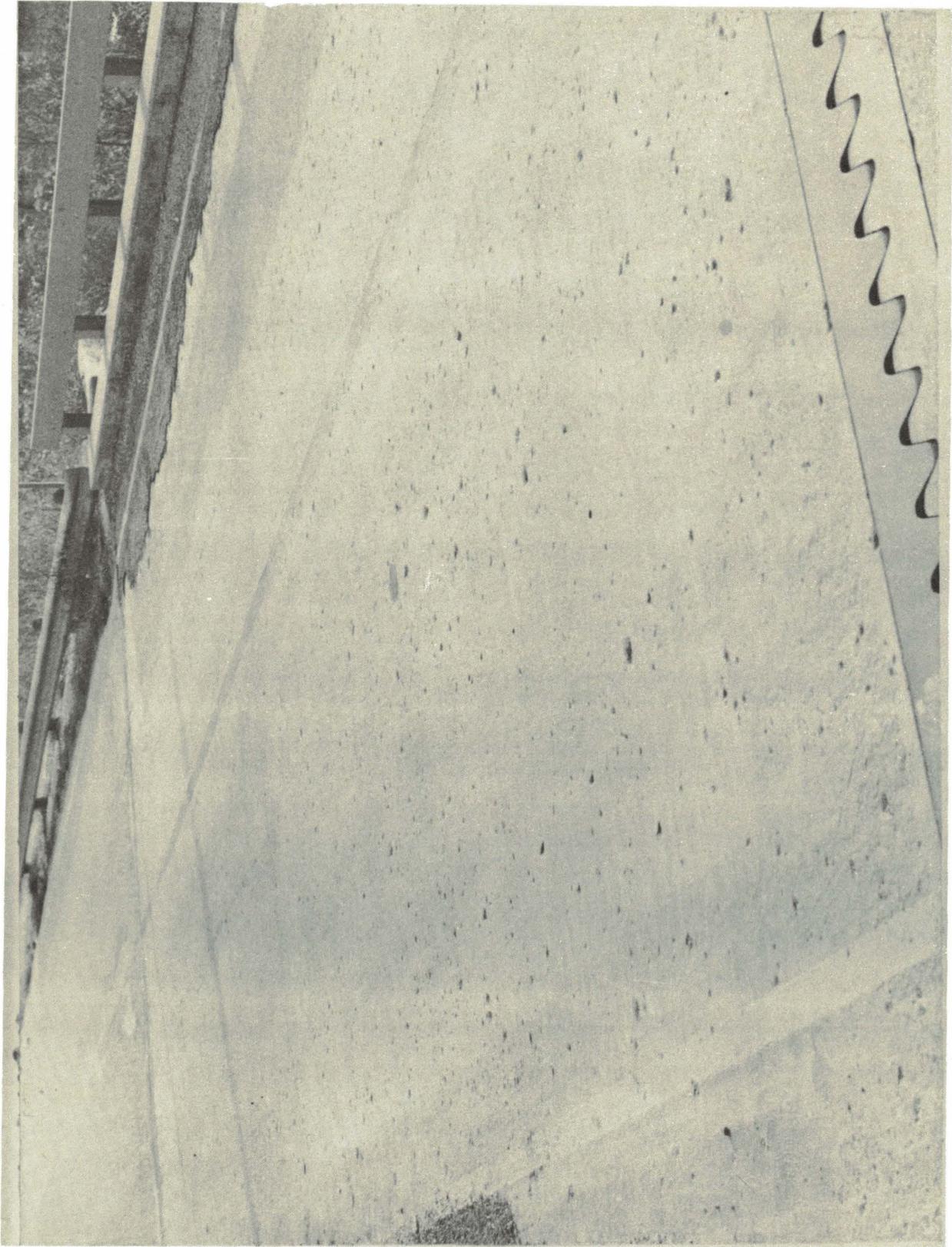


FIGURE 6
Light Concentration of Popouts



FIGURE 7

Deck Poured with Concrete Containing Shale Coarse Aggregate



FIGURE 8

Close up of Figure 7

that contained in the surrounding paste. This is analagous to the imbalances created in concrete paste discussed in the previous section. It is felt, however, that these last two forces play a relatively minor role in the deterioration process.

It has been found that the frost susceptibility of an aggregate in concrete is dependent on porosity, permeability, and size of the aggregate. Additional affecting factors are permeability and air content of the surrounding concrete paste and the freezing rate.

1. Aggregates with high porosity and low permeability tend to fracture when frozen in a condition exceeding the critical saturation. Critical saturation is again defined as water occupying over 91.7 percent of the available void space within an aggregate.

2. Aggregates with high porosity and high permeability tend to expell unfrozen water into the surrounding paste and fracture the paste or paste aggregate bond.

3. The freezing rate has the expected affect on aggregate; the higher the freezing rate the greater the potential for destruction.

4. The permeability and air content of the paste in the near vicinity of the aggregate particles affect the magnitude of hydraulic pressure created when water is forced from the aggregate during freezing.

5. The larger the aggregate, the greater the frost susceptibility. This may be because smaller aggregate particles have greater surface area per volume to expel unfrozen water or that water escaping in front of a growing ice crystal has a shorter average distance to travel before it can find relief at the aggregate boundry. Whatever mechanism is responsible for this difference in frost susceptibility, it is widely accepted that the freeze-thaw performance of an aggregate can be improved markedly by reducing the aggregate size.⁶

Many tests have been developed to delineate frost susceptible aggregates; however, all have the inability to accurately predict the field performance of

aggregates. Several of the better tests can arrange the aggregates in the order of their relative frost susceptibility. It would seem advantageous in Oklahoma to apply one of the better aggregate tests (possibly the Powers' one cycle test⁷) to aggregates of known performance and attempt to categorize sources of aggregates as to their suitability.

This proposed categorization need not entail the condemnation of any aggregate sources because they proved unsuitable for bridge deck use. Because of their high cooling rate and generally poor drainage bridge decks create quite unfavorable conditions for aggregates. For these reasons aggregates performing poorly in bridge deck use may be quite adequate in other types of concrete where ambient conditions are not so harsh. In addition, aggregates from marginal pits could be resized to take advantage of the increase in frost resistance with reduction in size. Only a minority of aggregates sources would have to adopt elaborate and expensive separation processes to provide satisfactory performance.

A report by Nevels⁸ lists the major aggregate sources in Oklahoma and their performance record in bridge deck usage. Care should be exercised when using aggregate from the poorly performing pits in bridge deck concrete or any portland cement concrete that would have the opportunity to reach critical saturation.

Pitting and popouts can be controlled by the judicious choosing of the aggregate sources to be used in bridge decks. Should economic necessity require the use of a poor performing aggregate, heavy media separation or acceptance of a finer gradation for coarse aggregate may permit acceptable performance from the marginal sources.

SPALLING

Spalling is a delamination or separation of the upper deck face away from the underlying concrete. This separation is the result of pressure created by the build up of corrosion products on the upper mat of reinforcing steel. In its early stages the spalled or separated area is roughly "V-shaped" on the spalled face with the

point of the "V" lying along the top of a reinforcing bar and the two lines forming the "V" are cracks that radiate from the bar to the deck surface. See Figures 9 and 10. When several adjacent bars are corroding, these radiating cracks may sometimes interconnect and form a delaminated face that is roughly horizontal and at the level of the upper rebar mat. The freezing of water in these cracks and the pounding of traffic soon break up these separated areas and leave spalls in the deck that can be several feet in diameter and up to two inches deep. See Figures 11 and 12.

Spalling of concrete due to corroding reinforcing steel is the most insidious, most destructive, and most difficult to control of all the deterioration types. Studies of this subject have been in existence for at least half a century but it was not until the recent widespread use of deicing salts that the problem reached the monumental proportions that it has now assumed. The mechanism of the deterioration process is now well understood, at least on the macro level. The following is an explanation of the mechanism of spalling of reinforced concrete as it is presently understood.

Steel imbedded in ordinary sound concrete will not corrode because of a phenomena called passivation. Passivation is a peculiarity of some metals that inhibits them, under certain circumstances, from corroding. This phenomena is shared by iron, chromiun, nickel, titanium, and their alloys. Iron and many of its alloys are passivated in highly alkaline environments. The degree of alkalinity of an environment or solution is measured by determining the concentration of the hydrogen ion. The negative logarithm of the hydrogen ion concentration is called pH and numbers from zero being very strongly acid to 14 being very strongly alkaline. Concrete has the required high alkalinity for passivating steel, generally possessing a pH between 12 and 13. This high pH is attributed to the free $\text{Ca}(\text{OH})_2$ generated through the hydration of cement. The passivation of steel is characterized by the formation of a tightly bound layer of metal oxide on the

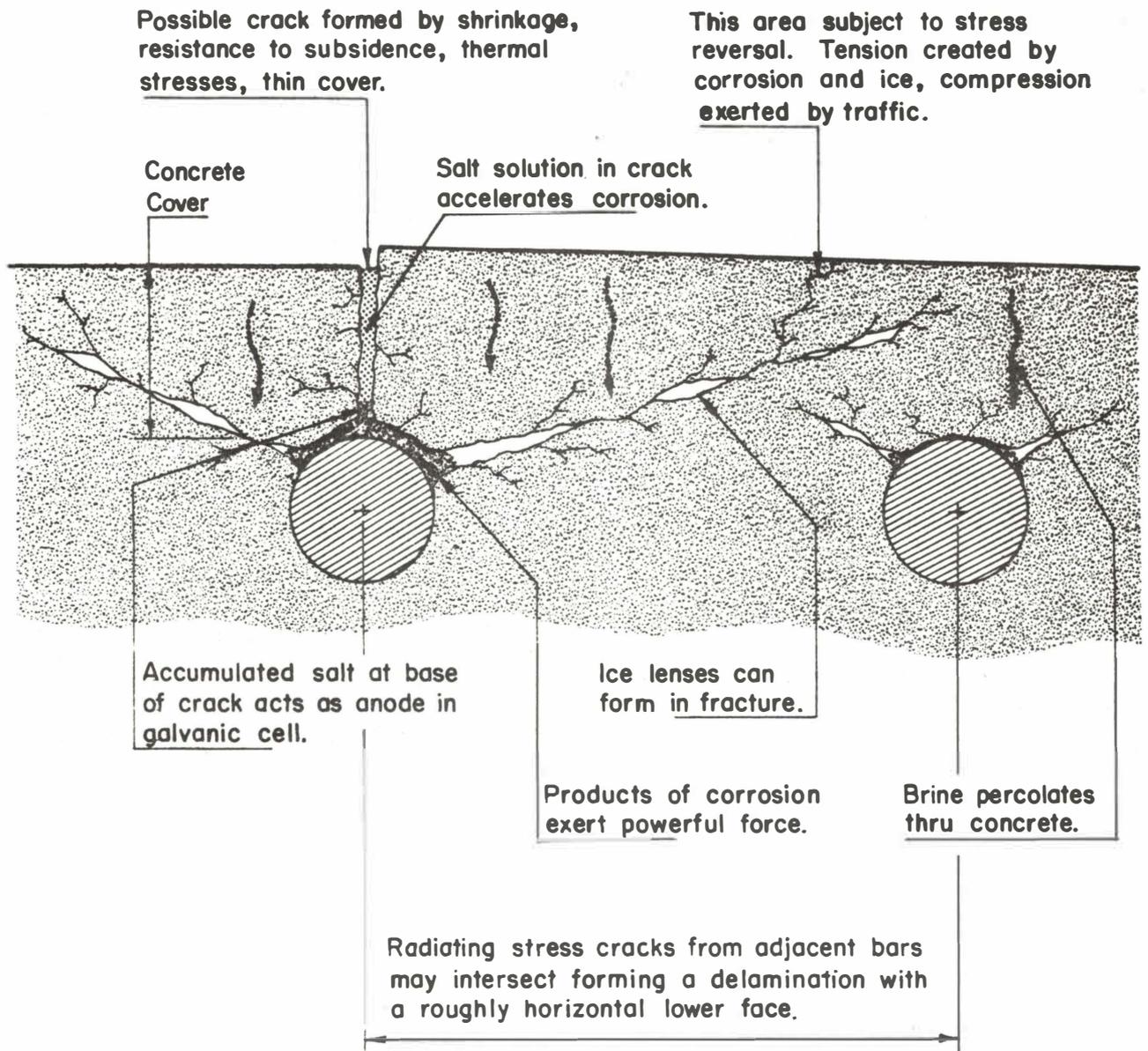


FIGURE 9

Factors Influencing and Accompanying Spall Formation

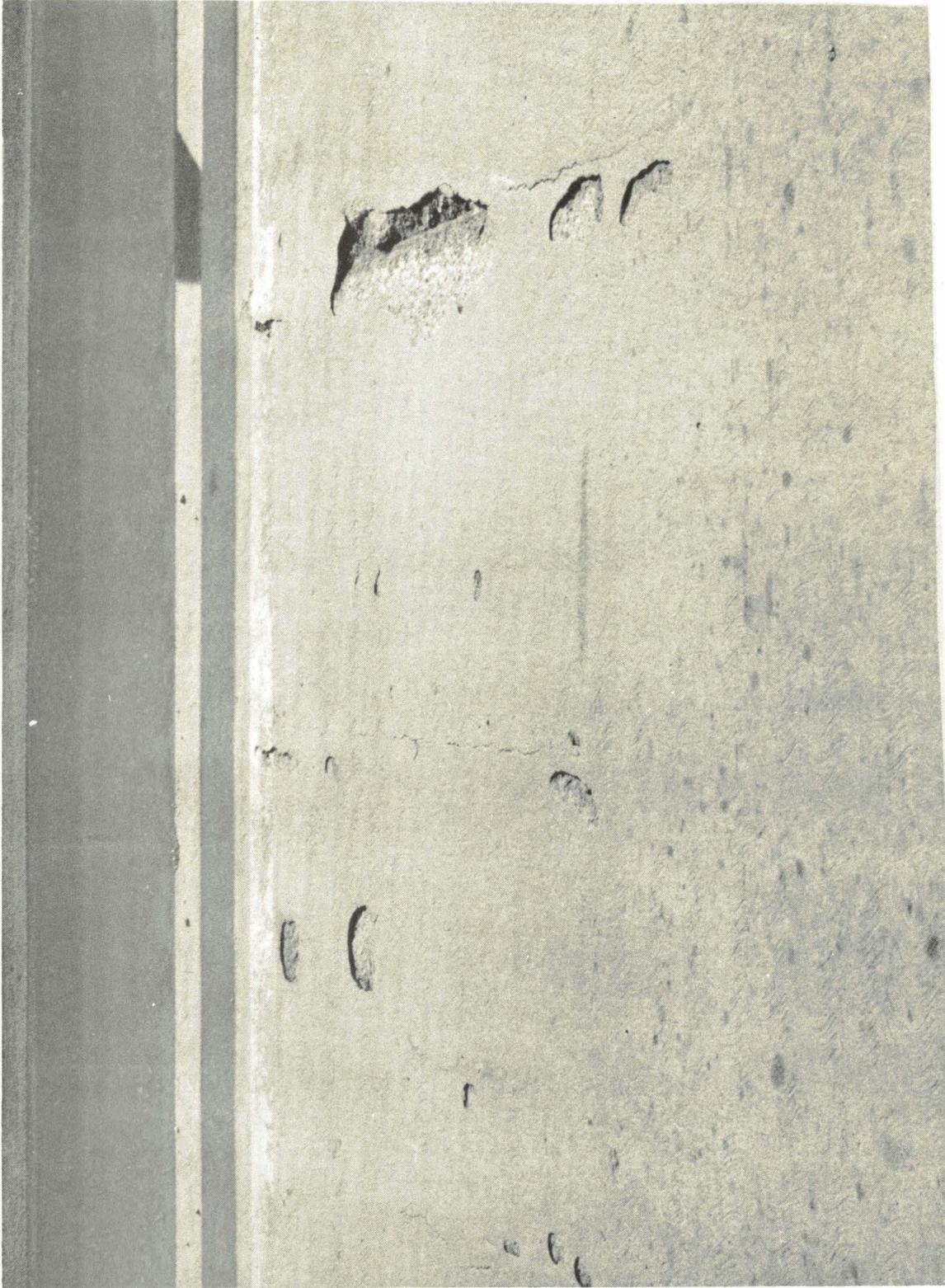


FIGURE 10
Single Bar Spall



FIGURE 11
Multibar Spall



FIGURE 12

Multibar Spall



FIGURE 12

Multibar Spall

surface of the steel. There is some conjecture concerning the make-up of this film with some researchers contending it is a layer of a single compound, Fe_2O_3 , while others assert it is a dual layer comprised of an inner layer of Fe_3O_4 and an outer layer of Fe_2O_3 . Total thickness of this layer or layers is estimated to be 40 to 50 anstroms.

This passive film is attached and destroyed by the chloride ion (other anions also attack these films but they are not pertinent to this discussion). During deicing operations chloride containing salts such as NaCl, and in some cases CaCl_2 , are broadcast on the surface of the bridge decks. These deicing salts solvated in water penetrate through the concrete to the level of the reinforcing bars where they attack the passive film formed on the steel. See Figure 13.

The mechanism through which the chloride ion attacks and renders this passive layer ineffectual is not completely understood; however, work done by Gouda⁹ suggests a finite level of chloride ion exists below which corrosion does not occur. This threshold level increases with increasing pH up to a pH of 13.5. At this point the chloride threshold reaches a maximum. This suggests that chloride induced depassivation will take place irregardless of the pH as long as sufficient chloride ion is present. Work done by Berman,¹⁰ et. al., indicates that the addition of NaCl to saturated solutions of $\text{Ca}(\text{OH})_2$ and cement extract reduces the pH of these solutions. The inference from the above research is that chloride attack on the passive films formed on steel is comprised of two actions; (1) direct attack on the film and (2) reduction of pH, making the film more vulnerable to attack. Chlorides also have a third detrimental effect in that the addition of chloride salts markedly reduces the dielectric strength of concrete allowing higher corrosion currents to flow at a given potential difference.

From the above discussion it would seem that each concrete would have its own chloride content threshold. Highly impervious and high alkali concretes should

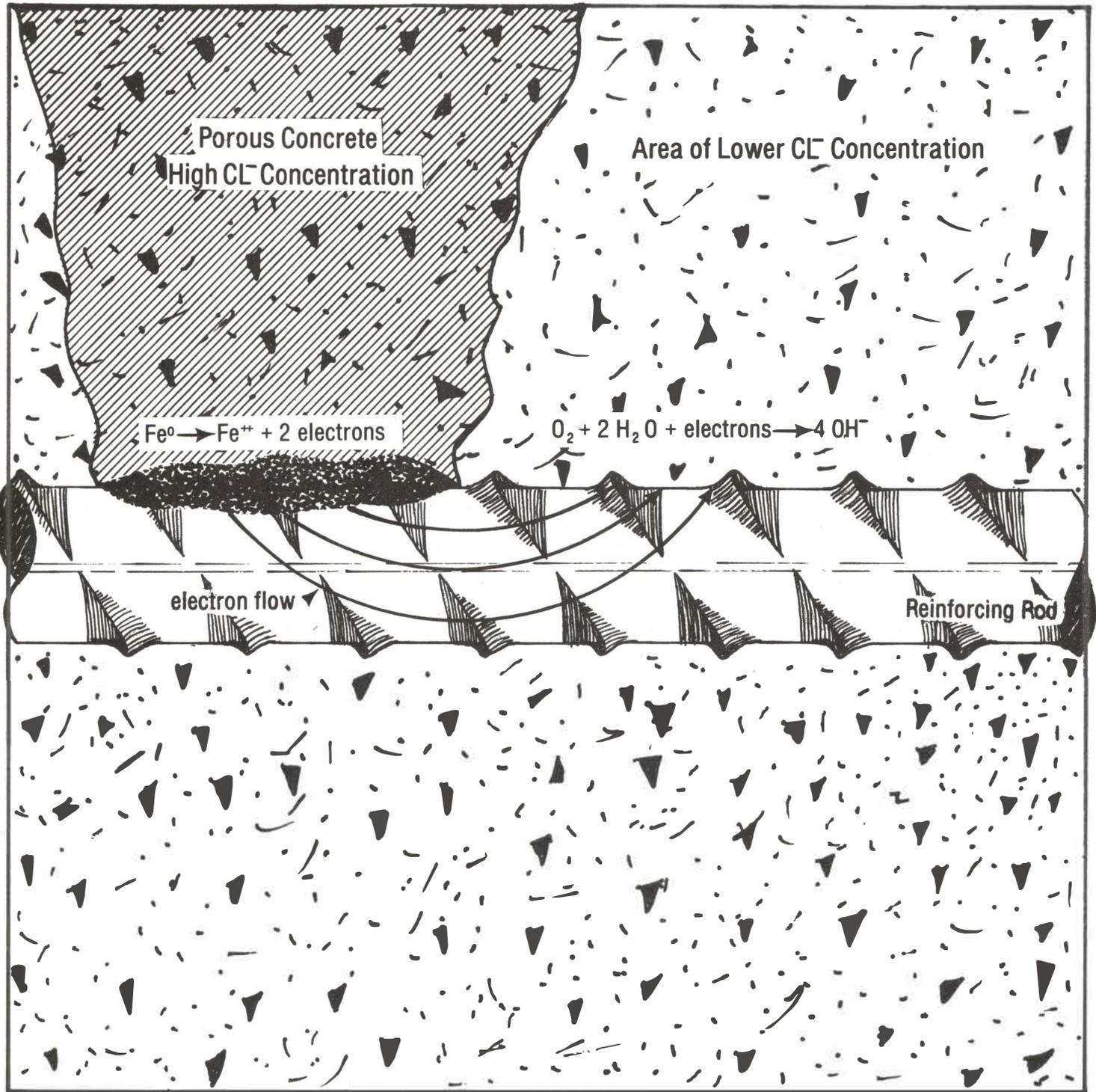


Figure 13

Reinforcing Steel in Concrete Containing unequal concentrations of Chloride ion

have higher thresholds than more porous concretes with low alkali content. However, even the most resistant of concretes cannot completely protect imbedded steel from chloride attack but may merely delay this attack until a higher concentration of chloride exists.

The above discussion lists two variables of concrete that may account for the fact that some areas of a bridge spall while other areas do not or, why some bridges spall while others with the same record of salt application do not. There are additional factors that affect the length of time that a concrete bridge deck can passivate its imbedded steel. They are porosity, age, integrity of concrete (cracking), concrete cover over the seal, pH (initial and ability to maintain pH), and amount of salt applied to the structure.

1. Porosity - more porous concrete will allow a more rapid intrusion of water-born chlorides, as well as oxygen, Thus hastening the onset and rate of cororrion.
2. Age- older concrete becomes less porous as a result of a greater degee of hydration. The possibility also exists that the passivating layer on steel may become thicker and more continuous with time; therefore, providing a greater degree of protection for the reinforcing.
3. Integrity of concrete - Loss of integrity through cracking provides ready avenues for chloride salts to penetrate to the rebar level. Cracks also allow the ingress of carbonic acid formed from CO_2 and water in the atmosphere. Carbonic acid leaches $\text{Ca}(\text{OH})_2$, KOH , and NaOH out of the paste thereby reducing the concrete's capacity to maitain a high pH.
4. Concrete cover over the steel - Data presented later in this report indicates a reduction of chloride content of approximately 40 percent for each additional inch of concrete cover over the rebars. Additional cover obviously would have the effect of delaying the onset of corrosion.
5. pH and the ability to maintain pH - As previously mentioned, work by

Gouda suggests that a higher pH may permit higher concentrations of chloride before corrosion takes place. A higher pH can be obtained and maintained through higher cement ratios and the use of cements with high alkali content; although the use of high alkali cement may pose the problem of alkali-aggregate reaction.

6. Amount of salt applied to the structure - since for each concrete there exists a finite threshold of chloride concentration above which corrosion of imbedded steel takes place, it is apparent that the length of time that a concrete can passivate reinforcing steel will be inversely proportional to the amount of salt placed on the bridge deck. Though this relationship undoubtedly exists for all reinforced structures, the proportionality constant will vary based upon the factors discussed above. It should be born in mind that the salt application rate is rarely uniform across a bridge deck. Generally the driving lanes receive the heaviest or only application. However, this may be offset by mechanical redistribution of salt laden snow or slush by snow removal equipment and vehicular traffic thus providing a much longer soaking period for areas in the waterway and along the centerline. Better bridge drainage would permit some salt to be flushed from the deck through rains and the melting of snow. Artificial flushing accomplished by pump trucks should also contribute to removal of salt laden sand and debris, thereby reducing the effective salt application rate.

All of the above are factors effect the susceptibility of a structure or an area within a structure to fall victim to rebar corrosion. These factors with the exception of age and salt application, are variables that reflect quality control during construction. Poor quality control during construction has been proven to produce the expected results; shortened time to the onset of rebar corrosion.

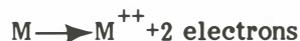
Corrosion

No discussion of bridge deck spalling would be complete without a discussion

of corrosion. Without an understanding of corrosion one cannot hope to understand the phenomena of spalling or the meaning of the data gathered during reinforcing steel potential surveys. These potential surveys provide the engineer with perhaps the most powerful tool now available for determining the extent and degree of deterioration that a bridge deck is experiencing. The following is a limited discussion of the nature of rebar corrosion and what information can be obtained from rebar potential surveys.

Corrosion is combining of a metal with oxygen, the OH^- radical, or other anions to form a compound in which the cation is at a lower energy state than in the uncombined form. All metals except the precious metals, i.e., gold and platinum, corrode spontaneously. Thermodynamically, they possess lower free energy in the combined state than in the uncombined state.

Corrosion is an electro-chemical process which means that although the process is a chemical one it also involves the transfer of electrons. This transfer of electrons and the potential difference that drives the transfer provides the best means of monitoring corrosion. The phenomena of corrosion, although quite complex, contains four essential elements, the loss of any one halting the process. The elements are (1) An Anode - Generally a metal or an area of a metal that goes from nonionic to ionic metal, very often forming a corrosion product in the process. The chemical notation for this reaction is



This release of electrons creates an excess of electrons on the anode. A chemical process of this type where the electrons are released is called oxidation; (2) A Cathode - A metal or an area on a metal where the excess electrons generated at the anode can be consumed. In alkaline solutions such as Portland Cement concrete the electrons are consumed in a chemical reaction of the type



This reaction where electrons are consumed is called reduction; (3) a metallic conductor for the electrons to flow from the anode to the cathode; and (4) an electrolyte that is continuous between the anode and cathode. This electrolyte completes the circuit allowing movement of ions or ion exchange between anode and cathode to prevent an overall charge build up at either.

Each metal has its own characteristics driving force to corrode. The relative driving force between metals can be and has been measured. A listing of metals and their driving force is called an electromotive (EMF) force listing. This listing is arrived at by immersing each metal in a very specific electrolyte and then measuring the potential difference between each metal - electrolyte cell and that of the hydrogen cell that has been arbitrarily chosen to be the standard and assigned a potential of zero. This potential difference is referred to as the cell's half-cell potential.

These metals are then listed along with their standard half-cell potential in descending order of activity or propensity to corrode. Figure 14 is an electromotive force listing. The metals near the bottom of the list corrode quite readily and are referred to as active or anodic metals. Metals near the top of the list corrode very slowly or not at all and are referred to as noble metals. The metal- electrolyte cell is actually referred to as a half-cell because it comprises one-half of the total measuring cell, the other half-cell being the standard, or reference cell. The hydrogen half-cell is rather inconvenient to use in routine field measurements; therefore, other more convenient half-cells have been developed. The most common of these cells are the copper/copper sulfate half-cell and the saturated calomel electrode. Silver/silver chloride and molybdenum/molybdenum oxide cells are also used in some instances. These half-cells offer the advantages of portability and ruggedness.

If one were to connect any two half-cells from the EMF series together with a continuous external conductor and a provision for a continuous electrolyte the

$\text{Au} = \text{Au}^{+3} + 3\text{e}$	+ 1.498
$\text{O}_2 + 4\text{H} + 4\text{e} = 2\text{H}_2\text{O}$	+ 1.229
$\text{Pt} = \text{Pt}^{+2} + 2\text{e}$	+ 1.2
$\text{Pd} = \text{Pd}^{++} + 2\text{e}$	+ 0.987
$\text{Ag} = \text{Ag}^+ + \text{e}$	+ 0.799
$2\text{Hg} = \text{Hg}_2^{++} + 2\text{e}$	+ 0.788
$\text{Fe}^{+3} + \text{e} = \text{Fe}^{+2}$	+ 0.771
$\text{O}_2 + 2\text{H}_2\text{O} + 4\text{e} = 4 \text{OH}$	+ 0.401
$\text{Cu} = \text{Cu}^{+2} + 2\text{e}$	+ 0.337
$\text{Sn}^{+4} + 2\text{e} = \text{Sn}^{+2}$	+ 0.15
$2\text{H}^+ + 2\text{e} = \text{H}_2$	0.000
$\text{Pb} = \text{Pb}^{+2} + 2\text{e}$	- 0.126
$\text{Sn} = \text{Sn}^{+2} + 2\text{e}$	- 0.136
$\text{Ni} = \text{Ni}^{+2} + 2\text{e}$	- 0.250
$\text{Co} = \text{Co}^{+2} + 2\text{e}$	- 0.277
$\text{Cd} = \text{Cd}^{+2} + 2\text{e}$	- 0.403
$\text{Fe} = \text{Fe}^{+2} + 2\text{e}$	- 0.440
$\text{Cr} = \text{Cr}^{+3} + 3\text{e}$	- 0.744
$\text{Zn} = \text{Zn}^{+2} + 2\text{e}$	- 0.763
$\text{Al} = \text{Al}^{+3} + 3\text{e}$	- 1.662
$\text{Mg} = \text{Mg}^{+2} + 2\text{e}$	- 2.363
$\text{Na} = \text{Na}^+ + \text{e}$	- 2.714
$\text{K} = \text{K}^+ + \text{e}$	- 2.925

FIGURE 14
Electromotive Force Series

metal lower in the listing would corrode (anode) while a reduction reaction would occur on the surface of the metal higher in the listing (cathode). Should a high impedance voltmeter be connected in series between the half-cells, the voltage read would be the algebraic difference between the two standard half-cell potentials. Figure 15 depicts such a cell coupling the standard half-cells of iron and copper through a porous membrane and an external conductor. In this instance Fe, being lower (more negative half-cell potential) in the EMF series, would corrode while the copper would be reduced. The potential difference between these two half-cells would be $-0.440\text{V} - (+0.337\text{V}) = 0.777\text{V}$. It is this potential difference that creates the driving force for electrons to flow from the anode to the cathode. Without this potential difference and electron flow, corrosion would quickly halt. This concept of potential difference forms the foundation for understanding corrosion.

Galvanic Cells

In this discussion the coupling of an anode and a cathode through a continuous electrolyte and external conductor will be referred to as a corrosion cell. A corrosion cell such as the one discussed above that is initiated by the coupling of two dissimilar metals is called a galvanic corrosion cell. Galvanic corrosion cells are one of the two major types of corrosion cells. The second type is the result of differences in electrolyte surrounding one metal and are called electrolytic cells. These corrosion cells will be discussed later. Although galvanic corrosion cells are created when dissimilar metals are connected in an electrolyte, galvanic corrosion cells very often occur on the surface of a single metal piece. This occurs because of the non-homogeneity of most metals. Metals, even nearly pure ones, are not homogeneous. They have microscopic as well as macroscopic surface differences. These non-homogeneities create galvanic corrosion cells. The following is a partial listing of surface non-homogeneities that can create galvanic

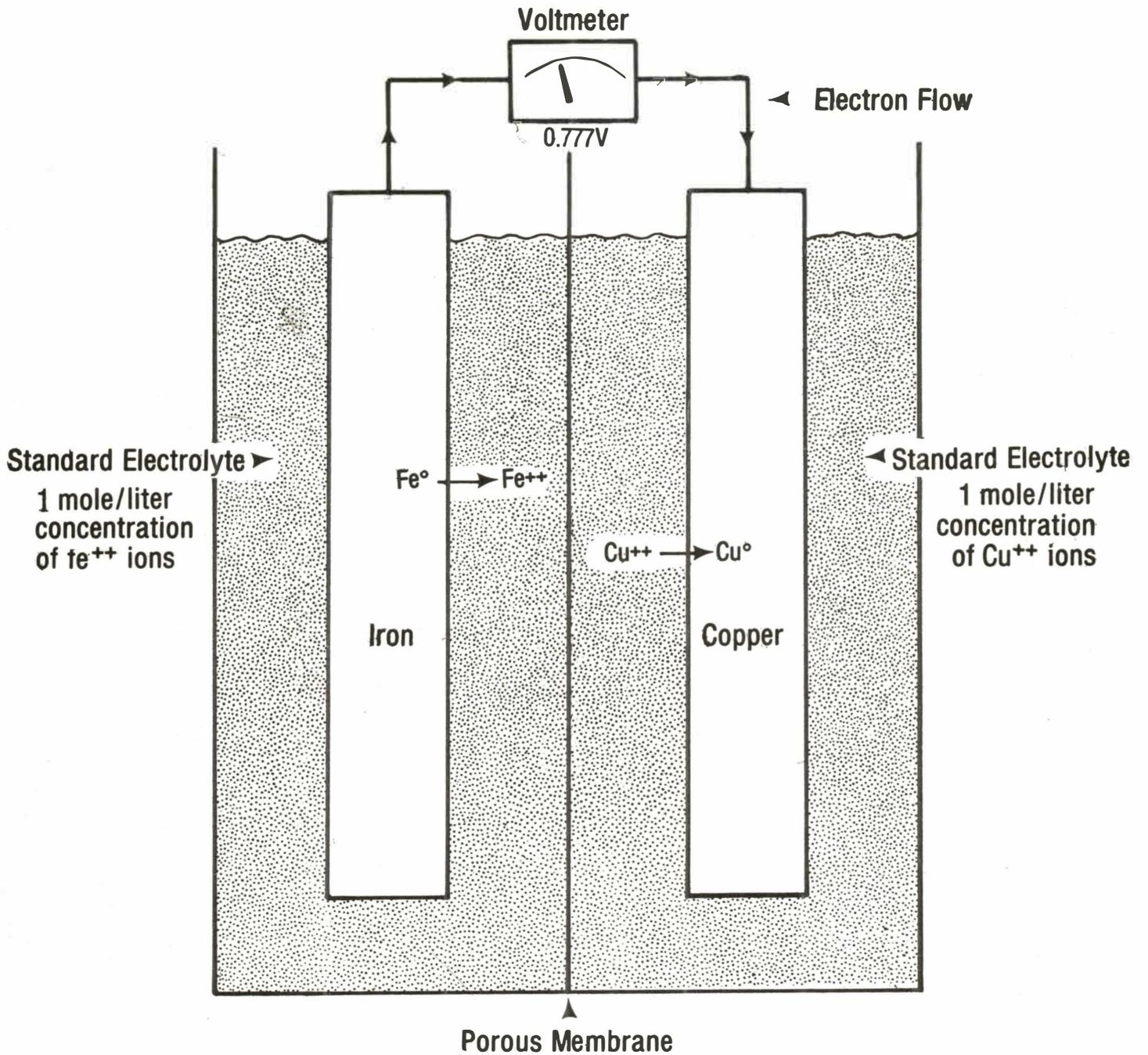


Figure 15

Standard Half-Cells of $Cu^{\circ} \rightarrow Cu^{++}$ and $Fe^{\circ} \rightarrow Fe^{++}$ Forming a Galvanic Cell

corrosion on a single metal surface. Areas on a hot rolled metal bar that are covered with mill scale are cathodic to other areas on the same bar that do not possess this covering. Bright metal surfaces are anodic to surfaces on the same metal that have a light corrosion covering. Stressed areas of metals are anodic to unstressed areas on the same metal. Most objects are made from alloys or mixtures of metals so chosen to provide the physical characteristics required of that object. This non-homogeneity quite naturally gives rise to galvanic corrosion cells. As an example of this type of cell, steel pieces are microscopically composed of grains of relatively homogeneous ferrite surrounded by grain boundaries containing ferric carbide, sulphur, and other constituents that give rise to microscopic galvanic cells in the presence of an electrolyte. These are some of the ways galvanic coupling is accomplished on the surface of only one or like metals.

Electrolytic Cell

The second form of corrosion cell is the electrolytic or differential concentration cell. As pointed out in the discussion of the EMF series, connecting any two metal half-cells together that possess different cell potentials will theoretically create a corrosion cell. With dissimilar metals and their characteristic half-cell potentials, their connection in galvanic cells and resultant corrosion is relatively easy to understand. Differential concentration cells are little less straightforward. In the development of the EMF series the relative ranking of the various metals is dependent upon their standard half-cell potential. It was found that this potential varied widely with different electrolytes; therefore, the standard electrolyte was established. This standard electrolyte has an ionic concentration of one molecular weight per liter of oxidized species of the metal electrode in question. For instance the $\text{Fe}^0 \rightarrow \text{Fe}^{++}$ standard half-cell electrolyte contains one mole per liter Fe^{++} ions. The $\text{Cu}^0 \rightarrow \text{Cu}^{++}$ half-cell electrolyte contains one mole per liter Cu^{++} ions and so on. The change in half-cell potential, with varying ionic concentration of the oxidized metal in the electrolyte, can be determined by the Nernst equation.

$$E = E_o + \frac{2.3 RT}{nF} \log \frac{a_{\text{oxid}}}{a_{\text{red}}}$$

where: E is the actual half-cell potential, E_o the standard half-cell potential from the EMF series, R is the gas constant, T is absolute temperature, n is the number of electrons transferred, F is the faraday constant and a_{oxid} and a_{red} are the activities (concentrations) of oxidized and reduced species of the metal. In essence what the Nernst equation states is that the electrode potential of a metal is directly proportional to the Logarithm of the concentration of the ions of that metal in the surrounding electrolyte. Virtually any change in the electrolyte will modify the half-cell potential either by altering the solubility of the metal ions in solution or the solubility of other critical constituents such as oxygen.

In general, the electrolytes surrounding metals such as soils, concrete, sea water, etc., are much less homogeneous than the metals themselves. Therefore, one would expect to find a great deal of corrosion caused by the potential differences created between differential concentration cells on the surfaces of these metals. This is in fact the case. Differential concentrations of dissolved and suspended constituents of all types create differential concentration cells but among the most common of these are oxygen, metal ions, oxidizing agents, hydrogen ion, and certain depassivating agents such as the chloride ion.

Corrosion of Reinforcing Steel

As mentioned previously more active metals exhibit larger negative half-cell potentials. It is also true that when a passivated metal becomes active, that is, when it begins to corrode, its half-cell potential also becomes more negative. As previously mentioned, reinforcing steel imbedded in sound uncontaminated concrete is normally passive; however, with the addition of sufficient quantities of chloride salts it becomes active. Researchers^{11, 12} in the field of reinforced concrete have found when imbedded steel changes from a passive to active state that its half-cell

potential takes a large negative jump. Normally passivated reinforcing steel will have a half-cell potential of 100 to 250 mv negative to the Cu/CuSO_4 electrode. When the rebar becomes depassivated and begins to corrode the potential climbs above 300 mV and may exceed 600 mV. This fact provides the bridge maintenance supervisor with a very valuable tool. By systematically comparing the half-cell potential of the reinforcing steel in a structure to a Cu/CuSO_4 half-cell, areas of corroding rebar can be delineated. From this knowledge about the extent of corroding rebar along with other tests such as visual observation, sounding with chain drag, and chloride analysis, repair types and monies can be more accurately scheduled.

Knowing the factors causing corrosion of reinforcing steel is a step forward. Being able to determine areas of corroding reinforcing prior to the spalling of the concrete is another step forward. However the greatest challenges remain ahead. Highwaymen are challenged to find a method or methods to eliminate or control this insidious menace. Several methods are now being tried both in the field and in the laboratory. The approaches to solving the corrosion problem will be only listed here. One of these, cathodic protection, will be discussed in detail in the third part of this report.

1. Thin bonded concrete overlays
2. Forced migration of the chloride ion by electro potential gradient
3. Cathodic protection
4. Attempting to tie up the chloride ion in a compound of low solubility.

Of these repair methods, thin bonded concrete overlay is the only fieldworthy approach now usable; although, cathodic protection is now nearing field worthiness. Forced migration of the chloride ion by electropotential gradient has been used in limited field trials with some degree of success but nevertheless requires more laboratory research to overcome problem areas. The author knows of no successful attempts at tying up the chloride ion in products of low solubility in Portland Cement concrete.

RESULTS

Test Procedure

Soon after the conception of this project it was decided to develop a test method that could categorize the degree of distress in individual bridge decks. Test methods were selected that would quantify the degree of deterioration of the types previously discussed. Freeze-thaw scale, since it is essentially a surface defect, could be quantified satisfactorily by visual methods. The bridge would be examined visually and the degree and extent of freeze-thaw scale would then be plotted on a work sheet that would ultimately contain all the information obtained from our tests. It was felt that aggregate frost susceptibility might not always be evidenced by the presence or absence of surface defects such as popouts or pitting and that excessive aggregate dilation might create internal stresses that would not be proportional to the degree of distress apparent from the bridge surface. For this reason, a device was purchased to make ultrasonic measurements of concrete in situ. The device consisted of a pair of hydrophones that transmitted and received the ultrasonic signal and peripheral electronics to display (on a cathode ray tube) the received signal as a function of time. The theory being, that unsound or internally damaged concrete would be a poorer transmitter of the ultrasonic signals than sound concrete. A signal would be sent from one hydrophone, received at the other, and the time required for the sound to travel this pathway could be read out. From this data and the known distance between the hydrophones, the speed of propagation of the ultrasonic wave through the concrete could be determined. It was also suggested by the manufacturer that the amplitude and shape of the received signal could provide some information about the concrete properties. After some lengthy attempts at interpreting the data from this device it was abandoned as a routine survey tool. Its principal drawbacks were excessive set up time, degree of skill required by the operator-interpreter, and ambiguity of much of the data. It was decided to estimate degree of frost susceptibility from visual inspection of the

presence and degree of popouts and pitting. This data was plotted on the work sheet along with the freeze-thaw scale data. Testing for extent of corroding steel reinforcing was accomplished by plotting of visible delaminations and by copper/copper sulfate half-cell survey. Later, chain drag sounding was incorporated into the testing for rebar corrosion. Even later, sampling for chloride analysis became a part of the bridge testing. The results of these tests were added to the bridge work sheets. Finally a visual inspection of the bridge soffit was performed and all cracks, areas of seepage, or other problem areas were added to the work sheet. The final product was a work sheet that appeared as an aerial view of the bridge deck with the above information plotted on it. See Figure 16.

Results of Division Survey

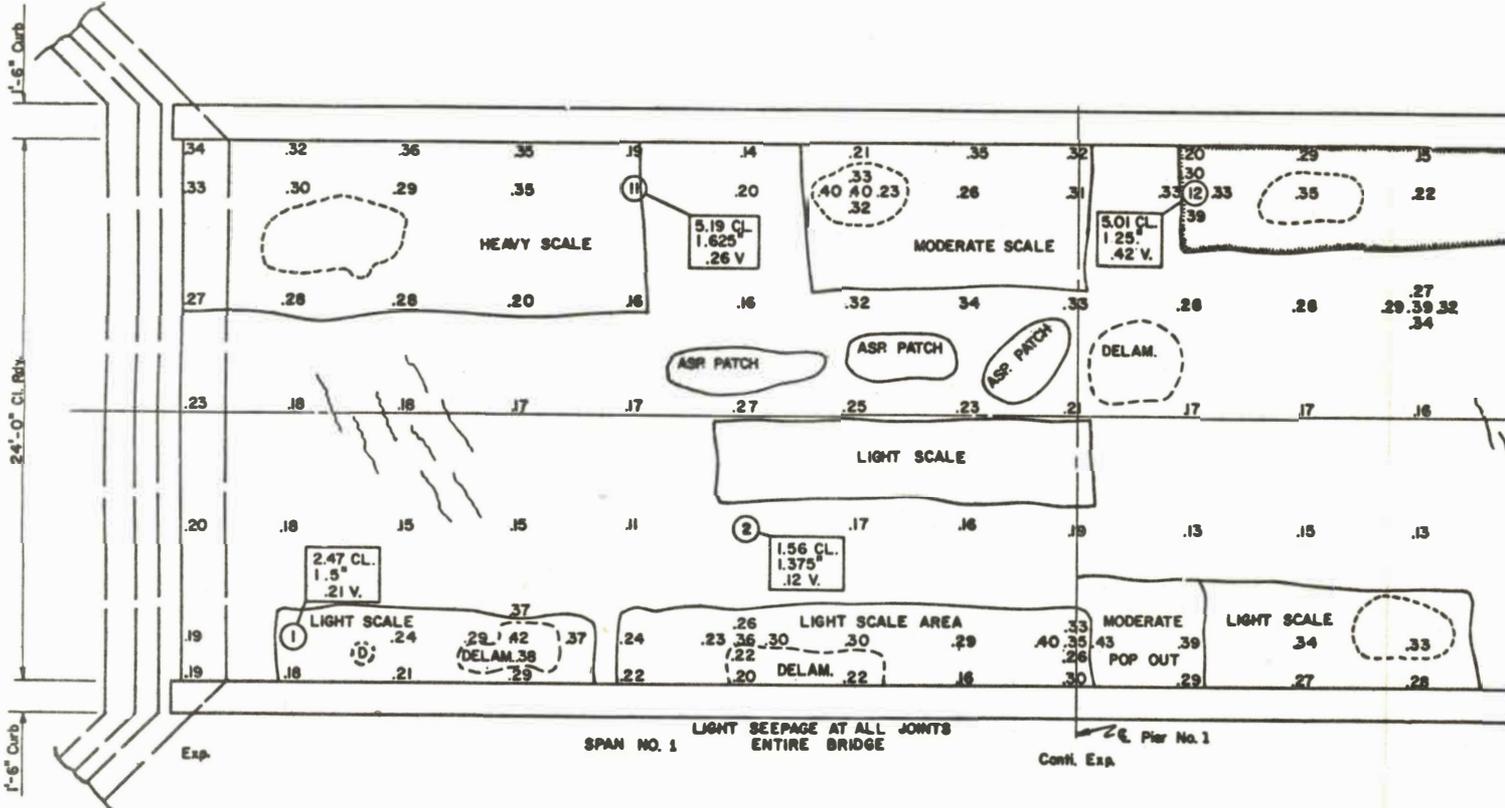
Armed with these tentative testing procedures a plan was developed to provide an opportunity to perfect our testing techniques and also discover to what degree Oklahoma's bridges had deteriorated. It was decided to choose 20 bridges from each of Oklahoma's eight field divisions to test for degree of deterioration. The selection of bridges to be included was made jointly by personnel from the Research Division assigned to this project and the individual field division engineers and assistant maintenance engineers. As a general rule, the bridges chosen included either the worst appearing bridges in the division or those having a poor appearance coupled with a critical or high traffic location. This survey was to occupy the better part of two years.

Before the survey was completed some divisions had received much more scrutiny than others. This resulted from an extensive bridge deck replacement program that was begun about a year after our survey was started. It was decided to test these structures that were to be replaced even if they were not included in the original 20 bridges chosen for each Division. The divisions that received the least attention were those that had structures overlaid with asphaltic concrete. This overlay prevented visual evaluation and also hindered the performance of the

LEGEND OF SYMBOLS

-  - Indicates areas of "Freeze"
-  - Indicates "Delaminated or Seepage" area
-  - Indicates "Crack through D"
- .23 - Indicates "Halfcell Test R"
- ⑤ - Indicates "Locations of Cl (See Note #1)"

Note #1: Information listed with test holes the tested chloride content in Lbs C.V., the depth of reinforcing steel the halfcell reading.



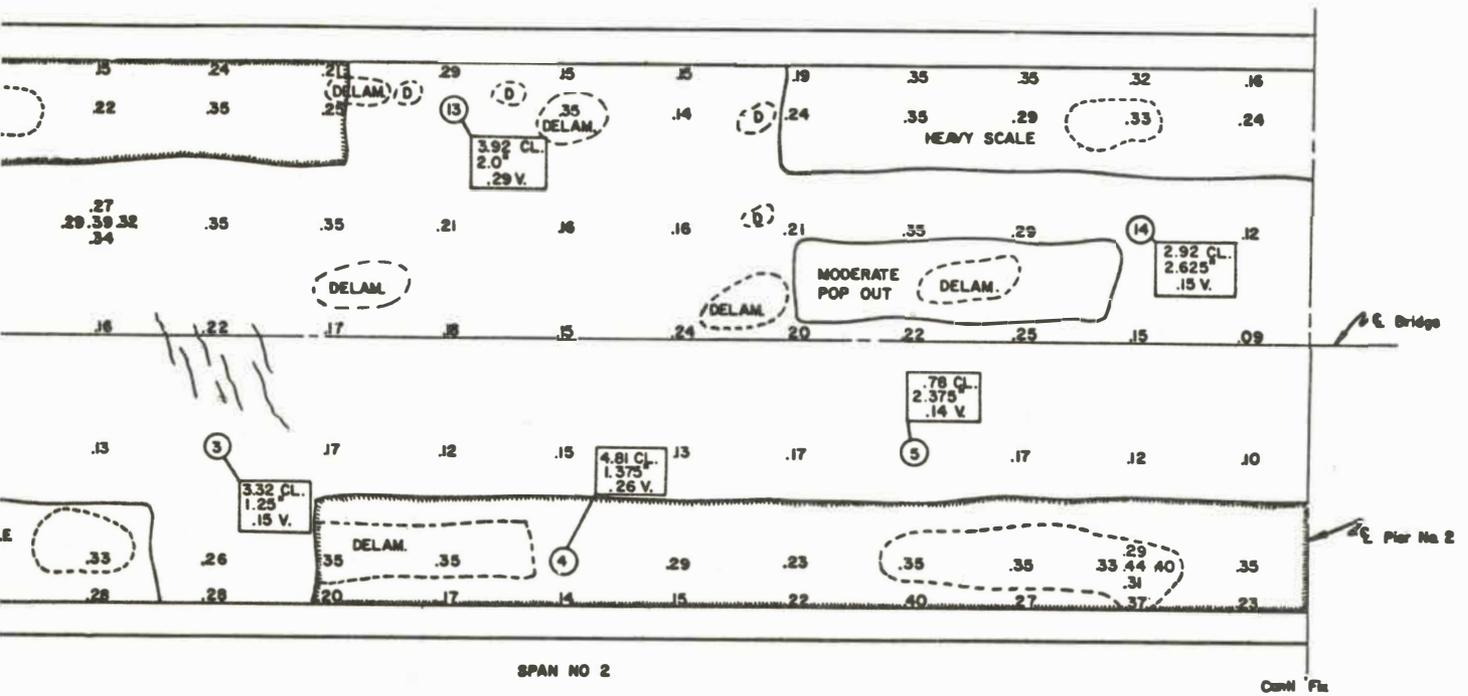
Typical Bridge Deck Wo

Figure 16

LEGEND OF SYMBOLS

- Indicates areas of "Freeze and Thaw Scale"
- Indicates "Delaminated or Spalled" area
- Indicates "Seepage" area
- Indicates "Crack through Deck"
- Indicates "Halfcell Test Reading (V vs Cu/Cu SO₄)"
- Indicates "Locations of Chloride Sampling Hole"
(See Note #1)

Information listed with test holes includes
measured chloride content in Lbs. Per
cu ft, the depth of reinforcing steel and
halfcell reading.



Bridge Deck Worksheet
Figure 16

Cu/CuSO₄ half-cell survey. The method used to survey overlaid structures with the Cu/CuSO₄ half-cell was to drill through the overlay at the grid intersections (five foot grid) and then later patch these holes. Even though the hole was small, (one-quarter of an inch) it was felt that the patching would not be 100 percent effective and might provide an avenue for deicing salt to reach the bridge deck, perhaps accelerating deterioration. For these reasons only nine bridges were surveyed in Division II and none were surveyed in Division V.

The results of the surveys in tabular form are presented in Figure Nos. 17 through 22. Very little in the way of general trends can be drawn from these data because of the less than random way in which the structures were chosen and also from the fact that 20 bridges in each division is a very small percentage of the total number which averages about 450 per division. However, some discussion of these results is in order if for no other reason than to discourage conclusion jumping.

Division I - Centered in the state from north to south, its structures in this study seem to reflect its climate. Division I structures exhibited somewhat less deterioration than did Divisions VIII, IV, and VI which have a harsher climate and somewhat more deterioration than Divisions II and VII with a milder climate.

Division I adopted two maintenance procedures that may have contributed to this relatively low deterioration. The first was the adoption of a policy to flush bridge decks after winter salt applications. The second was a reluctance to overlay bridges, but when done, the waterways were not overlaid, thus providing a space for the evaporation of rain water and melted snow. This procedure would undoubtedly contribute to fewer wet freeze-thaw cycles than would be the case with a full width overlay.

Division II - Located in the southeastern corner of the state, Division II has the mildest climate in the state. This is reflected in the generally good condition of their bridges. The incidence and extent of scale as reported in the data is biased by a series of bridges on US 69 in Atoka County that scaled soon after construction and

DIV

COUNTY	BRIDGE NO.	DATE BUILT	DECK AREA	TOTAL % SCALE	% LIGHT SCALE	% MOD. SCALE	% HEAVY SCALE	TOTAL % POPOUTS	% LIGHT POPOUTS
McINTOSH	69-46-03-X000-SB	1962	27067.5	0	0	0	0	0	0
	69-46-03-X000-NB	1962	27067.5	0.31	0.09	0.11	0.11	0	0
	69-46-03-X086-SB	1962	21667.5	4.93	4.75	0	0.18	0	0
	69-46-03-X086-NB	1962	21667.5	1.19	1.19	0	0	0.09	0.09
	9-46-14-X025	1962	28063	10.90	7.49	0.97	2.44	0	0
CHEROKEE	51-11-16-X003	1946	14826	6.39	6.39	0	0	0	0
SEQUOYAH	64-68-10-X210	1956	19076.4	0.94	0	0.94	0	0	0
	64-68-10-X206	1956	7876.4	0.05	0.05	0	0	0	0
	64-68-10-X195	1956	12356.4	2.58	2.10	0.48	0	0	0
ADAIR	59-01-06-X108	1956	11366.52	19.76	0.21	19.55	0	0	0
OKMULGEE	62-56-04-X094	1936	32278.08	0	0	0	0	0	0
WAGONER	51-73-14-X119	1955	4270	NA	NA	NA	NA	NA	NA
	51-73-14-X070	1955	3397.24	NA	NA	NA	NA	NA	NA
	51-73-14-X073	1955	23557.24	NA	NA	NA	NA	NA	NA
	51-73-14-X092	1955	22159.2	0	0	0	0	0	0
AVERAGES		20.27	18446.43	3.92	1.86	1.84	0.23	0.01	0.01
% OF BRIDGES AFFECTED				83.3	66.6	41.7	33.3	8.3	8.3

D

COUNTY	BRIDGE NO.	DATE BUILT	DECK AREA	TOTAL % SCALE	% LIGHT SCALE	% MOD. SCALE	% HEAVY SCALE	TOTAL % POPOUTS	% LIGHT POPOUTS
ATOKA	69-03-02-X078	1948	11200	26.29	24.38	1.91	0	0	0
	69-03-02-X075	1948	7280	20.10	18.45	1.65	0	0	0
	69-03-02-X066	1948	4200	34.88	34.88	0	0	0	0
	69-03-02-X065	1948	3360	21.80	21.80	0	0	0	0
	69-03-02-X077	1948	3360	37.41	32.65	4.76	0	0	0
	69-03-02-X070	1948	8120	42.54	41.01	1.53	0	0	0
	69-03-02-X111	1948	3920	32.10	12.30	17.35	2.45	0	0
	69-03-04-WX020	1951	5040	3.81	3.43	0.20	0.18	0	0
	70-07-10-X182	1939	3080	1.40	1.40	0	0	0	0
AVERAGES		28.8	5507	24.48	21.14	3.24	0.29	0.0	0.0
% OF BRIDGES AFFECTED				100.0	100.0	66.6	22.2	0.0	0.0

Figure

DIVISION ONE

% LIGHT POPOUTS	% MOD. POPOUTS	% HEAVY POPOUTS	% SPALLS	% SEEPAGE	% HALF CELLS >300mV	% HIGH HALF CELLS BY LOCATION			% Cl ⁻ SAMPLES >2 lb./yd ³	COMMENTS
						AT JOINTS	IN GUTTERLINE	IN ROADWAY		
0	0	0	0	0	6.14	5.41	0.22	0.51	NA	
0	0	0	0	0	12.80	11.07	1.59	0.14	NA	
0	0	0	0.31	0	20.94	7.82	5.94	7.18	NA	
0.09	0	0	0.07	0	7.85	7.08	0.42	0.35	NA	
0	0	0	1.31	0	0.72	0.17	0.33	0.22	NA	
0	0	0	1.15	0	6.24	4.80	0.47	0.96	NA	
0	0	0	0.11	0	0.07	0	0.07	0	NA	
0	0	0	0.04	0	0.18	0.18	0	0	NA	
0	0	0	1.81	0	58.63	6.17	10.37	25.55	NA	
0	0	0	0.07	0	4.53	2.02	2.51	0	NA	
0	0	0	0.10	0	4.76	3.05	0.77	0.94	NA	
NA	NA	NA	NA	0	0.82	0	0	0.82	NA	Overlaid
NA	NA	NA	NA	0	24.16	4.00	1.76	18.40	NA	Overlaid
NA	NA	NA	NA	0	36.87	0.92	11.83	24.11	NA	Overlaid
0	0	0	0.08	0	15.65	0.52	5.67	9.46	NA	Structure raised 1970
0.01	0.0	0.0	0.42	0.0	13.36	3.55	2.80	5.91	NA	
8.3	0.0	0.0	83.3	0.0	100.0	86.6	86.6	80.0	NA	

DIVISION TWO

% LIGHT POPOUTS	% MOD. POPOUTS	% HEAVY POPOUTS	% SPALLS	% SEEPAGE	% HALF CELLS >300mV	% HIGH HALF CELLS BY LOCATION			% Cl ⁻ SAMPLES >2 lb./yd ³	COMMENTS
						AT JOINTS	IN GUTTERLINE	IN ROADWAY		
0	0	0	0.49	0.0	1.26	0.14	1.12	0.0	NA	
0	0	0	3.80	0	1.26	0.84	0.42	0	NA	
0	0	0	5.10	0	0.34	0	0.34	0	NA	Epoxy Patches
0	0	0	4.85	0	0	0	0	0	NA	Epoxy Patches
0	0	0	0.06	0	2.67	0.89	1.78	0	NA	
0	0	0	0.21	0	0.76	0.38	0.38	0	NA	3 Panels Scaled
0	0	0	0	0	0	0	0	0	NA	
0	0	0	0.18	0	1.80	1.80	0	0	NA	
0	0	0	0	0	0	0	0	0	NA	
0.0	0.0	0.0	1.6	0.0	0.89	0.45	0.45	0.0	NA	
0.0	0.0	0.0	77.7	0.0	66.7	66.7	55.6	0.0	NA	

Figure 17

DIV

COUNTY	BRIDGE NO.	DATE BUILT	DECK AREA	TOTAL % SCALE	% LIGHT SCALE	% MOD. SCALE	% HEAVY SCALE	TOTAL % POPOUTS	% LIGHT POPOUTS	PC
OKFUSKEE	48-54-12-X055	1937	3391.2	6.02	5.19	0.83	0	0	0	
	40-54-22-NX025	1963	50100	8.07	6.85	1.18	0.04	0	0	
	40-54-22-SX025	1963	50007	0.68	0.62	0	0.02	0	0	
SEMINOLE	270-67-02-X088	1957	11228	1.89	0	1.89	0	0	0	
	59-67-26-X029	1933	7948.6	2.56	2.43	0.13	0	0	0	
HUGHES	75-32-04-X159	1961	11284	1.95	1.91	0.04	0	0	0	
JOHNSON	99-35-32-X067	1933	6042.74	11.55	5.08	6.47	0	10.16	10.16	
	48-35-20-X022	1960	4236.4	8.10	5.22	0.71	2.17	57.22	57.22	
	22-35-16-X036	1954	7308	20.61	16.90	3.61	0.10	0	0	
COAL	7-35-06-X025	1933	5827.8	0	0	0	0	0	0	
	43-15-14-X015	1956	5466	33.03	10.34	20.64	2.05	0	0	
	75-15-04-X107	1956	3060.4	40.82	11.44	6.57	22.81	0	0	
PONTOTOC	3-62-02-X000	1937	25310.4	1.50	1.38	0.09	0.03	0.43	0.43	
POTTAWATOMIE	40-63-40-SX053	1961	13425	7.89	0	2.26	5.63	0	0	
	40-63-40-NX053	1961	13425	0	0	0	0	0	0	
	40-63-40-SX061	1961	12000	0	0	0	0	0	0	
	40-63-40-NX061	1961	12000	0	0	0	0	0	0	
	40-63-40-X0503	1961	4932	25.91	20.36	2.45	3.10	0	0	
	40-63-40-SX076	1960	5624	3.38	0	0	3.38	0	0	
	40-63-40-NX076	1960	5624	0	0	0	0	0	0	
	270-63-23-X0383-EB	1961	21789	13.52	6.20	3.44	3.88	28.05	16.28	
	270-63-23-X0383-WB	1961	21789	9.84	3.76	3.60	2.48	9.86	2.44	
	35-14-06-X0657	1951	3080	NA	NA	NA	NA	NA	NA	
CLEVELAND	35-14-06-X0000-NB	1955	127310.4	0.41	0.33	0.02	0.06	0	0	
	77-14-04-X000	1938	117497.6	NA	NA	NA	NA	NA	NA	
AVERAGES		24.24	19988	8.60	4.26	2.34	1.99	4.60	3.76	
% OF BRIDGES AFFECTED				78.3	65.2	69.5	56.5	21.7	21.7	

Figure 18

DIVISION THREE

% HIGH HALF CELLS BY LOCATION

% LIGHT POPOUTS	% MOD. POPOUTS	% HEAVY POPOUTS	% SPALLS	% SEEPAGE	% HALF CELLS >300mV	% HIGH HALF CELLS BY LOCATION			% Cl ⁻ SAMPLES >2 lb./yd ³	COMMENTS
						AT JOINTS	IN GUTTERLINE	IN ROADWAY		
0	0	0	0	0.21	29.89	9.96	13.03	6.90	NA	
0	0	0	0.06	0	49.80	7.96	26.43	15.41	NA	
0	0	0	0.19	0	39.91	8.56	16.88	14.47	NA	
0	0	0	0	0	4.42	3.18	0.83	0.41	NA	
0	0	0	0.31	0	34.33	12.70	20.44	1.19	NA	
0	0	0	0.04	0	13.86	2.61	6.72	4.53	NA	
10.16	0	0	0.80	0	50.65	5.19	41.30	4.16	NA	
57.22	0	0	0	0.35	6.81	6.45	0.36	0	NA	
0	0	0	0.10	0	51.85	9.94	26.71	15.20	NA	
0	0	0	0	0	66.40	3.70	43.92	18.78	NA	
0	0	0	0.24	0.24	1.41	0	1.41	0	NA	
0	0	0	1.99	1.14	15.95	4.35	7.25	4.35	NA	Widened & Redecked 1956
0.43	0	0	1.19	0.35	18.95	10.64	7.00	1.31	NA	
0	0	0	13.73	0	44.20	4.56	23.37	11.27	40.00	
0	0	0	11.11	41.01	40.20	4.58	22.22	13.40	20.00	
0	0	0	24.91	0	56.20	2.48	33.75	19.97	50.00	
0	0	0	24.68	0	52.35	1.52	32.23	18.60	30.00	
0	0	0	0.61	8.76	28.89	3.33	23.06	2.50	76.47	
0	0	0	6.37	2.90	37.95	1.19	15.81	20.95	25.00	
0	0	0	17.48	22.07	76.00	2.82	31.77	41.41	90.00	
16.28	9.46	2.31	4.07	3.63	32.42	5.60	15.84	10.98	74.07	
2.44	3.89	3.53	1.50	1.31	22.87	6.97	9.75	6.15	51.85	
NA	NA	NA	NA	29.87	14.45	.58	8.09	5.78	20.00	Overlaid
0	0	0	0.30	0	28.91	9.00	12.78	7.13	NA	
NA	NA	NA	NA	NA	14.68	4.00	.02	2.66	NA	Overlaid
3.76	0.58	0.25	4.77	4.47	33.33	5.27	17.96	9.90	47.74	
21.7	8.7	8.7	82.6	48.00	100.0	96.0	100.0	92.0	100.0	

Figure 18

COUNTY	BRIDGE NO.	DATE BUILT	DECK AREA	TOTAL % SCALE	% LIGHT SCALE	% MOD. SCALE	% HEAVY SCALE	TOTAL % POPOUTS	% LIGHT POPOUTS	
PAYNE	1-35-60-29-WX005	1959	8439	NA	NA	NA	NA	NA	NA	
	177-60-12-X0152	1953	22470	NA	NA	NA	NA	NA	NA	
NOBLE	1-35-52-33-EX0637	1959	8190	NA	NA	NA	NA	NA	NA	
	WX0637	1959	8190	NA	NA	NA	NA	NA	NA	
	WX132	1961	10230	NA	NA	NA	NA	NA	NA	
	EX132	1961	10230	NA	NA	NA	NA	NA	NA	
	WX218	1960	9897.6	NA	NA	NA	NA	NA	NA	
	EX218	1960	9897.6	NA	NA	NA	NA	NA	NA	
	WX047	1960	7290	NA	NA	NA	NA	NA	NA	
	EX047	1959	7290	NA	NA	NA	NA	NA	NA	
	177-36-16-X093	1957	15736	NA	NA	NA	NA	NA	NA	
	1-35-36-25-WX145	1960	4940	NA	NA	NA	NA	NA	NA	
	EX145	1960	4940	NA	NA	NA	NA	NA	NA	
	WX181	1959	4560	NA	NA	NA	NA	NA	NA	
	EX181	1959	4560	NA	NA	NA	NA	NA	NA	
	WX2182	1959	18099	NA	NA	NA	NA	NA	NA	
	EX2182	1959	18099	NA	NA	NA	NA	NA	NA	
	WX2402	1959	4256	NA	NA	NA	NA	NA	NA	
	EX2402	1959	4256	NA	NA	NA	NA	NA	NA	
	WX2350	1959	5100	NA	NA	NA	NA	NA	NA	
	EX2350	1959	5100	NA	NA	NA	NA	NA	NA	
	WX1241	1959	5700	NA	NA	NA	NA	NA	NA	
	EX1241	1959	5700	NA	NA	NA	NA	NA	NA	
	CANADIAN	66-09-06-NX083	1951	4374	NA	NA	NA	NA	NA	NA
		SX083	1951	4374	NA	NA	NA	NA	NA	NA
	OKLAHOMA	1-240-55-71-SX0772	1963	5738	NA	NA	NA	NA	NA	NA
		NX0772	1963	5738	NA	NA	NA	NA	NA	NA
		NX0622	1963	5700	NA	NA	NA	NA	NA	NA
		SX0622	1963	5700	NA	NA	NA	NA	NA	NA
NX0722		1963	5738	NA	NA	NA	NA	NA	NA	
SX0722		1963	5738	NA	NA	NA	NA	NA	NA	
NX0519		1963	5852	NA	NA	NA	NA	NA	NA	
SX0519		1963	5852	NA	NA	NA	NA	NA	NA	
NX0822		1963	5738	NA	NA	NA	NA	NA	NA	
SX0822		1963	5738	NA	NA	NA	NA	NA	NA	
1-35-55-15-X0097WB		1957	10393	NA	NA	NA	NA	NA	NA	
X0097EB		1957	10393	NA	NA	NA	NA	NA	NA	
AVERAGES			16.0	7731	NA	NA	NA	NA	NA	NA
% OF BRIDGES AFFECTED					NA	NA	NA	NA	NA	NA

Figure 1

DIVISION FOUR

% HIGH HALF CELLS BY LOCATION

% LIGHT POPOUTS	% MOD. POPOUTS	% HEAVY POPOUTS	% SPALLS	% SEEPAGE	% HALF CELLS >300mV	% HIGH HALF CELLS BY LOCATION			% Cl ⁻ SAMPLES >2 lb./yd ³	COMMENTS
						AT JOINTS	IN GUTTERLINE	IN ROADWAY		
NA	NA	NA	NA	NA	14.21	1.58	10.35	2.28	NA	
NA	NA	NA	NA	NA	25.90	8.80	17.00	0.10	50.00	
NA	NA	NA	NA	25.27	45.54	9.36	17.88	18.30	87.50	
NA	NA	NA	NA	25.52	42.09	6.46	18.75	16.88	87.50	
NA	NA	NA	NA	8.58	31.76	11.73	17.51	2.52	NA	
NA	NA	NA	NA	23.87	28.91	9.00	17.40	2.51	NA	
NA	NA	NA	NA	11.74	29.70	8.03	17.12	4.55	NA	
NA	NA	NA	NA	7.26	49.10	6.88	12.86	0.45	NA	
NA	NA	NA	NA	18.61	39.38	4.38	20.00	15.00	NA	
NA	NA	NA	NA	14.62	49.53	3.96	24.53	21.04	NA	
NA	NA	NA	NA	43.49	72.68	11.62	39.38	21.68	68.18	
NA	NA	NA	NA	18.04	32.90	1.32	29.94	1.64	NA	
NA	NA	NA	NA	19.13	71.20	5.18	27.83	38.19	NA	
NA	NA	NA	NA	14.82	26.16	8.85	10.77	6.54	NA	
NA	NA	NA	NA	2.35	11.15	3.46	6.54	1.15	NA	
NA	NA	NA	NA	0	37.85	5.87	26.28	5.70	NA	
NA	NA	NA	NA	0.04	45.35	0.66	33.89	10.80	NA	
NA	NA	NA	NA	0	33.18	3.93	20.52	8.73	NA	
NA	NA	NA	NA	7.05	53.64	6.82	23.64	23.18	NA	
NA	NA	NA	NA	16.29	45.08	5.20	31.79	8.09	NA	
NA	NA	NA	NA	4.71	30.09	2.13	26.44	1.52	NA	
NA	NA	NA	NA	6.89	14.00	0.33	13.67	0	NA	
NA	NA	NA	NA	1.75	4.66	1.33	3.00	0.33	NA	
NA	NA	NA	NA	1.07	7.72	1.16	5.79	0.77	50.00	
NA	NA	NA	NA	1.71	24.33	2.32	16.22	5.79	70.00	
NA	NA	NA	NA	4.01	54.20	10.97	21.29	21.94	72.22	Spalls thru Deck
NA	NA	NA	NA	15.07	48.71	5.48	32.26	10.97	83.33	Honey comb on Bottom
NA	NA	NA	NA	3.16	36.45	7.10	20.00	9.35	NA	
NA	NA	NA	NA	2.11	14.55	2.90	4.88	6.77	NA	
NA	NA	NA	NA	13.07	78.39	2.58	24.52	14.84	63.16	
NA	NA	NA	NA	2.18	26.48	7.42	9.03	9.03	72.22	Spalls thru Deck
NA	NA	NA	NA	2.82	49.19	13.18	12.54	23.47	NA	Spalls thru Deck
NA	NA	NA	NA	0	7.43	1.94	4.52	0.97	NA	Honey Comb on Bottom
NA	NA	NA	NA	0	34.20	9.03	13.23	11.94	93.75	
NA	NA	NA	NA	0	23.86	0.63	10.00	13.23	75.00	Honey Comb on Bottom
NA	NA	NA	NA	8.63	29.66	4.34	10.31	15.01	53.33	
NA	NA	NA	NA	5.51	22.02	2.71	8.66	10.65	52.94	
NA	NA	NA	NA	9.41	34.87	6.34	17.85	9.67	69.94	
NA	NA	NA	NA	85.71	100	100	100	97.4	100.0	

Figure 19

DIVISION

COUNTY	BRIDGE NO.	DATE BUILT	DECK AREA	TOTAL % SCALE	% LIGHT SCALE	% MOD. SCALE	% HEAVY SCALE	TOTAL % POPOUTS	% LIGHT POPOUTS
TEXAS	95-70-30-X054	1950	23837.24	2.68	2.30	0.38	0	0	0
	64-70-10-X201	1946	29120	1.15	1.00	0.15	0	0	0
	3-70-24-X122	1947	14060.76	13.18	13.03	0.02	0.13	0	0
	95-70-32-X071	1955	8456	0.14	0.14	0	0	0	0
	95-70-32-X159	1953	5233.8	22.03	15.11	1.11	5.81	0	0
WOODS	50-76-26-X000	1921	27111	NA	NA	NA	NA	NA	NA
	64-76-26-X081	1929	3426.6	NA	NA	NA	NA	NA	NA
	11-02-20-X083	1961	5637.24	NA	NA	NA	NA	NA	NA
ALFALFA	64-02-04-X149	1928	2106.6	NA	NA	NA	NA	NA	NA
	11-02-20-X090	1935	4351.92	18.47	18.38	0.09	0	0	0
	64-02-06-X016	1928	2066.6	NA	NA	NA	NA	NA	NA
	83-04-08-X191	1935	38191.92	1.33	1.33	0	0	0	0
BEAVER	46-23-26-X030	1960	9548	3.15	3.15	0	0	0	0
	283-23-12-X111	1940	31711.2	NA	NA	NA	NA	NA	NA
HARPER	283-30-20-X104	1954	37839.2	11.04	0.56	0.56	0.02	0	0
	AVERAGES	33.2	16180	7.31	6.11	0.26	0.66	0.00	0.00
% OF BRIDGES AFFECTED				100.0	100.0	66.7	33.3	0.00	0.00

Figure 2

DIVISION SIX

% LIGHT POPOUTS	% MOD. POPOUTS	% HEAVY POPOUTS	% SPALLS	% SEEPAGE	% HALF CELLS >300mV	% HIGH HALF CELLS BY LOCATION			% CI ⁻ SAMPLES >2 lb./yd ³	COMMENTS
						AT JOINTS	IN GUTTERLINE	IN ROADWAY		
0	0	0	0.10	0	18.15	6.70	9.45	2.00	NA	
0	0	0	0.02	0	12.10	5.95	5.95	0.20	NA	
0	0	0	0.36	0	28.65	1.56	23.07	2.01	NA	
0	0	0	0	0	0	0	0	0	NA	
0	0	0	0.04	0	36.85	12.20	22.22	2.43	NA	
NA	NA	NA	NA	0	5.25	0	5.21	0.04	NA	Overlaid
NA	NA	NA	NA	0	10.72	0	10.72	0	NA	Overlaid
NA	NA	NA	NA	0	0	0	0	0	NA	Overlaid
NA	NA	NA	NA	0	0	0	0	0	NA	
0	0	0	0.46	0	5.26	4.09	0	1.17	NA	
NA	NA	NA	NA	0	15.22	6.52	8.70	0	NA	Only gutterline tested
0	0	0	0.64	0	37.41	1.87	33.10	2.44	NA	
0	0	0	0	0	40.52	5.39	28.10	7.03	NA	
NA	NA	NA	NA	0	0	0	0	0	NA	Overlaid
0	0	0	3.60	0	37.16	5.27	22.30	9.59	NA	
0.00	0.00	0.00	0.58	0.00	22.09	3.30	11.25	1.79	NA	
0.00	0.00	0.00	77.8	0.00	73.3	60.0	66.7	60.0	NA	

Figure 20

DIVISIO

COUNTY	BRIDGE NO.	DATE BUILT	DECK AREA	TOTAL % SCALE	% LIGHT SCALE	% MOD. SCALE	% HEAVY SCALE	TOTAL % POPOUTS	% LIGHT POPOUTS	% M POPC	
CARTER	177-10-12-X035	1951	179152	15.88	6.24	4.57	5.07	0	0		
	77-10-08-WX050	1960	10710	4.89	2.30	1.19	1.40	0	0		
	77-10-08- EX050	1957	9719.6	0	0	0	0	0	0		
COTTON	277-17-06-WX000	1963	117000	3.00	1.11	1.81	0.08	0.14	0.14		
LOVE	35-43-17-WX000	1965	41910	0.88	0.88	0	0	0	0		
	35-43-17- EX081	1962	14952	1.28	0.08	1.20	0	100.00	50.00		
	35-43-17-WX081	1962	14952	0	0	0	0	11.24	0		
JEFFERSON	32-34-14-X081	1950	3197	13.98	2.50	10.26	1.22	0	0		
COMANCHE	81-34-14-X087	1936	28680	1.37	0.79	0.44	0.14	0	0		
	62-16-04-EX055	1959	5076.4	2.43	1.64	0.79	0	0	0		
	62-16-04-X048-NB	1959	6615	0	0	0	0	0	0		
	62-16-04-X048-SB	1959	8820	3.41	3.41	0	0	0	0		
	62-16-04-X046-NB	1959	5616	0	0	0	0	0	0		
	62-16-04-X046-SB	1959	5616	3.74	3.74	0	0	0	0		
	62-16-04-X034-EB	1959	3409.2	0	0	0	0	0	0		
	62-16-04-X034-WB	1959	3409.2	0	0	0	0	0	0		
	CADDO	62-08-04-X123	1938	7377.6	5.77	3.90	0.95	0.92	0	0	
		281-08-12-X198	1960	7896	15.12	15.12	0	0	0	0	
GRADY	19-26-18-X183	1953	14554.4	8.99	6.96	1.74	0.29	0	0		
	81-26-12-X215	1955	47818.4	0.32	0.32	0	0	0	0		
	81-26-06-EX187	1957	11533.2	23.50	19.41	3.36	0.73	0	0		
AVERAGES		19.7	26096	4.98	3.26	1.25	0.47	5.30	2.39		
% OF BRIDGES AFFECTED				71.4	71.4	47.6	38.1	14.3	9.5		

Figure 21

DIVISION SEVEN

% HIGHHALF CELLS BY LOCATION

% LIGHT POPOUTS	% MOD. POPOUTS	% HEAVY POPOUTS	% SPALLS	% SEEPAGE	% HALF CELLS >300mV	% HIGHHALF CELLS BY LOCATION			% CI ⁻ SAMPLES >2 lb./yd ³	COMMENTS
						AT JOINTS	IN GUTTERLINE	IN ROADWAY		
0	0	0	0.11	0	53.81	4.51	32.29	17.01	NA	
0	0	0	0	0	67.01	0.73	34.59	31.69	NA	
0	0	0	0.99	0	46.09	1.52	33.59	10.98	NA	
0.14	0	0	0.08	0	3.12	1.82	0.65	0.65	NA	
0	0	0	0	0	5.72	2.52	2.73	0.47	NA	
50.00	50.00	0	0.05	0	5.78	2.38	2.38	1.02	NA	
0	11.24	0	0.40	0	4.42	2.61	0.34	1.47	NA	Popouts all thru Bridge
0	0	0	0.12	0	2.70	0.45	0	2.25	NA	
0	0	0	6.27	0	33.04	4.17	28.28	0.59	NA	
0	0	0	0.68	0	1.01	0	1.01	0	NA	
0	0	0	0	0	6.46	2.46	0.92	3.08	NA	
0	0	0	0	0	1.41	1.41	0	0	NA	
0	0	0	0	0	0.62	0.62	0	0	NA	
0	0	0	0	0	0.93	0.31	0.31	0.31	NA	
0	0	0	0	0.06	0	0	0	0	NA	
0	0	0	0	0	0	0	0	0	NA	
0	0	0	0	0	19.54	1.97	14.16	3.41	NA	
0	0	0	0	0	3.83	0	0	3.83	NA	
0	0	0	0	0.04	19.66	2.03	14.42	3.21	NA	
0	0	0	0	0	31.15	3.29	27.14	0.72	NA	
0	0	0	0	0	7.86	0.81	4.34	2.71	NA	
2.39	2.92	0.00	0.47	0.00	14.96	1.60	9.39	3.98		
9.5	9.5	0.00	38.1	9.5	90.5	85.7	71.4	76.2		

Figure 21

DIVISION

COUNTY	BRIDGE NO.	DATE BUILT	DECK AREA	TOTAL % SCALE	% LIGHT SCALE	% MOD. SCALE	% HEAVY SCALE	TOTAL % POPOUTS	% LIGHT POPOUTS
NOWATA	169-53-08-X1722	1962	3668	39.53	13.74	7.31	18.48	0	0
OSAGE	64-53-42-X000WB	1963	27499.5	0.03	0.03	0	0	0	0
	64-57-42-X000EB	1963	27499.5	4.11	2.00	1.56	0.55		
DELAWARE	20-21-22-X0137	1943	5888.5	0.25	0.25	0	0	0	0
TULSA	169-72-81-WX0695	1960	10780	2.08	1.62	0.46	0	0	0
	169-72-81-EX0695	1960	10780	17.75	7.8	0.97	8.91	0	0
	169-72-81-WX0716	1960	7840	NA	NA	NA	NA	NA	NA
	169-72-81-EX0716	1960	7840	NA	NA	NA	NA	NA	NA
	169-72-81-WX0755	1960	9100	NA	NA	NA	NA	NA	NA
	169-72-81-EX0755	1960	9100	NA	NA	NA	NA	NA	NA
	169-72-81-WX070	1958	10878	20.01	11.74	7.17	1.10	0	0
	169-72-81-EX070	1958	10878	11.02	6.25	2.88	1.89	0	0
	169-72-81-WX067	1961	4576	0	0	0	0	0	0
	169-72-81-EX067	1961	4576	2.14	2.01	0.13	0	0	0
	64-72-86-SX003	1963	3962	5.75	5.75	0	0	0	0
	64-72-06-X105	1938	45600	0	0	0	0	0	0
	266-72-33-X0595	1971	11546	0	0	0	0	0	0
	266-72-33-X0011	1971	27646	0	0	0	0	0	0
	266-72-33-X0560	1971	23920	11.54	11.54	0	0	0	0
	44-72-78-NX0938	1957	5130	NA	NA	NA	NA	NA	NA
	44-72-78-SX0938	1957	5130	NA	NA	NA	NA	NA	NA
	44-72-78-SX0823	1957	5562	NA	NA	NA	NA	NA	NA
	44-72-78-NY0805	1957	5130	NA	NA	NA	NA	NA	NA
	44-72-78-SX0805	1957	5130	NA	NA	NA	NA	NA	NA
	44-72-78-NX0866	1963	11000	0	0	0	0	0	0
	44-72-78-SX0866	1963	11000	0	0	0	0	0	0
	266-72-31-X0195	1971	20700	0	0	0	0	0	0
ROGERS	266-66-02-X0733	1971	19251	0	0	0	0	0	0
AVERAGES		15.4	12557	5.73	3.30	1.08	1.72	0.00	0.00
% OF BRIDGES AFFECTED				57.9	57.9	36.8	26.3	0.00	0.00

Figure 22

DIVISION EIGHT

% LIGHT POPOUTS	% MOD. POPOUTS	% HEAVY POPOUTS	% SPALLS	% SEEPAGE	% HALF CELLS >300mV	% HIGH HALF CELLS BY LOCATION			% Cl ⁻ SAMPLES >2 lb./yd ³	COMMENTS
						AT JOINTS	IN GUTTERLINE	IN ROADWAY		
0	0	0	4.09	0	5.32	0.88	4.44	0	69.23	
0	0	0	0.82	0	12.21	7.02	3.42	1.77	NA	Epoxy injected
0	0	0	3.63	0	21.70	7.13	9.17	5.40	NA	Epoxy injected
0	0	0	12.31	0	24.65	12.09	7.44	5.12	50.00	
0	0	0	0	1.31	51.43	12.11	22.08	17.24	41.18	
0	0	0	3.09	6.59	59.55	19.09	19.09	21.37	45.00	
NA	NA	NA	NA	9.34	24.32	5.64	12.45	6.23	43.75	
NA	NA	NA	NA	10.51	25.10	3.31	15.76	6.03	17.65	Overlaid
NA	NA	NA	NA	10.03	23.21	1.13	15.32	6.76	28.57	Overlaid
NA	NA	NA	NA	7.29	34.02	1.58	21.40	11.04	22.22	Overlaid
0	0	0	5.68	0	39.38	11.96	19.57	7.85	NA	Overlaid
0	0	0	6.09	0	61.23	18.95	22.20	20.08	NA	
0	0	0	0	0	0.32	0	0	0.32	NA	
0	0	0	1.68	0	2.86	0	2.86	0	NA	
0	0	0	0.13	0.25	1.60	1.20	0	0.40	NA	
0	0	0	7.74	0	3.05	2.91	0	0.14	NA	
0	0	0	0	0	2.54	2.18	0	0.36	25.00	
0	0	0	1.18	0	10.38	1.44	2.65	6.29	60.00	
0	0	0	0	0	10.13	4.22	3.00	2.91	9.38	
NA	NA	NA	NA	4.00	17.55	4.30	11.59	1.66	20.00	Overlaid
NA	NA	NA	NA	10.33	15.62	8.97	4.32	2.33	0	Overlaid
NA	NA	NA	NA	4.91	10.30	2.17	7.32	0.81	13.33	Gutter Epoxy patched
NA	NA	NA	NA	3.06	2.35	1.47	0.29	0.59	16.67	Overlaid
NA	NA	NA	NA	4.68	9.31	0.60	8.11	0.60	9.09	Overlaid
0	0	0	8.18	8.50	4.35	0	1.34	3.01	7.69	Spalls all in 1 Panel
0	0	0	0	0.90	0	0	0	0	0	Overlaid
0	0	0	0	0	1.23	0.41	0.51	0.31	NA	
0	0	0	0	0	24.19	6.48	3.59	14.12	72.41	
0.00	0.00	0.00	3.03	2.92	17.78	4.9	7.78	5.09	29.00	
0.00	0.00	0.00	66.7	50.0	100.0	85.7	82.1	89.3	88.9	

Figure 22

is not reflected throughout the remainder of the division. The mild winters account for, in part, the low percentage of half-cell readings in excess of 300 mV as well as the fact that these bridges were quite old when they were first salted.

Division III - The structures in Division III that were surveyed exhibited a greater degree of deterioration in two areas than would be predicted from the division's geographical position in south-central Oklahoma. These areas are half-cells in excess of 300 mV, and the incidence of popouts. The first area has no ready explanation; however, the second reflects the use of aggregate from the Bromide quarry, a traditionally poor performer.

Division IV - These structures as a group were in the poorest condition of any other: although the data supports this in only the most subtle of ways. The critical criteria is the amount of through deck seepage. Division IV structures had considerably more seepage than any other group and had the deck surfaces been visible they very likely would have shown high incidences of scaling and popouts also. Many of the structures in this group are on I-35 and were built in 1959 and 1960, just prior to Oklahoma's adoption of air-entrainment. They were constructed of a poor performing coarse aggregate containing shale and received applications of deicing salts relatively soon after construction. These factors contributed to freeze-thaw scale soon after construction. In an effort to reduce the severity of scale the structures received an asphaltic concrete overlay. This overlay retarded the evaporation of surface moisture and may have contributed to the high incidence of seepage experienced by these structures.

Division V - All the structures chosen for study in Division V had asphaltic concrete overlays and were not surveyed.

Division VI - This division occupies the northwestern portion of the state and has the most severe winters. This fact is not reflected in the condition of the structures studied. With the possible exception of freeze-thaw scale these structures exhibited low degrees of visible deterioration and only a moderate degree

of half-cell readings greater than 300 mV. The only explanation for this low incidence of deterioration is the fact that these structures were quite old at the time of their first salting.

Division VII - Located in the south central Oklahoma, this division's relatively mild climate is mirrored by the relatively small amounts of deterioration found on their bridges.

Division VIII - The structures studied exhibited moderate amounts of scale, low incidence of popout and seepage and moderate amounts of high half-cell readings in keeping with their climatological position in the northeastern corner of the state.

Chloride Analysis

In an effort to reduce the number of chloride analyses that were to be performed as an integral part of Oklahoma's Bridge Testing Routine, a statistical analysis was done to determine if a correlation existed between copper/copper sulfate half-cell readings and chloride ion content at the level of the upper mat of reinforcing. Regression analyses were performed on 850 samples taken from 51 different structures ranging in age from 5 to 33 years old, with seven different superstructures. In the histograms of Figure 23 the range of half-cell values are broken into increments of 50 mV except for the first and last increment. In the upper histogram the mean chloride content is plotted for each increment. The lower histogram plots the percent of samples within each increment that contain in excess of one and one-half pounds of chloride per cubic yard. The results of this statistical analysis were disappointing as the data scatter was too great to predict chloride ion contents from half-cell data alone, although from Figure 23 it is evident that an overall proportionality does exist. The analyses were performed on the following groupings: (1) individual bridges, (2) all bridges, (3) structures of the same age, and (4) structures with the same superstructure design. These group results are summarized below.

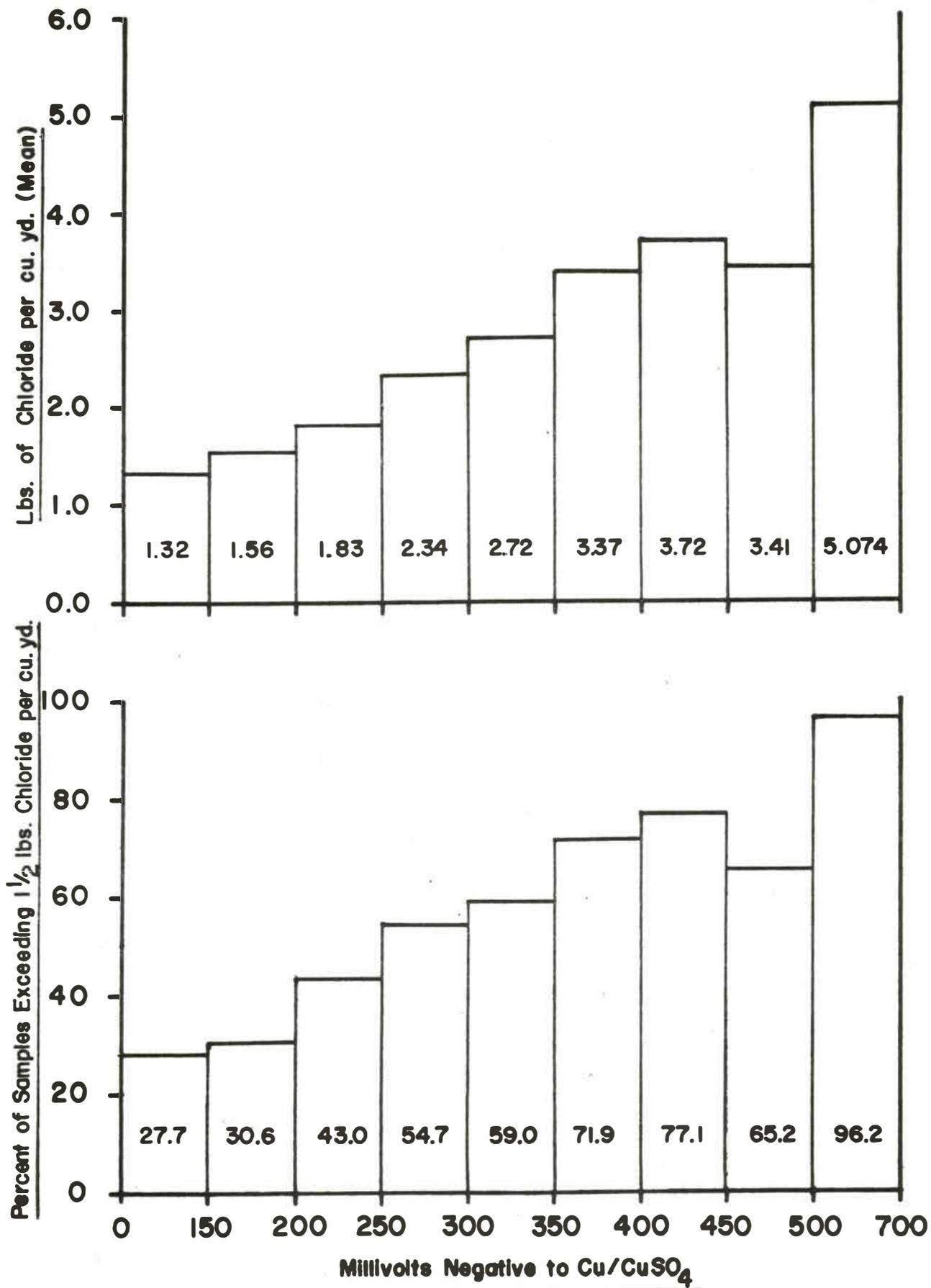


Figure 23
Millivolts Negative Cu/CuSO₄

1. Individual Bridges-The standard deviation for the raw chloride data was 1.908 lb. CL^-/yd^3 , the standard error as corrected by the regression lines relating half-cell data to chloride content at the level of the reinforcing for bridges taken as individuals was 1.432 lb. CL^-/yd^3 . This represents only a 25 percent improvement in estimation of chloride content by using this regression line vs. estimating chloride content by simply using the mean of the raw data. This standard error represents approximately 64 percent of the mean chloride content. When using these half-cell data, chloride contents at the rebar level could not be predicted with accuracy.
2. All Bridges - Regression analysis was performed on all the data taken as a population and resulted in a regression line with the following equation

$$CL = 7.603V + 0.2676$$

Where CL is the chloride ion content at the level of the reinforcing in pounds per cubic yard of concrete and V is the half-cell potential at the location of the chloride sampling. The correlation coefficient is 0.44 with a standard error of estimate of 1.715 lbs. CL^-/yd^3 . This standard error is 76 percent as large as the mean chloride content. This regression line provided the poorest estimate of chloride content of the four types of regression analysis.

3. Analysis by age of structure - Regression analysis on the data by structure age yielded a mean standard error of estimate of 1.597 lb. CL^-/yd^3 a quantity that is approximately 71 percent as large the mean chloride content.
4. Analysis by Superstructure - Regression analysis on the data separated by

superstructure design yielded a mean standard error of $1.688 \text{ lbs CL}^-/\text{yd}^3$ which is 75 percent as large as the mean chloride content.

It is apparent from the relatively large standard errors, that for the above groupings chloride ion content at the level of the upper reinforcing mat could not be predicted with a suitable degree of precision from half-cell data alone. Other factors not studied here, such as concrete pH, moisture content, and temperature may very well prove to be additional variables that must be considered when attempting to predict chloride contents.

CONCLUSIONS AND RECOMMENDATIONS

1. Presently Oklahoma has an adequate supply of quality aggregates to use in bridge deck concrete. The poor performing aggregates are for the most part known as a result of past performance. However, occasionally these poor performing aggregates are still used in bridge decks. Steps should be taken to prevent this from happening. Oklahoma should also investigate the use of the Power's one cycle test to categorize aggregates concerning their frost susceptibility in order to better utilize the finite quantities of suitable aggregates now available. By better utilization it is meant that there is no need to use an aggregate with exceedingly good freeze thaw resistance in concrete where this degree of resistance is not needed. These good performers should be conserved for use where harsh conditions require such good behavior.

2. Chloride content at the level of the rebar mat, or any other level for that matter, cannot be predicted from surface potential readings alone. Other variables known and perhaps unknown must contribute significantly to this relationship.

3. Salt induced deterioration of Oklahoma's bridges has reached sizable proportions. The mechanism for these problems is well understood; however, the prevention of these problems after the fact is difficult indeed. Few procedures are available to prevent or retard the growth of these deterioration mechanisms once they have begun. Future efforts in this area will undoubtedly result in additional preventative measures, but it is doubtful that these measures will be inexpensive. Thin bonded concrete overlays and cathodic protection are both expensive. It should seem prudent that future research work should have as its main thrust the investigation of the many ways deterioration can be avoided through design of new structures. In this area lies the greatest potential for savings. Areas that should be investigated are epoxy coated rebars, thin bonded overlays incorporating wax beads, the use of the new super water reducers in low water/cement ratio overlays, polymer impregnated concrete, and the use of sodium nitrate as a corrosion inhibitor in fresh concrete. Waterproof membranes could also be added to this list if design

engineers are willing to incorporate this membrane as a package. Presently design for membraned structures consists of little more than raising expansion dams, thus creating resevoirs to marinate asphalt in brine solutions. To make membranes effective over the long term better drainage facilities must be incorporated as part of the design package.

As a result of this project Oklahoma Department of Transportation has established a routine testing procedure for bridge decks that provides an efficient and effective means of determining the extent and degree of deterioration that a structure suffers. Personnel in five of Oklahoma's eight field divisions have been trained in these testing techniques and are routinely surveying structures in their divisions and reporting the findings to the Central Offices where decisions are made concerning the types of repair that are to be applied. In addition these same personnel routinely apply these testing techniques during construction to delineate areas and quantities of concrete to be removed during rehabilitation work.

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