DIVISION OF CONSTRUCTION AND RESEARCH
TRANSPORTATION LABORATORY
RESEARCH REPORT

CORROSION TESTING OF BRIDGE DECKS

75-10

CA-DOT-TL-5116-12-75-10
INTERIM REPORT
JANUARY, 1975

Prepared in Cooperation with the U.S. Department of Transportation,
Federal Highway Administration

Caltrans
CALIFORNIA DEPARTMENT OF TRANSPORTATION

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TECHNICAL REPORT STANDARD TITLE PAGE

1 REPORT NO	2 GOVERNMENT ACCESSION NO	3 RECIPIENT'S CATALOG NO
4 TITLE AND SUBTITLE Corrosion Testing of Bridge Decks		5 REPORT DATE January 1975
		6 PERFORMING ORGANIZATION CODE 19502-762503-635116
7 AUTHOR(S) R. F. Stratfull, W. J. Jurkovich, and D. L. Spellman		8 PERFORMING ORGANIZATION REPORT NO CA-DOT-TL-5116-12-75-10
9 PERFORMING ORGANIZATION NAME AND ADDRESS Transportation Laboratory 5900 Folsom Boulevard Sacramento, California 95819		10 WORK UNIT NO
		11 CONTRACT OR GRANT NO D-3-11
12 SPONSORING AGENCY NAME AND ADDRESS California Department of Transportation Division of Construction and Research Sacramento, California 95807		13 TYPE OF REPORT & PERIOD COVERED Interim
		14 SPONSORING AGENCY CODE

15 SUPPLEMENTARY NOTES

Contracted in cooperation with the U. S. Department of Transportation, Federal Highway Administration, "Corrosion Control of Steel in Concrete.

16 ABSTRACT

When the percentage of corrosive halfcell potentials on a bridge deck proper exceeded about 10% or concrete corrosion caused delamination exceeded about 1% of the deck area, a chloride analysis generally would not be required as it was already too great.

For the average depth of reinforcing steel, the quantity of chloride apparently needed to cause corrosion was statistically related to a maximum amount at the 95% confidence limits of 1.0 lb./cu.yd., or 0.59 Kg/m^3.

The accuracy of chloride determinations was about equal when the concrete was either drilled or cored.

Although the halfcell potential of -0.35 volts CSE is indicative of active corrosion, an equipotential contour map is the most reliable means for evaluating the corrosion activity of steel in concrete.

After repair of concrete delaminations, the percentage of corrosive potentials reduced by about 50%. Repairing concrete delaminations does not prevent or necessarily control corrosion at other other locations.

17 KEY WORDS Bridge deck, corrosion, concrete delamination, halfcell potential, deicing salt		18 DISTRIBUTION STATEMENT Unlimited	
19 SECURITY CLASSIF (OF THIS REPORT) Unclassified	20 SECURITY CLASSIF (OF THIS PAGE) Unclassified	21 NO OF PAGES 23	22 PRICE

DMRT 1, 42 ORIG 9 72

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STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF CONSTRUCTION AND RESEARCH
TRANSPORTATION LABORATORY

January 1975

D-3-11
TL No. 635116

Mr. R. J. Datel
Chief Engineer

Dear Sir:

I have approved and now submit for your information this interim research project report titled:

CORROSION TESTING OF BRIDGE DECKS

Study made by . . . . . Concrete Branch

Under the Supervision of . . . . . D. L. Spellman

Principal Investigator . . . . . R. F. Stratfull

Co-Investigator . . . . . W. J. Jurkovich

Report Prepared by . . . . . R. F. Stratfull

Very truly yours,

JOHN L. BEATON
Chief Engineer
Office of Transportation Laboratory

Attachment

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ACKNOWLEDGEMENT

This project was performed in cooperation with the U. S. Department of Transportation, Federal Highway Administration Agreement No. D-3-11.

The contents of this report reflect the views of the Transportation Laboratory which is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

The author wishes to acknowledge the work and other contributions of Mr. D. R. Higgins and G. Hood, former Bridge and Assistant Bridge Maintenance Engineers respectively, who cooperatively initiated the investigation. Also, the contributions of Paul Jurach, G. H. C. Chang, George Heller, V. Van Matre, Galen Yeaw, and D. Krizan are acknowledged.

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INTRODUCTION

There are several reports of investigations that relate the cause or factors that influence the corrosion of steel in concrete bridges[1-12]. However, some also contain laboratory[1,3,10,13-15] types of investigations of field structures[2-12]. For the most part, the published reports are not oriented to screening large numbers of structures for an operational type of evaluation of the corrosion condition.

As reported by Kliethermes[4], Hall[5], and others[6,7], the corrosion of steel in concrete bridge decks is related to the use of deicing salts and the problem was found in 46 states. Also, in NCHRP Synthesis No. 4[6], concern was expressed for 200,000 bridge decks in the United States.

Because of the large number of structures that have been exposed to deicing salts, it is obvious that research techniques must be streamlined to operationally determine the methods of repair and/or preservation of the decks[2-6,8,10,15]. Except for special cases, there are simply too many bridges and too few people and dollars to spend a long period of time to thoroughly evaluate each structure. Therefore, the intent of this investigation was to find a reasonable means to obtain the necessary technical information with a minimum of effort and resources.

In general, it has been demonstrated that the corrosion activity of steel in concrete can be nondestructively determined by half-cell potential measurements[7,10]; evaluation of physical concrete distress by means of visual observation, sounding[10], or chaining[11]; ascertaining the chloride content associated with the corrosion of steel in concrete[1,3,7-10,13]; and determining the concrete cover over the corroding steel[2,6-10,12,13,16].

From a corrosion standpoint, with a few exceptions, concrete quality in California is sufficiently good so that it is not of special significance. This is not to be inferred that concrete quality does not affect the time to corrosion of the steel. It is only pointed out that once the bridge is built, it cannot be changed, and concrete that is structurally and physically sound has no real function in correcting, controlling, or preventing corrosion of steel in existing bridge decks. Therefore, this corrosion investigation was specifically oriented toward evaluating corrosion investigation techniques and did not include a systematic evaluation of concrete strength, absorption, air entrainment, etc., although these factors do have a significant effect on performance if seriously deficient.

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SUMMARY AND CONCLUSIONS

Chloride, Potentials, and Delaminations

The quantity of chlorides in concrete associated with the incidence of active corrosion of the steel is about 1 lb./cu.yd. (0.59 Kg/m^3) of concrete.

It was observed that the maximum quantity of chloride at the 95% confidence limits found at the average level of the steel related to other work[10] as the best indication of salt content in a structure that is causing corrosion of the steel.

Except for isolated cases, the data indicate that the chloride content need not be determined if more than about 1% of the surface area of the bridge deck concrete is delaminated, or if more than about 10% of the total potential measurements are numerically greater than -0.35 volts to the saturated copper-copper sulfate halfcell (CSE).

The data also indicates that if the average chloride content at the level of the steel is greater than about 1.0 lb./cu.yd. (0.59 Kg/m^3), an analysis to determine the maximum statistical amount may have no practical significance as the chloride content is already too great.

Sampling for Chloride Determination

On six bridges it was observed that it did not seem to make any significant difference in the accuracy of the chloride determination when the concrete sample was obtained either by coring or drilling. This would indicate that a major variable in concrete sampling is controlled by the variation in the salt content, per se. In this respect, it may be that the variations in salt content are controlled by concrete properties, salting and snow removal practices, drainage, etc., but the size of the sampling apparatus as used in this investigation demonstrates that this latter variable, per se, is not significant. Therefore, it is obvious that concrete samples for chloride analysis could be obtained by recovering drilling dust or by cutting and pulverizing concrete cores.

With a coefficient of variation of the chloride contents found to be in the average range of 33% to 36%, no less than six samples for chloride analysis should be obtained to make a valid survey.

Potentials and Concrete Condition

The average potential of the steel in nondelaminated concrete was found to be -0.180 volts saturated copper-copper sulfate halfcell (CSE). For delaminated concrete, the average potential

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BRIDGES INSPECTED

Twenty-two bridges were inspected in 1972-73 to determine the operational feasibility of using various inspection techniques to evaluate the corrosion behavior of the steel in the decks. The overall results are shown in Tables 1 and 1a, Bridge Condition Variables.

As shown on Tables 1 and 1a, the data accumulation for various bridges consisted of: A chloride analysis[8,15], the measurement of halfcell potentials[7], measurements of concrete delaminations[11] and, where the concrete cover is shown to the closest 0.01 inch, it was measured by a Pachometer; or the indicated concrete cover was not measured but was that specified when the value is shown to be plus or minus.

As will be recognized, when dealing with a random investigation of individual field structures, the data obtained does not always result in information that is suitable for an overall analysis. For example, the investigation was directed at evaluating inspection techniques for structures that had been exposed to deicing salts; however, one older bridge was found to have been constructed with calcium chloride added to the concrete. In addition, another bridge deck, selected as a bridge in "good condition", was found to have an average of five pounds of chloride per cubic yard at the level of steel (2.97 Kg/m^3) and corrosion was neither active nor was there evidence of past distress. The reason for the passive condition of this latter bridge deck was not investigated although previous work[10] has shown that corrosion may be dormant for a period of time in salt contaminated concrete under conditions of low moisture content.

As shown in Tables 1 and 1a, the chloride content at the level of the steel is shown in terms of the average, the maximum content analyzed in any sample, and the maximum content that would be indicated by calculating a statistical distribution and deriving the maximum quantity at the 95% confidence limits of the data. The 95% confidence limits was used to define a repeatable maximum limit of chloride content rather than to depend upon a randomly obtained maximum value. In Tables 1 and 1a, there is fair agreement between the actual maximum chloride content found in a bridge deck as compared to the maximum calculated from the 95% confidence limit of the data.

To obtain the chloride content at the level of the steel, the original data was plotted on semilog paper. The chloride content was plotted on the log scale ordinate vs. its associated depth below the surface of the deck on the linear scale (abscissa).

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and delaminations. Since the structures are those that have received deicing salts over a period of time, increasing salt content only reflects upon a gain of salt in the concrete with time. The continuing corrosion of the steel in concrete is time dependent and not necessarily reflected to increasing salt content above some threshhold level. For example, it is not required that the bridge shown on Tables 1 and 1a have more than 3 lbs. chloride per cubic yard (1.8 Kg/m^3) in order to contain 26% corrosive potentials and 13% of its area delaminated. Once corrosion begins, it is time dependent in that it becomes more extensive as time increases. In concrete, the primary function of the chloride-ion is to destroy the passivity of the steel. Once this occurs, the actual corrosion rate of the steel is controlled by polarization and other effects as well as the continuance of the concrete to be an electrolyte. An increase in salt content is not necessary to keep corrosion active.

For example, on Tables 1 and 1a, one bridge is listed with eight years of service, contains a maximum of 9.9 lbs. Cl/cu.yd. (5.8 Kg/m^3), and the relative area of corrosion caused concrete delamination on the deck is 6.7%. Conversely, another bridge with six years of service has a maximum chloride content of 1.6 lbs. Cl/cu.yd. (0.94 Kg/m^3) and also has 6.7% of its deck delaminated by corroding steel. From these and other data shown, it is obvious that once the concrete becomes chloride contaminated, the corrosion caused distress is not controlled by chloride content beyond that needed to depassivate the steel, but is controlled by other variables that specifically relate to the corrosion process and its effect.

Chloride Sampling

On six bridges, the chloride content of cores, which were sliced and then pulverized in the laboratory, were compared to that obtained by drilling the concrete and analyzing samples of concrete that are pulverized in-situ. Results are shown on Tables 2 and 2a.

At each bridge, 3-inch (7.6 cm) diameter cores were obtained and sliced into 1-inch thick discs that were then pulverized and analyzed. At each core location and approximately 1-inch (2.5 cm) from the edge of the core hole, four drill samples were made at 90° intervals about the circumference of the core hole. The drillings of the two different diameter drills were first made to a 1-inch depth (2.5 cm) below the surface, then the pulverized concrete was removed with a small spoon. The hole was then air blown to clean out the residue, and the process was repeated for the depth of between 1-inch (2.5 cm) and 2 inches (5.1 cm) below the deck surface.

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POTENTIALS AND CONCRETE CONDITION

Halfcell potentials were measured and tabulated (Table 3) for eight bridges having concrete delamination. In large delaminated areas where two or more measurements were made, the potentials were tabulated for a maximum and minimum value. In small areas of delaminations where only one potential measurement was made, it was listed as an "isolated" potential.

In general, where the length in area of delamination was about 3 feet (1.0 m) or more, at least two potential measurements were made. The average of the maximum potentials was -0.453 volts CSE, and the average of the minimum potential within the same delaminations was -0.334 volts CSE. At concrete delaminations that had a diameter (roughly) of about one foot (0.3 m) or less, the average of the single potential readings made in these locations was -0.385 volts CSE.

As shown on Table 3, in locations where the concrete was not delaminated, the average of all potential measurements was -0.180 volts CSE.

On a field structure, as compared to laboratory specimens, there is a greater possibility of error in defining corrosive and noncorrosive potentials. This is because of the polarizing effects of between the anodes and cathodes on the large masses of steel in a field structure. This effect will reduce the actual open circuit potentials of the anodes and simultaneously increase the potentials of the cathodes.

The maximum measured potential in any corrosion cell should be used as a criterion for analysis. In this regard, since concrete delamination on the eight bridges surveyed was observed, when the average of the maximum values was -0.453 and the average of the isolated values was -0.385 volts CSE, it is obvious that these values signify active corrosion of the steel. Even though these values indicate the potential of corroding steel, it must be recognized that they represent an advanced stage of corrosion and are of a numerically greater potential than when corrosion was initiated.

The determination of noncorroding potentials of steel is not defined by the criterion that the concrete is not delaminated. This is because the corrosion of the steel may not as yet have progressed to the stage where it causes the concrete to fracture. This fact is illustrated on Table 3 where the average potential of the steel in nondelaminated concrete for one bridge (S. Mt. Shasta) was -0.382 volts CSE while the average for all structures was -0.180. Although nondelaminated concrete cannot be used to clearly establish either an active or passive potential of the steel, it is obvious that the bridge deck concrete was observed to be sound when the average potential was -0.180 volts CSE.

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POTENTIAL SURVEY METHODS AND DECK REPAIR

Halfcell potentials can serve two or more purposes which can encompass (1) determination of the locations where steel is corroding, and (2) classification of corrosion activity of steel according to the percent of corrosive potentials; (3) determining the effectiveness of a repair method.

In order to develop an economical method to classify the condition of bridges, potentials were made on a grid pattern of either 4 feet (1.2 m), 2 feet (0.6 m), and on a random basis. The random selection method of obtaining potentials was performed on the basis of obtaining the measurements in the curb area of lowest elevation at spacings of approximately 4 feet (1.2 m) longitudinally, and a minimum of 30 values was required.

A complete potential survey was made of five bridges before and after concrete repairs (see Table 4). The repairs were only made at locations of delaminated concrete.

There appeared to be no significant difference in the percentage of corrosive potentials when the potential measurement spacing was either 4 feet (1.2 m) or 2 feet (0.6 m). However, when smaller intervals were used and results plotted on equipotential contour maps (not shown), there was better definition of corroding areas.

With regard to evaluating the corrosion activity of the decks, as determined by the percentage of corrosive potentials (percent of values numerically greater than -0.35 volts (CSE), the overall average of corrosive potentials computed either by the complete or random survey for all structures was 19%. However, as will be noted in Table 4, the random survey did not detect any corrosive potentials on two structures (Canyon Creek and Sawmill), while the complete survey did detect corrosion activity. In one case (Sawmill), the amount of concrete delamination of the deck was 0.4%; in the other case, the bridge had 6.7% delaminated area. It is obvious that the random survey, though a rapid system for evaluating corrosion activity as compared to the time involved in obtaining a complete potential record, will not be perfect. Discrepancies can be minimized in the random type survey by obtaining potential values in areas of delamination.

As previously mentioned, the percentage of corrosive potentials were determined for five bridge decks before and after repairs. From these data, it is shown that after repairs are made, there was a reduction in the percentage of corrosive potentials by an average of about 50%. Therefore, this type of repair is basically a mechanical repair which can initially reduce but not prevent or control additional corrosion of the steel.

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TABLE 1

Bridge Condition Variables *
(English Units)

Bridge	Average Cl	Maximum Sample Cl	Maximum Cl, 95% Limit	Corrosive Potentials Percent	Delamination, Percent	Years Service	Average Concrete Cover, Inches
Weimar, Rt.	0.2	0.4	0.6	3	0.1	14	1.5+
Weimar, Lt.	0.2	2.0	0.8	5	0	14	2.43
New England Mills,							
Left	0.2	1.6	1.6	7	0	14	2.50
Right	0.5	1.6	1.6	4	0	14	2.71
Grapevine, Left	0.9	1.4	1.1	-----	5	23	1.5+
" Right	0.9	1.2	1.2	-----	22	23	1.5+
Cressy	1.0	2.4	3.1	-----	18	22	1.5+
Lebec Road	1.3	3.4	2.9	-----	28	9	1.5+
Gray Creek	1.0	1.6	1.9	17	6.7	6	1.90
Canyon Creek	1.3	1.9	2.3	4	5.5	7	1.66
Ft. Tejon	2.0	3.6	3.2	-----	40	23	1.5+
Pony Bar	1.9	3.1	3.4	35	30	6	1.70
Milagra Drive	3.0	6.4	6.4	6	11	7	1.30
101/116 Sep.	2.7	4.9	5.6	2	7.6	17	1.43
Sawmill	2.2	3.5	3.7	10	0.4	8	1.78
Mt. Shasta, N.	2.6	4.1	4.2	21	1.3	8	1.58
Sly Park	3.5	6.0	6.0	37	12	8	1.68
Lake St.	4.0	5.8	5.7	47	10	8	1.56
Long Valley	5.0	8.0	8.6	0	0	14	1.5+
Lebec	5.0	8.8	10.0	-----	28	9	1.5+
Mt. Shasta, S.	6.1	9.9	8.2	73	6.7	8	1.58

* Chloride Contents are in Lbs./cu.yd. at the depth of steel

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TABLE 2

Concrete Sampling Variations
(English Units)

Bridge	Sample Type	0-1 Inch Depth n	0-1 Inch Depth $\bar{X}$	0-1 Inch Depth Std. Dev.	0-1 Inch Depth Coef. Var. %	1-2 Inch Depth n	1-2 Inch Depth $\bar{X}$	1-2 Inch Depth Std. Dev.	1-2 Inch Depth Coef. Var. %
Lebec	3" cores	6	9.74	3.11	31.9	6	4.54	2.50	55.1
	All drillings	24	8.00	1.98	24.8	24	5.42	1.74	32.1
	3/4" drillings	4	10.2	3.53	34.6	4	6.00	1.18	19.7
	1" drillings	20	7.56	1.24	16.4	20	5.30	1.83	34.5
Cressy	3" cores	3	1.60	0.80	50.0	3	0.93	0.61	65.6
	3/4" drillings	12	1.83	1.20	65.6	12	1.12	0.72	64.3
Lebec Road	3" cores	6	2.60	0.92	35.4	6	1.20	0.62	51.7
	3/4" drillings	24	2.73	0.87	31.9	24	1.40	0.80	57.1
Ft. Tejon	3" cores	6	3.93	0.99	25.2	6	1.67	0.39	23.4
	3/4" drillings	24	3.77	1.31	34.7	24	2.19	0.77	35.2
Grapevine, Right	3" cores	6	1.37	0.23	16.8	6	0.73	0.16	21.9
	3/4" drillings	24	1.44	0.21	14.6	24	1.04	0.08	7.7
" Left	3" cores	6	1.13	0.48	42.5	6	0.70	0.11	15.7
	All drillings	24	1.27	0.21	16.5	24	1.03	0.11	10.7
	3/4" drillings	16	1.20	0.21	17.5	16	1.01	0.09	8.9
	1" drillings	8	1.40	0.15	10.7	8	1.07	0.15	14.0

Note: Chlorides in pounds per cubic yard.

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TABLE 3

Potentials and Delaminations

Bridge	Delaminated Concrete: Maximum Potential - Volts n	Mean	Std. Dev.	Minimum Potential - Volts n	Mean	Std. Dev.	Isolated Potentials - Volts n	Mean	Std. Dev.	Non-delaminated Concrete: All Potentials, Volts n	Mean	Std. Dev.	Percent Delaminated Concrete
Canyon Cr.	6	.390	.049	6	.247	.106	4	.278	.064	1742	.096	.094	5.5
Gray Creek	26	.462	.059	26	.343	.068	80	.356	.100	376	.180	.010	6.7
S. Mt. Shasta	10	.523	.076	10	.429	.091	97	.466	.066	391	.382	.084	6.7
Lake Street	25	.496	.036	25	.383	.044	107	.388	.070	377	.227	.093	10.0
Pony Bar	22	.381	.074	22	.247	.090	86	.314	.074	198	.024	.104	30.0
N. Mt. Shasta	2	.50	.20	2	.41	.15	48	.420	.095	609	.263	.095	1.3
Sawmill	2	.43	.08	2	.36	.06	42	.383	.044	592	.231	.100	0.4
Weimar	1	.34	----	1	.23	----	10	.29	.062	392	.181	.071	0.1
Weighted Average	94	.453	.060	94	.334	.072	474	.385	.075	4677	.180	.086	

Note: Isolated potentials signify that only one measurement was made in localized area.

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FIGURE I

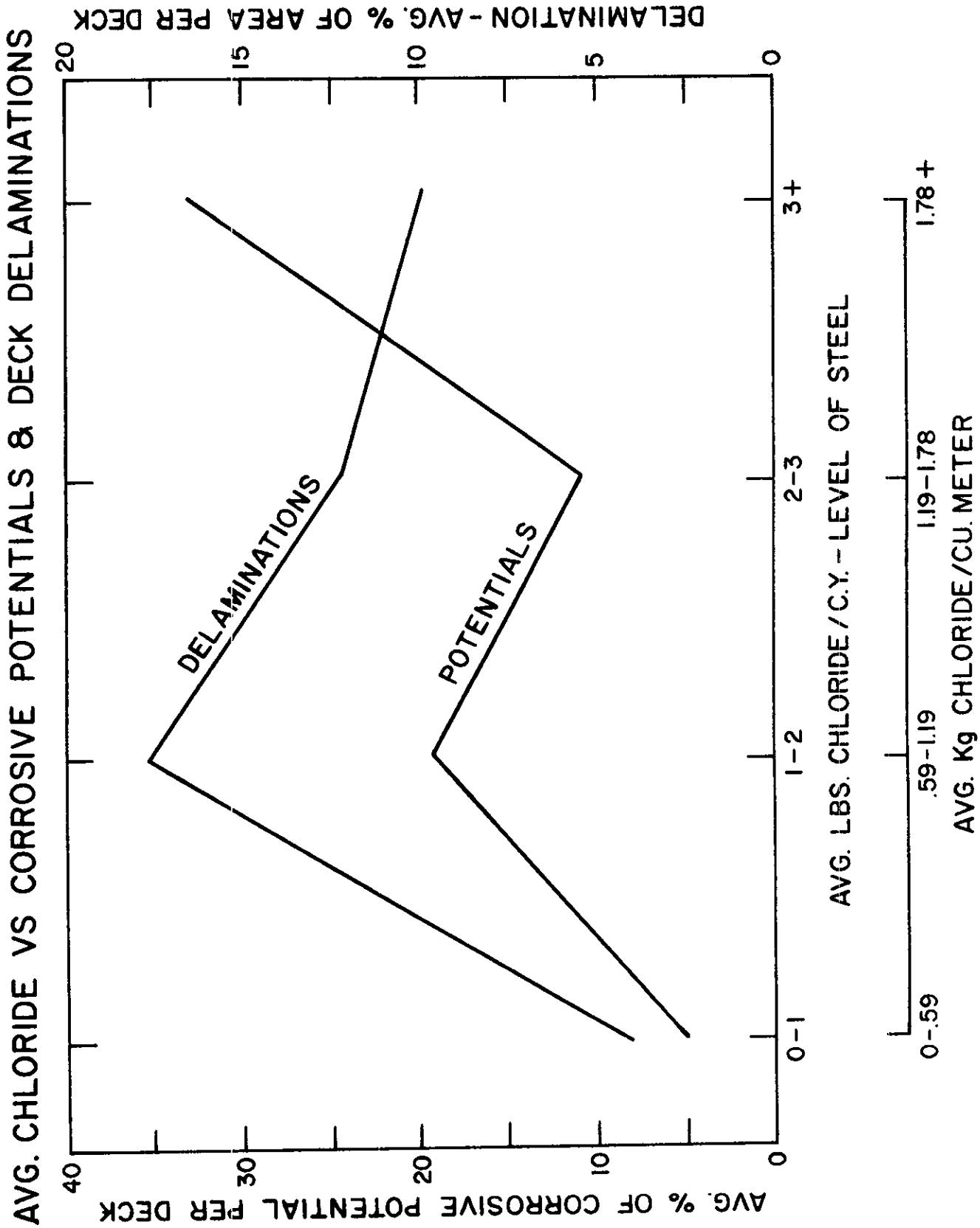

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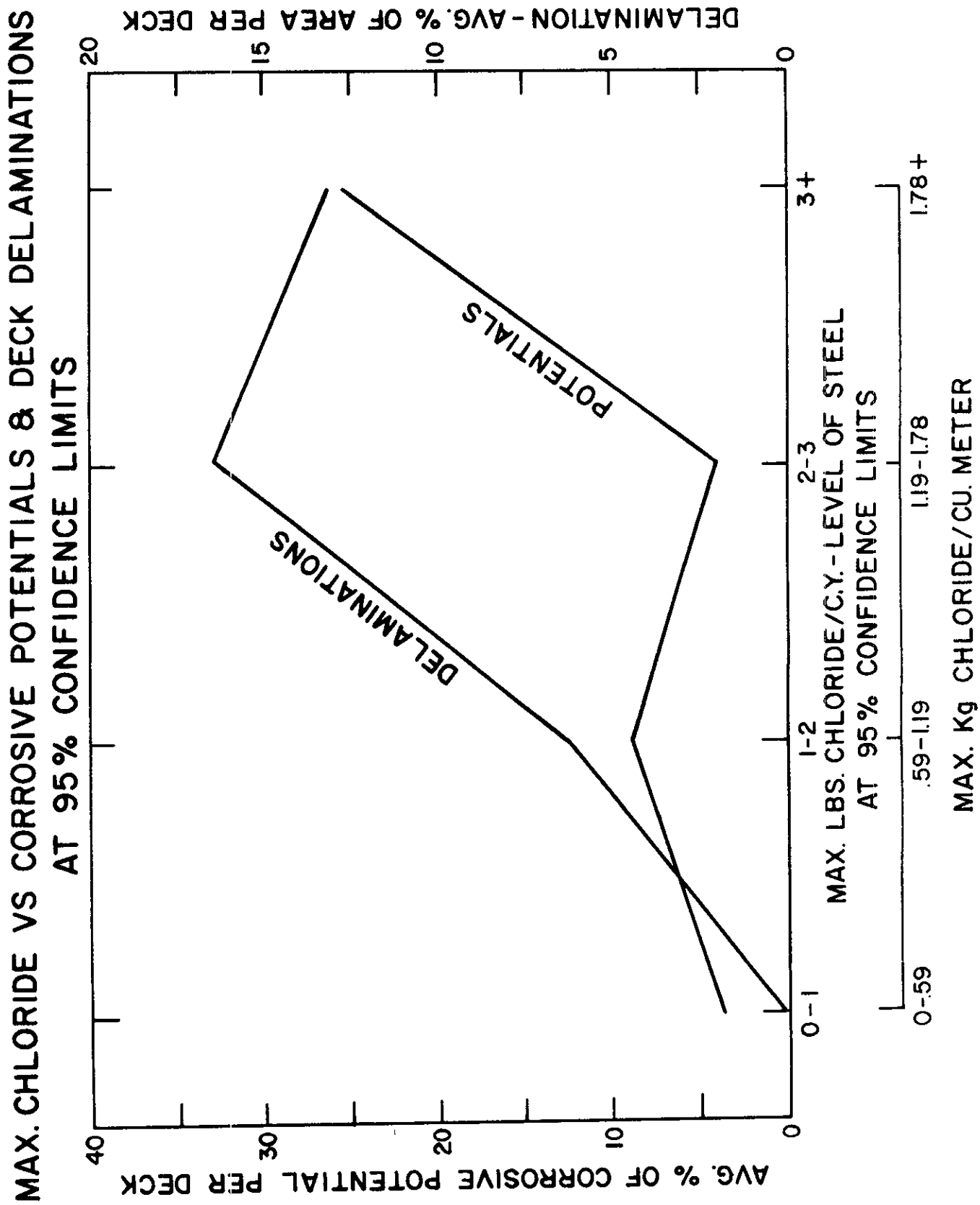
FIGURE 3



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As a result, chloride analyses were made of the bridge deck concrete to evaluate quality, effects of sampling method, and number of samples, relationship to concrete distress, and to the two techniques employed for obtaining halfcell potentials of the steel. In addition, an evaluation was made on five structures to determine the effect of repairs on the change in the halfcell potentials of the steel.

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in small isolated areas was -0.385 volts, while the average of the minimum and maximum values found in large corroding areas was -0.334 and -0.453 volts CSE respectively. These values confirm that there is great assurance of active corrosion when the potential of the steel is numerically greater than -0.35 volts CSE.

The determination of active corrosion at halfcell potential values numerically less than -0.35 volts CSE, will require interpretation and consideration of evidence, such as corrosion caused concrete delaminations being present, or plotted equipotential contours that indicate an anodic or corroding area at a lesser maximum potential.

Potentials - Random Survey

It was found that a random survey of the electrical halfcell potential could be a rapid and economical means for evaluating the corrosion activity of the steel in numerous bridges. This method has limitations as any other statistical sampling, but it appears to be economically worthwhile where a rapid evaluation of numerous structures is concerned.

Potentials - Before and After Repair

From potential measurements made on five bridge decks both before and after the repair of concrete delaminations, it was found that the percentage of corrosive potentials was reduced by about one-half. Therefore, repairs of delaminations in chloride contaminated concrete is a mechanical type of repair which can initially reduce some but not all locations of corrosion activity.

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For the approximate 4-inch (10.2 cm) depth of the cores, the best fit of the points was made by a straight line, then the chloride content at the level of the steel was obtained for the determined average depth of cover.

On Tables 1 and 1a, the column "Corrosive Potentials - Percent", represents the percentage of potential measurements of the steel in the bridge deck relative to the saturated copper-copper sulfate halfcell (CSE) that are more negative than -0.35 volts[13,14]. Also, on Tables 1 and 1a, the column "Delamination, Percent", represents the percentage of the bridge deck of which the concrete was delaminated as determined by the chain drag[11]. The "Delamination" designation also included any surface area of the deck from which concrete had already spalled as a result of corrosion.

Chlorides vs. Potentials and Deck Delaminations

Data were grouped according to ranges of chloride content and then the averages for potentials and delaminations for the bridge decks were plotted against the associated chloride content as shown on Figures 1, 2, and 3.

As shown by Figure 1, when the average chloride content was 0 to 1.0 lb./cu.yd. (0.59 Kg/m^3) or less, the average percentage of corrosive potentials was 4.8% of the total measurements, while the average percent of concrete delamination was 4.5% of the total deck area.

On Figure 2, where the maximum chloride content in any sample at the level of the steel was used as a criterion, it will be observed that for chloride contents of less than 1.0 lb./cu.yd. (0.59 Kg/m^3), the percentage of corrosive potentials was found to average 4%, while the average area of delamination was 0.05%.

In Figure 3, which uses the maximum chloride content at the average level of the steel calculated to the 95% confidence limit, the average percentage of corrosive potentials was 4%, and the average area of delamination was 0.05%.

From Figure 1 through 3, it is obvious that a maximum chloride content at the average level of the steel is related to the incidence of active corrosion potentials and the resulting concrete delaminations confirms the amount previously associated with corrosion of the reinforcing steel[10].

Figures 2 and 3 may give a misleading impression that increasing chloride contents result in an increase in corrosive potentials

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As shown on Tables 2 and 2a, the mean values for the chloride contents that were obtained either by drilling or coring do not appear to be consistently statistically different as evidenced by the standard deviation and coefficients of variation.

In some cases, as shown on Tables 2 and 2a, the coefficients of variation for the results of the drilled samples are both greater and less than that for core samples. For an equal number of observations, the greater the coefficient of variation, the less the accuracy of the mean. Therefore, it seems that the major variable is an inconsistent distribution of chlorides through the structure rather than the diameter of sampling apparatus, per se. This is further emphasized by the fact that the average of the coefficients of variation for the chloride analysis of the drill samples was about 33%, while the average of the coefficients of variation for the cores was 36%. In a previous study[7] the coefficient of variation for cores was about 30%. From this, it is obvious that a sufficient number of samples (say, at least six) should be obtained in order to evaluate the chloride content with a much higher level of confidence than can be obtained with a smaller number of samples.

Bridge Repairs

The five bridges listed on Table 4 were generally repaired in the following manner:

1. The delaminated concrete was outlined on the surface of the deck as indicated during sounding by means of chain being swept over its surface;

2. Then the concrete in the delaminated areas was removed by means of a jackhammer below the depth of the transverse reinforcing steel in the top mat;

3. Steel is then either sandblasted clean or replaced.

4. An epoxy adhesive is then used to coat all exposed concrete and steel surfaces,

5. A high strength concrete mortar is then used to fill the holes.

After the concrete repairs were made, a waterproof membrane, and a 2 or 3-inch (5.1 or 7.6 cm) thick asphalt concrete overlay was placed.

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For a high level of assurance that corrosion is active, potentials numerically greater than -0.350 volts CSE seems to be a reliable criterion. In certain cases, because of the probable effects of polarization, etc., there will be measurement of potentials of steel in the range of -0.25 to -0.35 volts CSE and concrete distress may be evident.

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DISCUSSION

The corrosion of steel in concrete is a dynamic process. There is a continual replenishment of oxygen; conversion of iron to its final form of rust; polarization effects; variations in the halfcell potential due to oxygen, chloride, and hydrogen-ion concentrations; and variations in the moisture content of the concrete which affects its resistivity and ability to act as an electrolyte.

If the preceding is ignored, then the interpretation of the influence of particular variables such as halfcell potential values and chloride content of the concrete, can lead to erroneous conclusions. For example, when the chloride-ion content of the concrete is greater than say 1.0 lb./cu.yd. (0.59 Kg/m^3), there is no reason to believe that there always is an automatic and irrevocable start of the corrosion process. A corrosion threshhold of amount of chloride is only a point in the concentration where corrosion can begin. This is emphasized by previous work[9,10] where it was demonstrated that corrosion activity was mil when the specific electrical resistance was greater than 60,000 ohm/cm in salt contaminated concrete[10]. However, it was also demonstrated that when nondistressed salt contaminated concrete was painted, there were electrical potential[18] and visual[10] indications that corrosion was accelerated. In addition, evaluations of electrical potential measurements have indicated that there is corrosion activity of steel in concrete when corroding bridge decks are overlaid with concrete or waterproof membrane[4,5].

When suitable data are available, the economics of bridge deck repair or corrosion prevention with procedures such as epoxy injection to bond the delamination[19,20], concrete removal and replacement[4,5,6,8,11], cathodic protection[19], or the use of waterproof membranes[4,5,6,10] can be determined.

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Corrosive Potentials Percent	Delamination, Percent	Years Service	Average Concrete Cover, cm
3	0.1	14	3.8±
5	0	14	6.17
7	0	14	6.35
4	0	14	6.88
-----	5	23	3.8±
-----	22	23	3.8±
-----	18	22	3.8±
-----	28	9	3.8±
17	6.7	6	4.83
4	5.5	7	4.22
-----	40	23	3.8±
35	30	6	4.32
6	11	7	3.30
2	7.6	17	3.63
10	0.4	8	4.52
21	1.3	8	4.01
37	12	8	4.27
47	10	8	3.96
0	0	14	3.8±
-----	28	9	3.8±
73	6.7	8	4.01

of steel.

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TABLE 1a

Bridge Condition Variables *
(Metric Units)

Bridge	Average Cl	Maximum Sample Cl	Maximum Cl, 95% Limit	Corrosive Potentials Percent	Delamination, Percent	Years Service	Average Concrete Cover, cm
Weimar, Rt.	0.14	0.24	0.26	3	0.1	14	3.8+
" , Lt.	0.14	0.25	0.26	5	0	14	6.17
New England Mills,							
Left	0.12	0.95	0.95	7	0	14	6.35
Right	0.31	0.95	0.95	4	0	14	6.88
Grapevine, Lt.	0.53	0.83	0.65	-----	5	23	3.8+
" Rt.	0.53	0.71	0.71	-----	22	23	3.8∓
Cressy	0.59	1.42	1.84	-----	18	22	3.8∓
Lebec Rd.	0.77	2.02	1.72	-----	28	9	3.8∓
Gray Creek	0.62	0.96	1.10	17	6.7	6	4.83
Canyon Creek	0.78	1.14	1.35	4	5.5	7	4.22
Ft. Tejon	1.18	2.14	1.90	-----	40	23	3.8+
Pony Bar	1.14	1.83	2.02	35	30	6	4.32
Milagra Dr.	1.78	3.80	3.80	6	11	7	3.30
101/116 Sep.	1.59	2.89	3.32	2	7.6	17	3.63
Sawmill	1.29	2.09	2.19	10	0.4	8	4.52
Mt. Shasta, N.	1.56	2.40	2.52	21	1.3	8	4.01
Sly Park	2.08	3.56	3.56	37	12	8	4.27
Lake St.	2.37	3.44	3.35	47	10	8	3.96
Long Valley	2.97	4.75	5.10	0	0	14	3.8+
Lebec	2.96	5.22	5.93	-----	28	9	3.8∓
Mt. Shasta, S.	3.64	5.87	4.86	73	6.7	8	4.01

* Chloride contents are in Kg/m^3 at the depth of steel.

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TABLE 2a

Concrete Sampling Variations
(Metric Units)

Bridge	Sample Type	0-2.54 cm Depth n	0-2.54 cm Depth $\overline{X}$	0-2.54 cm Depth Std. Dev.	0-2.54 cm Depth Coef. Var. %	2.54-5.08 cm Depth n	2.54-5.08 cm Depth $\overline{X}$	2.54-5.08 cm Depth Std. Dev.	2.54-5.08 cm Depth Coef. Var. %
Lebec	7.62 cm cores	6	5.78	1.84	31.9	6	2.69	1.48	55.1
	All drillings	24	4.75	1.17	24.8	24	3.21	1.03	32.1
	1.91 cm drillings	4	6.05	2.09	34.6	4	3.56	0.70	19.7
	2.54 cm drillings	20	4.48	0.735	16.4	20	3.14	1.09	34.5
Cressy	7.62 cm cores	3	0.949	0.47	50.0	3	0.55	0.36	65.6
	1.91 cm drillings	12	1.09	0.712	65.6	12	0.65	0.43	64.3
Lebec Rd.	7.62 cm cores	6	1.54	0.55	35.4	6	0.712	0.37	51.7
	1.91 cm drillings	24	1.62	0.52	31.9	24	0.830	0.47	57.1
Ft. Tejon	7.62 cm cores	6	2.33	0.59	25.2	6	0.991	0.23	23.4
	1.91 cm drillings	24	2.24	0.777	34.7	24	1.30	0.46	35.2
Grapevine, Right	7.62 cm cores	6	0.813	0.14	16.8	6	0.43	0.095	21.9
	1.91 cm drillings	24	0.854	0.12	14.6	24	0.617	0.05	7.7
Left	7.62 cm cores	6	0.670	0.28	42.5	6	0.42	0.07	15.7
	All drillings	24	0.753	0.12	16.5	24	0.61	0.07	10.7
	1.91 cm drillings	16	0.712	0.12	17.5	16	0.60	0.05	8.9
	2.54 cm drillings	8	0.830	0.089	10.7	8	0.63	0.09	14.0

Note: Chlorides in Kg/m^3.

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TABLE 4

Potential Measurements

Bridge	Before Repair: Full Survey C/C	Before Repair: Full Survey n	Before Repair: Full Survey Percent Corrosive	Before Repair: Random Survey n	Before Repair: Random Survey Percent Corrosive	After Repair: Full Survey C/C	After Repair: Full Survey n	After Repair: Full Survey Percent Corrosive
Gray Creek	4'	454	26	56	14	4'	453	16
S. Mt. Shasta	4'	459	73	99	82	4'	460	37
N. Mt. Shasta	4'	630	21	148	32	4'	630	6
Lake Street	4'	477	47	49	45	4'	477	22
Canyon Creek	4'	478	4	--	--	4'	482	2
	2'	1802	3	99	0	--	---	--
Sawmill	4'	607	10	30	0	--	---	--
Milagra	2'	1041	7	180	5	--	---	--
Pony Bar	4'	342	35	49	41	--	---	--
Long Valley	4'	142	0	30	0	--	---	--
Weimar, Rt.	4'	380	3	90	9	--	---	--
	2'	1424	4	--	--	--	---	--
Weimar, Lt.	4'	389	5	90	9	--	---	--
New England Mills, Right	4'	466	4	90	3	--	---	--
Left	4'	465	7	90	12	--	---	--

Note: C/C is minimum center to center spacing; 4' = 1.2 m, 2' = 0.6 m.
n is number of observations.
% Corrosive is percent of potentials numerically greater than -0.35 volts.

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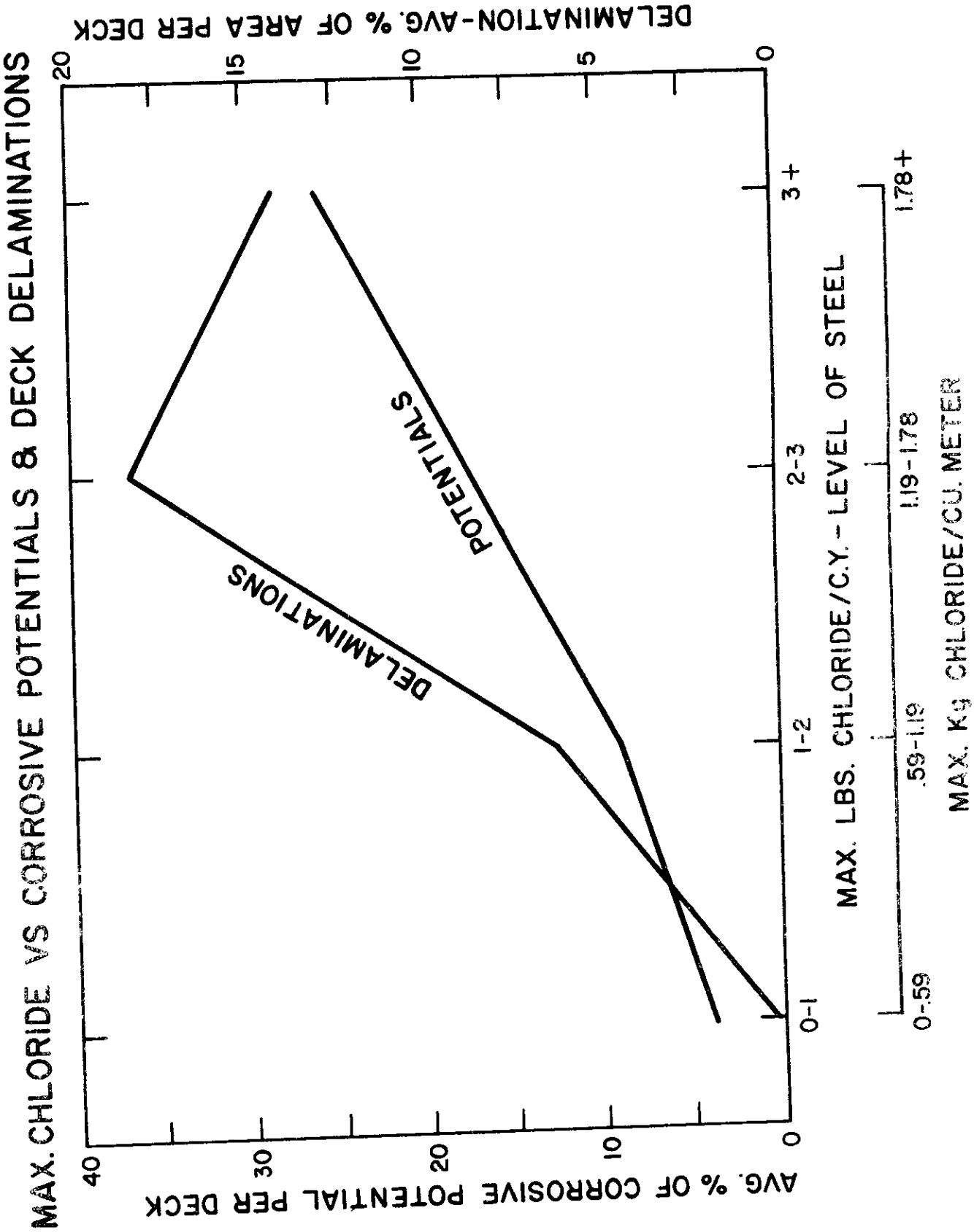
FIGURE 2



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