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THE EFFECT OF SALT IN CONCRETE ON COMPRESSIVE STRENGTH, WATER VAPOR TRANSMISSION, AND CORROSION OF REINFORCING STEEL

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THE EFFECT OF SALT IN CONCRETE ON COMPRESSIVE STRENGTH, WATER VAPOR TRANSMISSION, AND CORROSION OF REINFORCING STEEL

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Type C Final Report

by

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ABSTRACT

The purpose of this investigation was to determine the effects of sodium chloride and sea-water salts separately in concrete. The investigation covered the effects of salt on the compressive strength and water vapor transmission (WVT) of concrete, as well as the corrosive effects of salt on mild reinforcing steel. Variables included water-cement ratio, salinity of mixing water, and diameter and thickness of the specimens. The test environments included 20, 50, and 75 percent RH at 73.4 F.

The data presented herein supports the general conclusion stated in a previous report, namely, that at a mixing-water salinity of approximately 25 grams of salt per kilogram of solution, compressive strength is increased, WVT is minimized, and corrosion of mild steel is not significant.

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The Laboratory invites comment on this report, particularly on the results obtained by those who have applied the information.

CONTENTS

INTRODUCTION

The purpose of this task was to determine the effect of sea-salt spray on portland cement concrete and to determine the permissible amounts of salt in concrete when it is mixed. Specifically, it was desired to establish the separate effects of sodium chloride and sea-water salts in concrete on strength, water vapor transmission (WVT), and corrosion of mild reinforcing steel.

This report reviews the findings presented in previous reports[1, 2] and coordinates all information developed throughout the investigation. The wet-cup system is discussed in greater detail in References 3 and 4. For convenience, the mix design data in Reference 1 is included here as Appendix A. References 1 and 2 reported the results of series of tests from 360 to 520 days age. The present report extends the results of these tests up to 1000 days age and includes some data for additional series of tests.

COMPRESSIVE STRENGTH

Sodium Chloride Series

Some rather uniquely shaped curves of compressive strength versus salinity of mixing water were obtained previously[1] using NaCl. In order to verify the characteristics of these curves, two additional series of tests were performed. Plots of these data confirm the previous findings even though a different brand of cement (Victor) was used in place of the brand first used (Colton). Selected example curves of each water-cement ratio (W/C) for the Victor cement are presented in Figure 1. The data show that, for maximum compressive strength, the optimum salinity of mixing water is between 18 and 36 gm/kg for the W/C ratios used — 0.444 or 0.702.

Sea-Water Series

The effect of sea water on compressive strength of concrete also resulted in some rather unusual curves of strength versus salinity of mixing water.[1] Consequently, these tests were repeated with the same brand and type of cement except that a W/C of 0.444 was used instead of 0.702, and the salinity range was extended

from about 63 to about 88 gm/kg. Sea water with a salinity of 31.32 gm/kg and distilled water were proportioned by weight to obtain salinities less than 31.32 gm/kg. Sea-Rite salt, a simulated sea-salt mixture containing elements found in natural sea water in quantities greater than 0.004 percent, was added to sea water to obtain greater concentrations.

Figure 2 shows an example of the results of these tests. In general, all of the curves are similar to and verify the previous curves and findings. Although there were variations of compressive strength with increased sea-water salinities, there is a general increase in strength of concrete with age and with increasing salinities of mixing water up to about 88 gm/kg. This is true for Figure 2 as well as for the previous work. A report[5] recently published in England is in complete agreement with this finding.

Wall Specimens

Twelve small reinforced concrete walls were cast, the characteristics of which are described in Appendix B. One side of each wall and one-half of the cylinders for each wall (sea side) have received sea-water spray for a few minutes each day since cast (3 years). The other side of each wall and the other one-half of the cylinders for each wall (land side) received no spray.

Compressive strength values for the entire series, including two additional ages not previously reported, are presented in Table I. Compressive strength values from Table I were plotted versus age in Figure 3. Although the land-side curve is above the sea-side curve for each aggregate, indicating greater strengths for the land-side cylinders (except for the GMR high W/C ratio curve which is coincident for land and sea side), curves for both sides show increasing strength with time. The effect of poor-quality aggregate on strength of concrete can be seen in Figure 3 by comparing the GMR concrete strength to the SG and ENG concrete strengths.

WATER VAPOR TRANSMISSION

Sodium Chloride Series

Water vapor transmission (WVT) is defined as the rate of migration of water through a material, the units of which are grains per square inch per day, using a system like the wet cup shown in Figure 4.

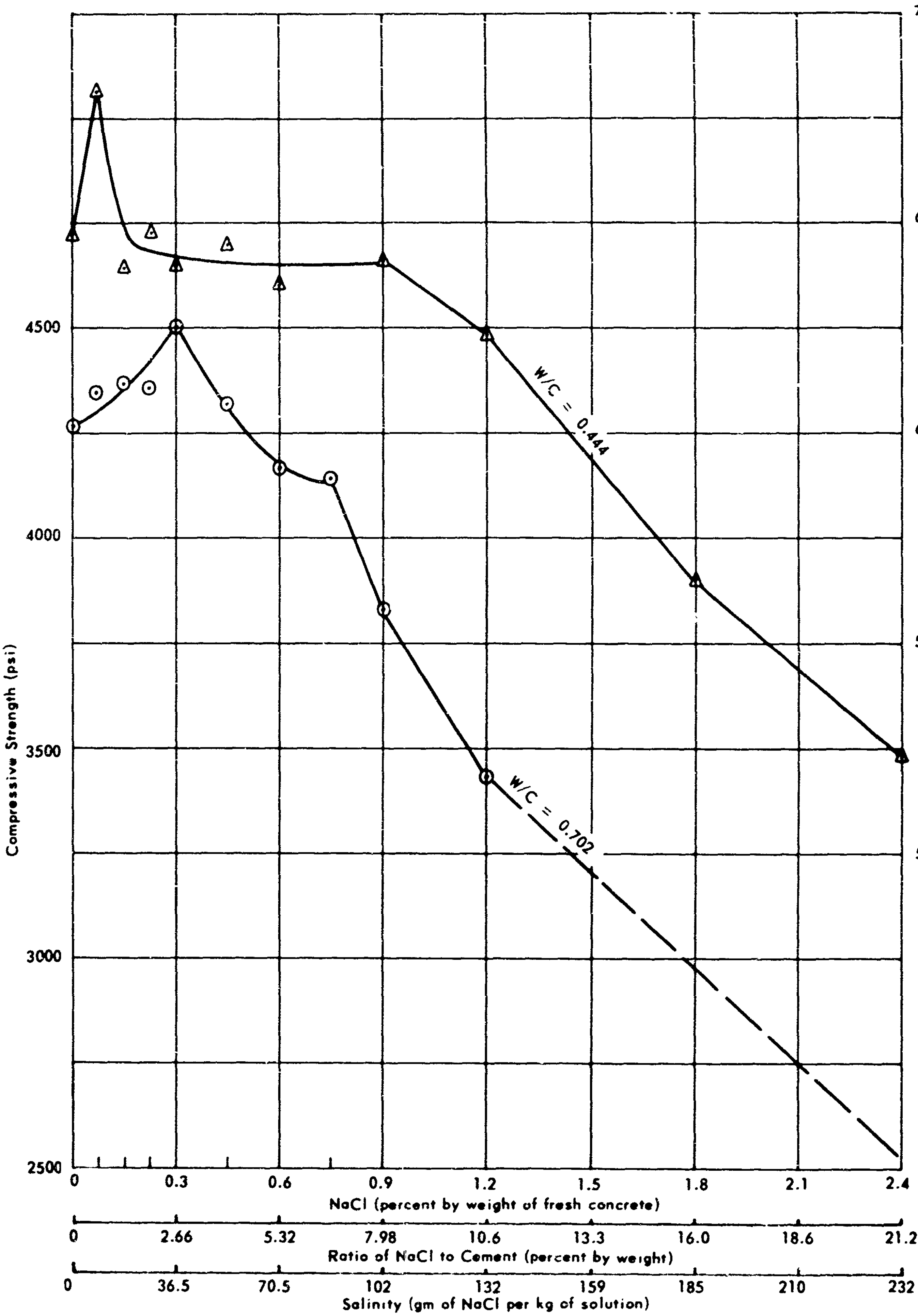


Figure 1. Compressive strength versus NaCl content for Victor cement at 14 days age.

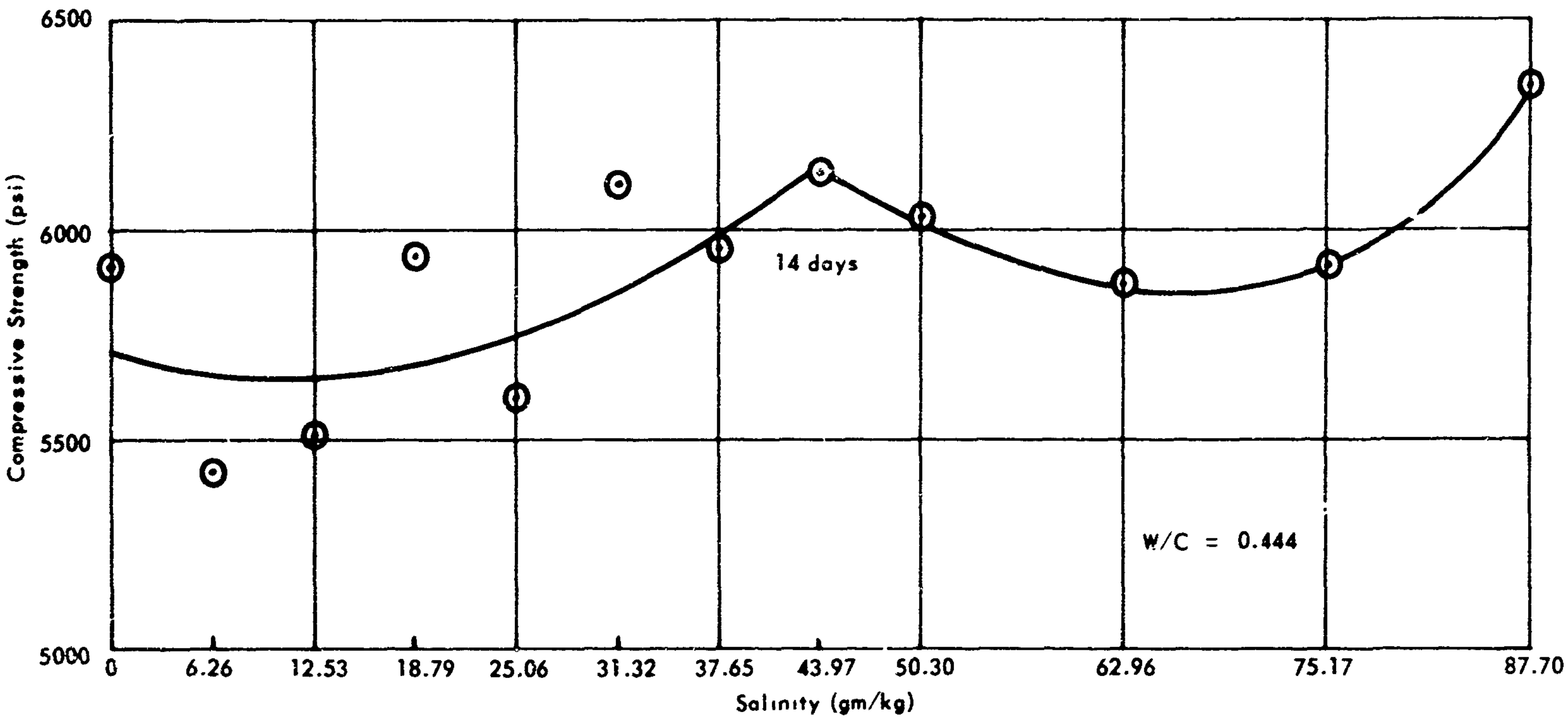


Figure 2. Compressive strength versus salinity of mixing water (sea water).

Table I. Compressive Strength of Wall Specimens

Aggregate Type[a] and W/C	Compressive Strength (psi) Age of 3-in.-dia by 6-in. concrete cylinders (days)						
	28d[b]	56d	112d	224d	448d	728d	975d
SG, 0.702							
Land Side	3330	4420	5250	4450	5990	5200	6090
Sea Side		4390	5060	4940	4600	5190	5620
SG, 0.444							
Land Side	6590	7990	7990	8140	8500	8030	9270
Sea Side		7410	7390	8460	7880	8180	8240
ENR, 0.702							
Land Side	3120	4490	4600	4510	4780	4920	6370
Sea Side		3240	5230	4550	4670	4660	4930
ENR, 0.444							
Land Side	6150	7410	7820	8150	8850	9230	9330
Sea Side		7600	7610	8110	8280	8370	8570
GMR, 0.702							
Land Side	1630	1990	2230	2040	2400	2360	2550
Sea Side		1960	2430	1980	2190	2390	2630
GMR, 0.444							
Land Side	3610	4670	4550	4840	6050	5800	6380
Sea Side		4460	5180	4880	5240	5310	5470

a/ See Appendix A.

b/ Each value is average of four fog-cured cylinders; other values are averages of three field-cured cylinders.

Note: C-Type II cement.

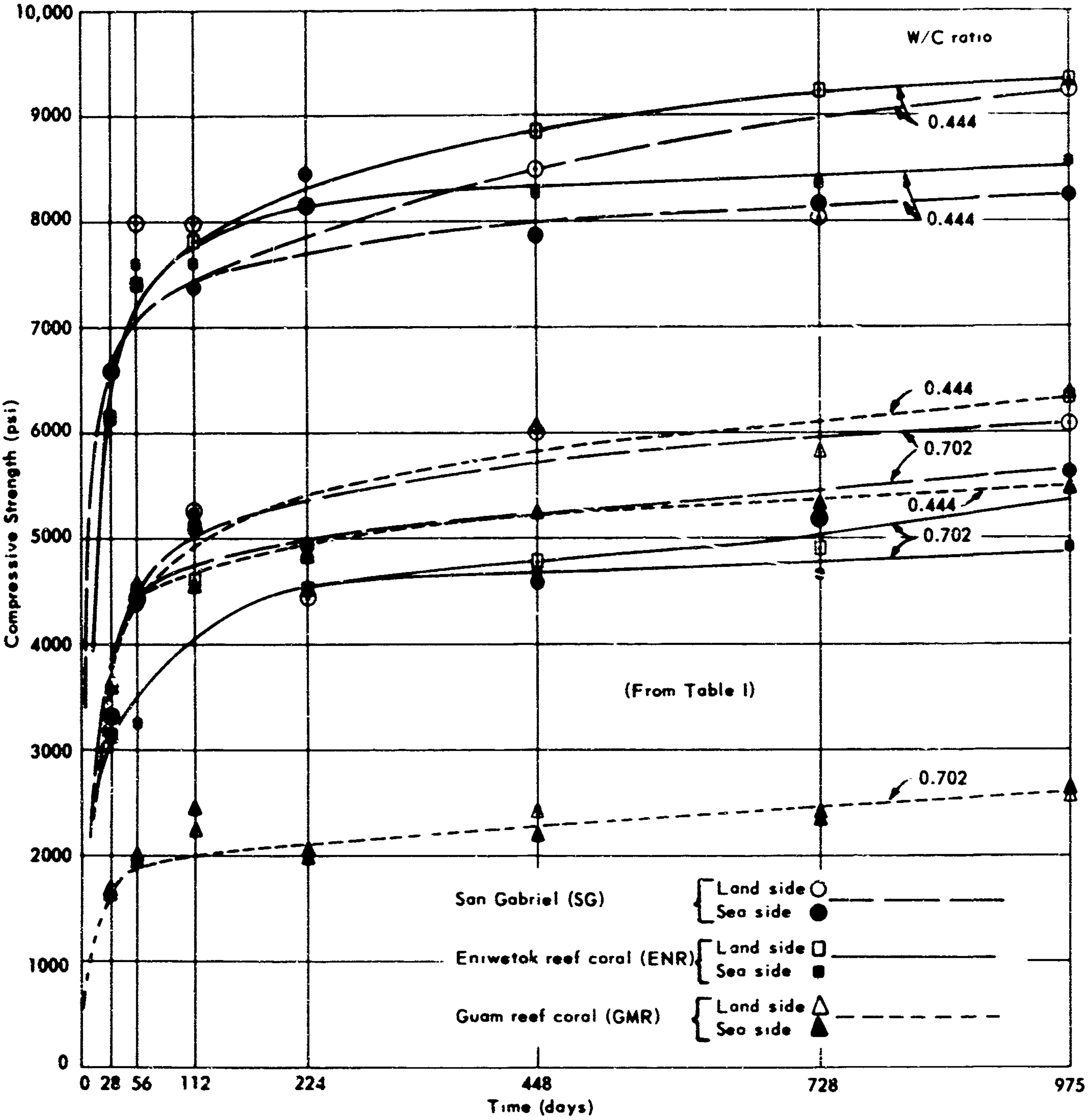


Figure 3. Compressive strength versus age of walls (from Table I).

Figure 4. Wet Cup assembled with corrosion probe added.

For the NaCl series previously reported, continued observations of the WVT values have shown no significant numerical changes. Still valid are the facts that WVT reaches a minimum value at a mixing water salinity of about 70 gm/kg, and that with further increases in salinity, the WVT values remain virtually constant.

Sea-Water Series

The values of WVT previously reported for varying salinities of mixing water using sea water have shown no changes after approximately 950 days of test.

For additional data on WVT with higher salinities of mixing water (using up to 87.70 gm/kg of sea water in the concrete), two new series of specimens were cast, and wet-cup specimens were made.

Wet cups from the first series of 12 batches (W/C = 0.444) and wet cups from the second series of eight batches (W/C = 0.702) were placed in the 20 percent RH environment, and values for WVT were obtained. Values of WVT from both

series are plotted in Figure 5 along with a curve reprinted from Reference 1 for comparison. Previous findings, that WVT decreases with an increase in strength and that WVT decreases with an increase in salinity to approximately 25 gm/kg and thereafter levels off with further increases in salinity, are verified. Being of like design, the two top curves are compatible enough to present virtually the same results.

Wall Specimens

The WVT values for wet-cup specimens fabricated at the time the walls were cast show no changes after extending the test period from 360 days to 800 days.

WVT Hysteresis

Hysteresis is herein defined as a reduction in WVT caused by aging or by previous exposure of the specimen.[6] After a test period of approximately 950 days, WVT values were computed as listed in Table II.

Figure 6 is a plot of WVT versus salinity of mixing water, with separate curves for each thickness in each RH room. The salinity at which the curves become nearly horizontal, i.e., where the WVT is not significantly changed by increased salinities, is approximately 19 gm/kg. This salinity is in fairly close agreement with the finding previously reported;[1] although the curves of Figure 6 are generally of the same order, they are lower on the WVT scale. The data wholly supports the definition of hysteresis; i.e., the WVT rate is reduced when the specimen is first exposed to an environment different from the test environment.

Vented Cups

In order to assure equal total pressure both internal and external to the wet cup, vents were installed in two cups and omitted in two cups as previously reported. After approximately 500 days age, WVT values of the four specimens were calculated to be:

1. Two 1-inch-thick specimens, one with and one without vent; WVT for each = 0.490 grains per square inch per day.

2. One 2-inch-thick specimen, with vent; WVT = 0.401 grains per square inch per day.

3. One 2-inch-thick specimen, without vent; WVT = 0.347 grains per square inch per day.

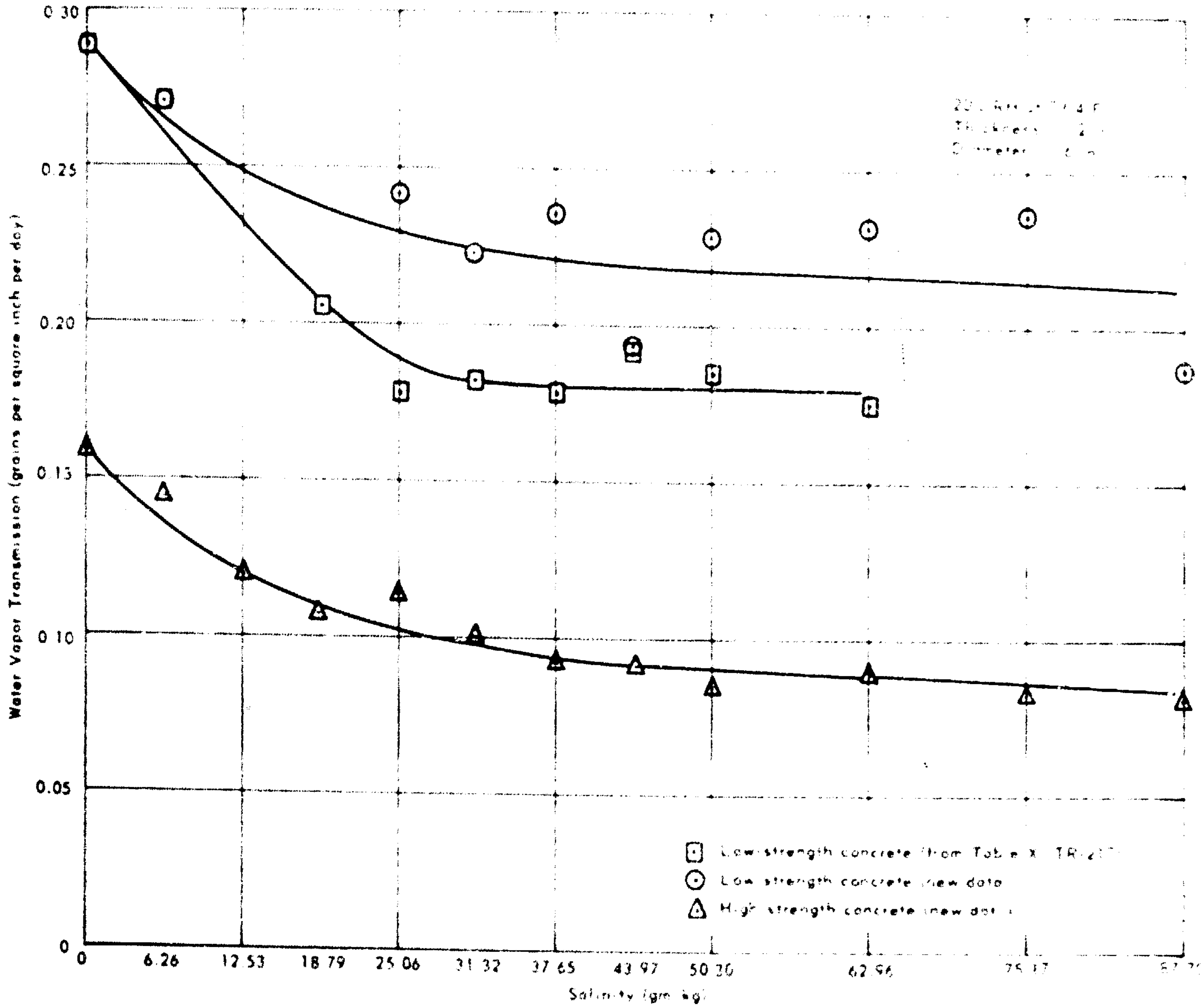


Figure 5. Water vapor transmission versus salinity of mixing water (sea water) — strength comparison.

Table II. WVT Versus Salinity of Mixing Water for 4-Inch-Nominal-Diameter Concrete Specimens With W/C = 0.702 (Sea Water and C-Type II Cement)

Batch No.	Cup No.	Sea-Water Concentration (% by weight)	Salinity (gm/kg)	l[a] (in.)	RH at 73.4 F (%)	W/t[b]	WVT[c]
S3L-2	U415-D	20	6.26	3.050	20	0.098	0.135
S3L-2	U415-C	20	6.26	1.507	20	0.154	0.213
S3L-4	U481-D	60	18.79	3.070	20	0.090	0.125
S3L-4	U481-C	60	18.79	1.500	20	0.122	0.170
S3L-6	U570-D	100	31.32	3.057	20	0.076	0.106
S3L-6	U570-C	100	31.32	1.501	20	0.118	0.164
S3L-8	U639-D	140	43.97	3.076	20	0.083	0.116
S3L-8	U639-C	140	43.97	1.490	20	0.124	0.172
S3L-10	U705-D	200	62.96	3.082	20	0.066	0.091
S3L-10	U705-C	200	62.96	1.481	20	0.124	0.172
S3L-2	U415-A	20	6.26	3.073	50	0.058	0.080
S3L-2	U415-B	20	6.26	1.516	50	0.088	0.123
S3L-4	U481-A	60	18.79	3.032	50	0.042	0.058
S3L-4	U481-B	60	18.79	1.494	50	0.069	0.096
S3L-6	U570-A	100	31.32	3.047	50	0.041	0.057
S3L-6	U570-B	100	31.32	1.514	50	0.069	0.096
S3L-8	U639-A	140	43.97	3.051	50	0.042	0.059
S3L-8	U639-B	140	43.97	1.511	50	0.069	0.095
S3L-10	U705-A	200	62.96	3.057	50	0.039	0.054
S3L-10	U705-B	200	62.96	1.522	50	0.078	0.108
S3L-2	U416-F	20	6.26	3.033	75	0.055	0.077
S3L-2	U416-E	20	6.26	1.527	75	0.085	0.118
S3L-4	U482-F	60	18.79	3.009	75	0.049	0.068
S3L-4	U482-E	60	18.79	1.516	75	0.065	0.090
S3L-6	U571-F	100	31.32	3.010	75	0.046	0.064
S3L-6	U571-E	100	31.32	1.536	75	0.063	0.087
S3L-8	U640-F	140	43.97	3.021	75	0.039	0.055
S3L-8	U640-E	140	43.97	1.568	75	0.058	0.081
S3L-10	U706-F	200	62.96	3.011	75	0.033	0.046
S3L-10	U706-E	200	62.96	1.510	75	0.063	0.087

a/ Length of flow path (or thickness of specimen).
b/ Slope of straight-line portion of graph of water loss, grams versus time in days; 950-day record.
c/ Grains per sq in. per day (1 gram = 15.43 grains).

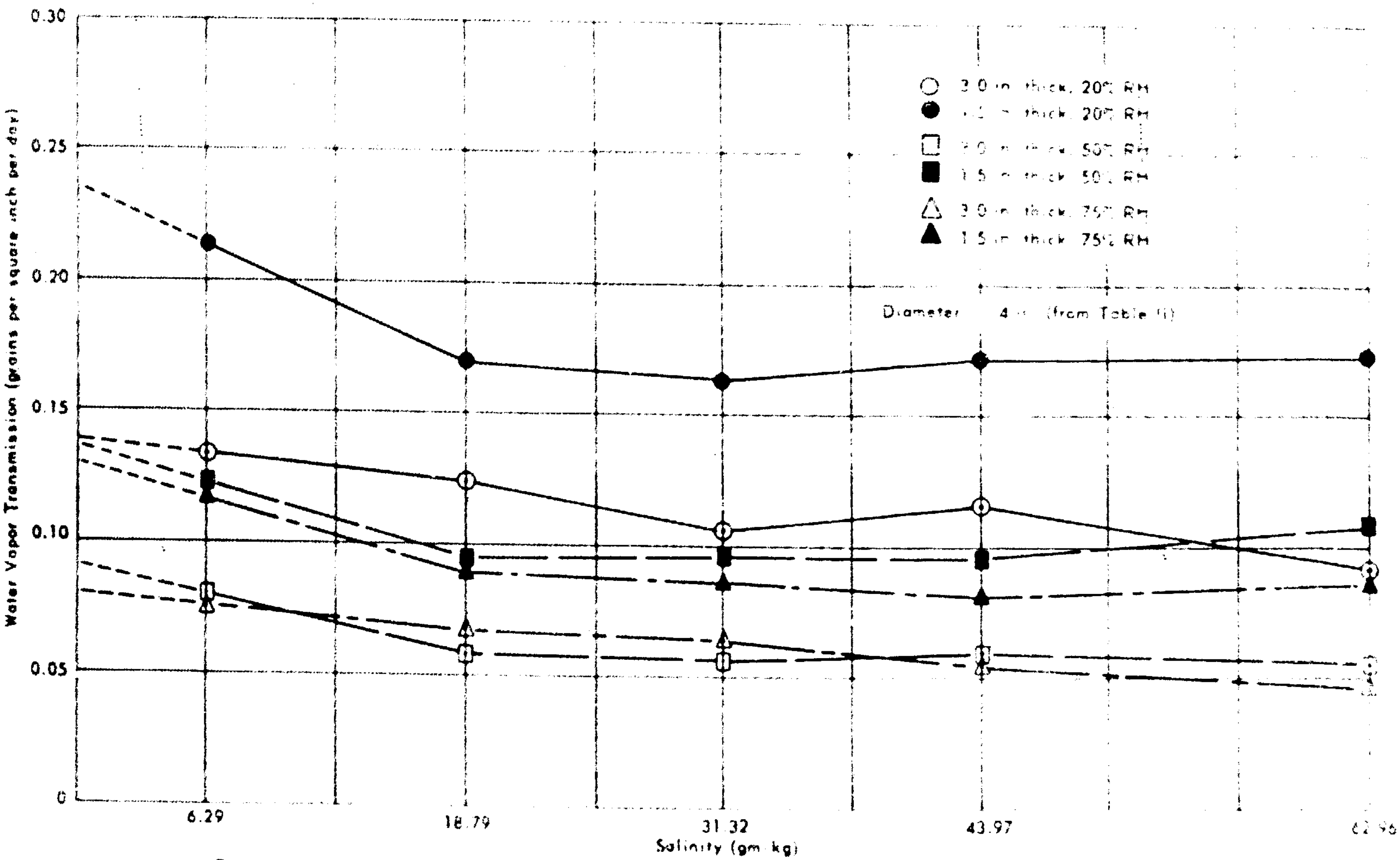


Figure 6. Water vapor transmission versus salinity of mixing water (sea water) — hysteresis.

Since these WVT differences are slight, it may be concluded that there are apparently no significant differences between the vented and unvented cups or between the 1- and 2-inch-thick specimens. If a total pressure change had occurred within the wet cups of this task, this total pressure change did not significantly affect the true WVT rates as reported.

CORROSION OF STEEL

Corrosion-Detection Probe

A complete description of the detection probe used in this task is presented in Appendix C.

Sea-Water Series

The data from the series of wet cups (like that shown in Figure 4) with probes previously reported[1, 2] remained unchanged to 860 days age, but did not reveal what might occur with higher salinities and higher-strength concrete. To obtain more information, two new series of concrete wet cups (each cup with a probe) were prepared — one high-strength (W/C = 0.444) and one low-strength (W/C = 0.702).

Twelve high-strength concrete wet-cup specimens with probe were prepared and placed in the 20 percent RH room. Probe readings were taken periodically and plotted versus time in days. These curves are presented in Figure 7 for each salinity of mixing water. From these plots the corrosion rate was calculated in mils per year (MPY), using the following equation:

$$\text{MPY} = \frac{(\Delta D)\, M\, (365)}{(\Delta T)\,(1000)} \tag{1}$$

where ΔD = change in dial divisions (probe reading) between data points, microinches (the 1000 converts this to mils)

ΔT = elapsed time between data points in days (the 365 converts this to years)

M = multiplier (for the probes of this task $M = 2$)

$\Delta D/\Delta T$ = slope of curves in Figure 7

which reduces to:

$$MPY = 0.730\,(\text{slope})$$

The maximum corrosion rates for the high-strength concrete are presented in Table III. The maximum rate is determined by using the maximum slope of the curve during the life of the test, and the maximum rate was chosen because the slope varied throughout the life of the test for most of the specimens.

Figure 8 is a plot of maximum corrosion rate versus salinity for the high-strength series. The points plotted in the figure are observed corrosion rate values; whereas, the curve drawn in the figure is a least-squares fit to the polynomial equation,

$$Y = A + Bx + Cx^2 + Dx^3 \qquad (2)$$

with a correlation coefficient, r, of 0.81. The product moment correlation coefficient, r, was computed to indicate the degree of correlation between the observed data and the computed curve. The curve in Figure 8 (high-strength concrete) develops a significant corrosion rate at lower salinities and continues at a slower rate of increase than the curve in Figure 9 (low-strength concrete). Figure 8 confirms that the most favorable salinity in high-strength sea-water concrete for the maximum corrosion rate is evidently beyond 87.70 gm/kg.

An extrapolation of the theoretical least-squares curve from a salinity of 87.70 gm/kg out to 156.65 gm/kg indicates that the point of maximum corrosion rate should occur at a salinity of approximately 119 gm/kg (dashed line in Figure 8). This is presented for interest only as there are no supporting data.

Eight low-strength concrete wet-cup specimens, each with probe, were prepared and placed in the 20 percent RH room. These have the same design as those reported in Table IV and Figure 9 except that the salinity was extended to 87.70 gm/kg. Probe readings were plotted against time in days for each salinity as shown in Figure 10. From these plots the corrosion rate in MPY was calculated using Equation 1. The maximum corrosion rates for the low-strength concrete are presented in Table V.

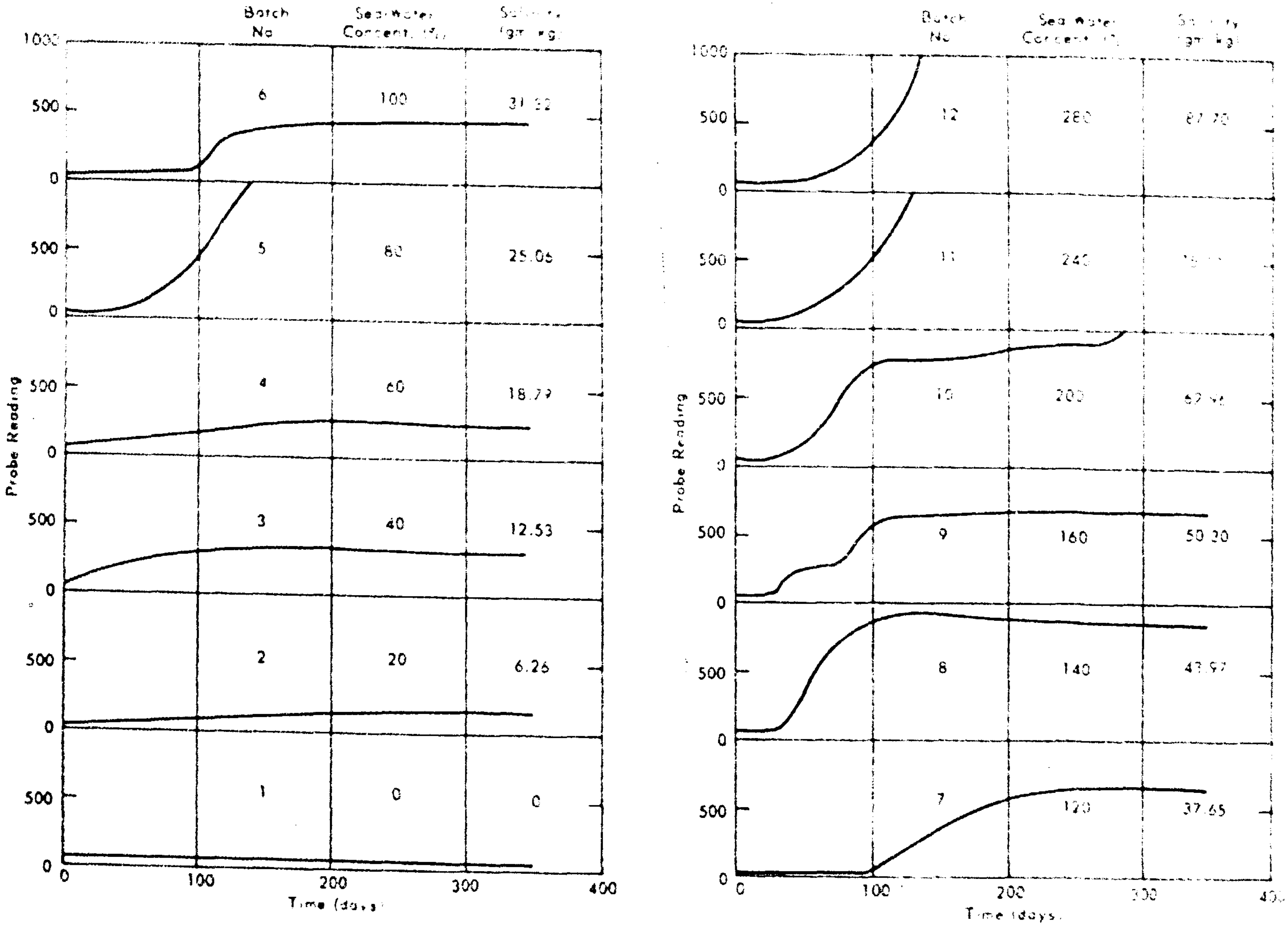


Figure 7. Probe reading versus time in days — high-strength concrete (W/C = 0.444).

Table III. Maximum Corrosion Rate of Steel in Concrete Versus Salinity of Mixing Water for W/C = 0.444 (Sea Water and C-Type II Cement)

Batch No.	Cup No.	Sea-Water Concentration (% by weight)	Salinity (gm/kg)	Time to Begin Corrosion (days)	Maximum Corrosion Rate (mils per yr)
SIH-1-C	Y67	0	0	none	0
SIH-2-C	Y98	20	6.26	28	0.3
SIH-3-C	Y129	40	12.53	29	1.1
SIH-4-C	Y160	60	18.79	28	0.9
SIH-5-C	Y191	80	25.06	28	9.8
SIH-6-C	Y239	100	31.32	28	8.8
SIH-7-C	Y270	120	37.65	74	4.6
SIH-8-C	Y301	140	43.97	31	11.7
SIH-9-C	Y332	160	50.30	28	9.4
SIH-10-C	Y363	200	62.96	33	12.6
SIH-11-C	Y394	240	75.17	30	10.7
SIH-12-C	Y425	280	87.70	46	16.3

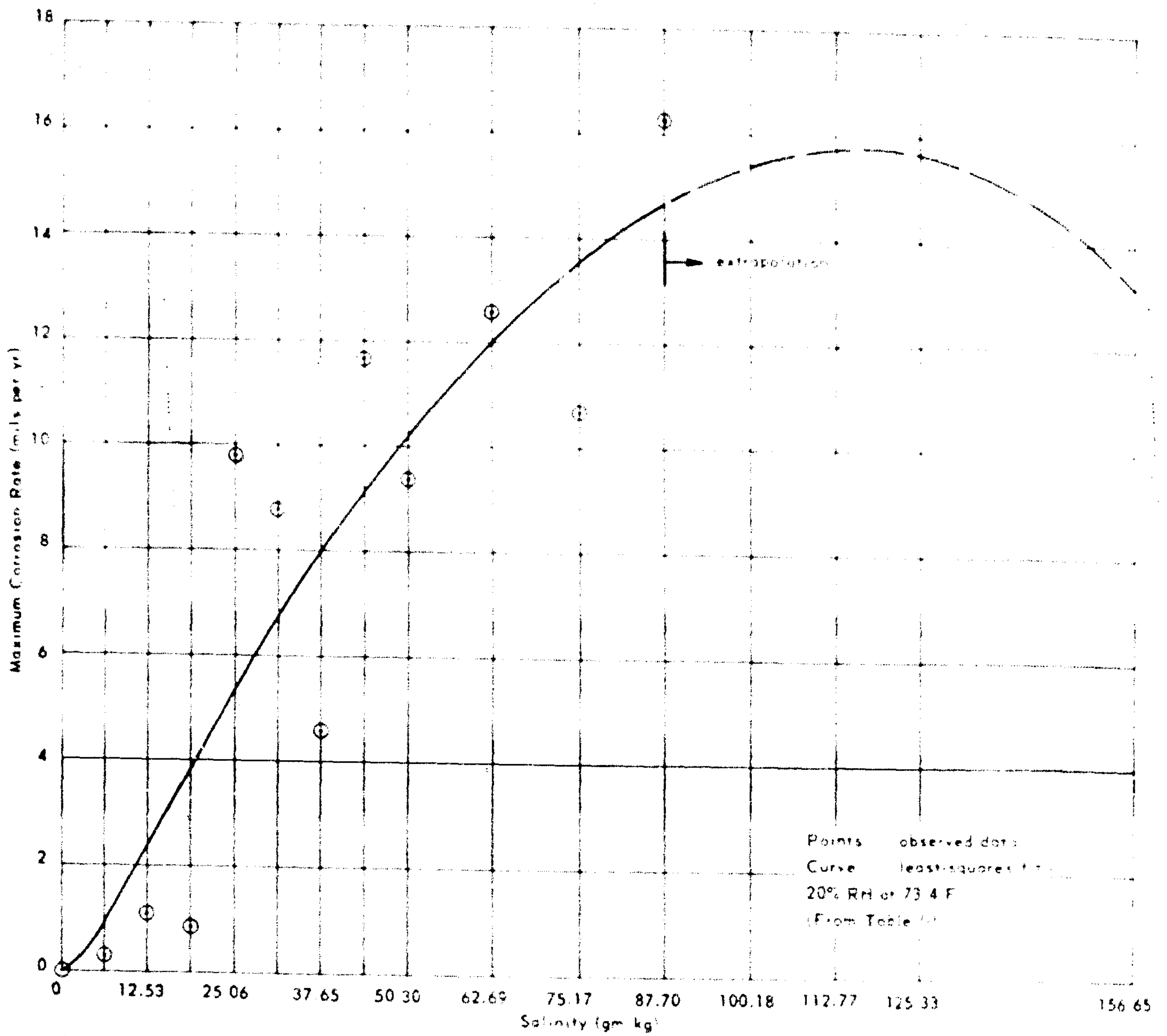


Figure 8. Corrosion versus salinity of mixing water — sea water and high-strength concrete (W/C 0. 444).

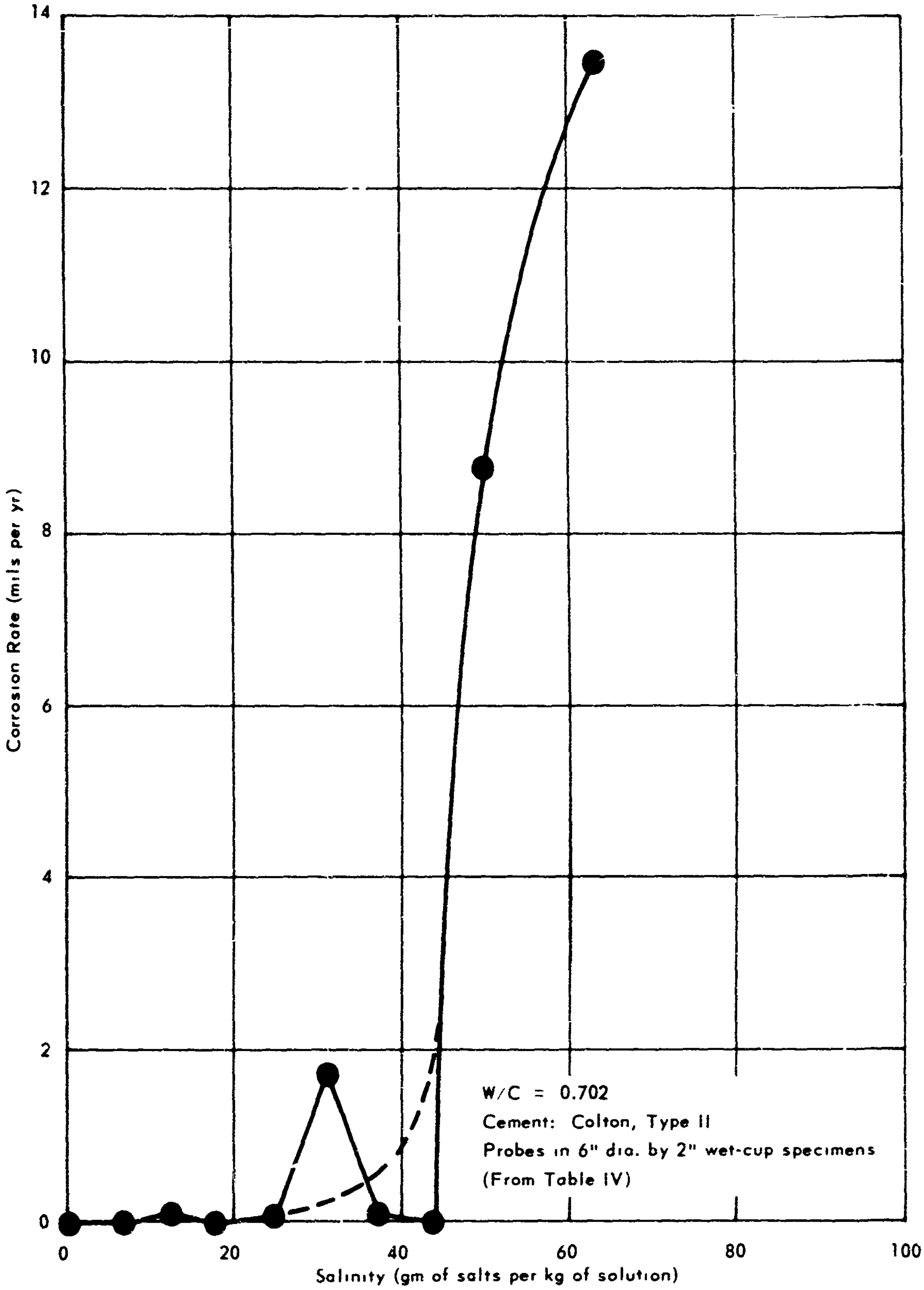


Figure 9. Corrosion versus salinity using sea water (reprinted from TR-217).

Table IV. Sea-Water Concentration Versus Corrosion of Steel in Concrete for W/C = 0.702 (Reprinted From TR-217)

Cup No.	Sea-Water Concentration (% by weight)	Salinity (gm/kg)	Time to Begin Corrosion (days)	Maximum Corrosion Rate (mils per yr)
U306	0	0	none	—
U414	20	6.26	none	—
U447	40	12.53	25	0.1
U480	60	18.79	none	—
U514	80	25.06	193	0.1
U569	100	31.32	360	1.7
U605	120	37.65	60	0.1
U638	140	43.97	none	—
U671	160	50.30	25	8.8
U704	200	62.96	315	13.5

Table V. Maximum Corrosion Rate of Steel in Concrete Versus Salinity of Mixing Water for W/C = 0.702 (Sea Water and C-Type II Cement)

Batch No.	Cup No.	Sea-Water Concentration (% by weight)	Salinity (gm/kg)	Time to Begin Corrosion (days)	Maximum Corrosion Rate (mils per yr)
S3L-5-C	Y611	80	25.06	15	0.7
S3L-6-C	Y612	100	31.32	22	10.1
S3L-7-C	Y613	120	37.65	76	12.7
S3L-8-C	Y614	140	43.97	16	16.0
S3L-9-C	Y615	160	50.30	146	5.7
S3L-10-C	Y616	200	62.96	61	18.7
S3L-11-C	Y617	240	75.17	56	42.2
S3L-12-C	Y618	280	87.70	16	23.8

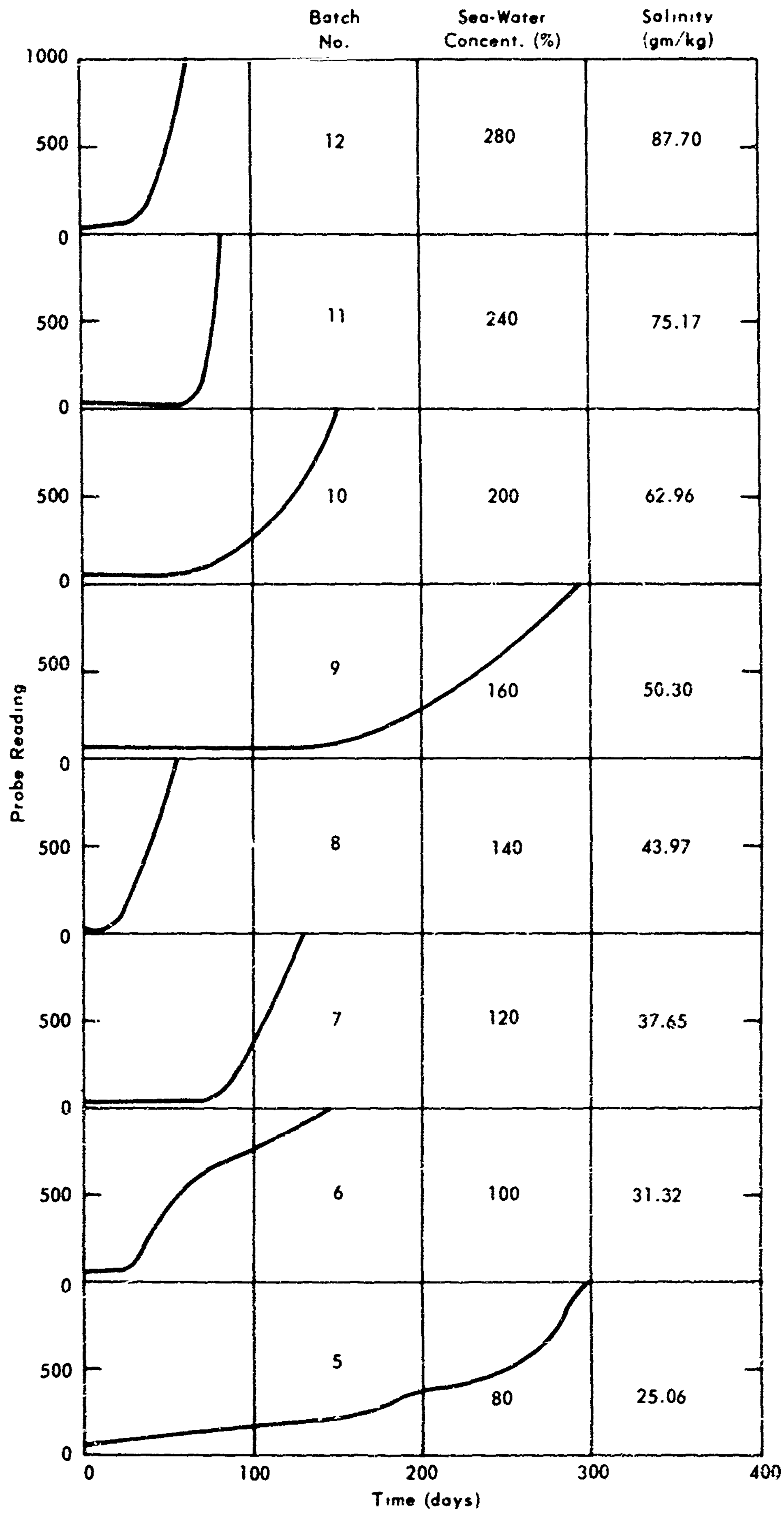


Figure 10. Probe reading versus time in days — low-strength concrete (W/C = 0.702).

The corrosion data in Tables IV and V were each subjected to the least-squares method of curve fitting. The r value for each set of data was computed to be 0.89 and 0.67, respectively. Then the two sets of data were both fitted to the same curve by the least-squares method. The r value for the combined data set was 0.70.

An analysis of the variance of each set of data with the computed curve was conducted using the F-test:

$$F = \frac{s_1^2}{s_2^2} = \frac{58.7}{19.4} = 3.02$$

where s_1^2 = variance of new data about the curve

s_2^2 = variance of old data about the curve

and where $F_{95(7.9)} = 3.29$

$F_{05(7.9)} = 0.27$

which indicates that there is no significant difference in the variances at the 90 percent confidence level.

With this degree of confidence, the data points from Tables IV and V were plotted (Figure 11) as maximum corrosion rate versus salinity for the low-strength series. The points in the figure are observed corrosion-rate values; whereas, the curve represents the least-squares fit of Equation 2, with r value of 0.70.

An extrapolation of the theoretical least-squares curve in Figure 11 (like that in Figure 8) indicates that the point of maximum corrosion rate should occur at a salinity of approximately 100 gm/kg. Again, this extrapolation has no data to support it, but is presented for interest.

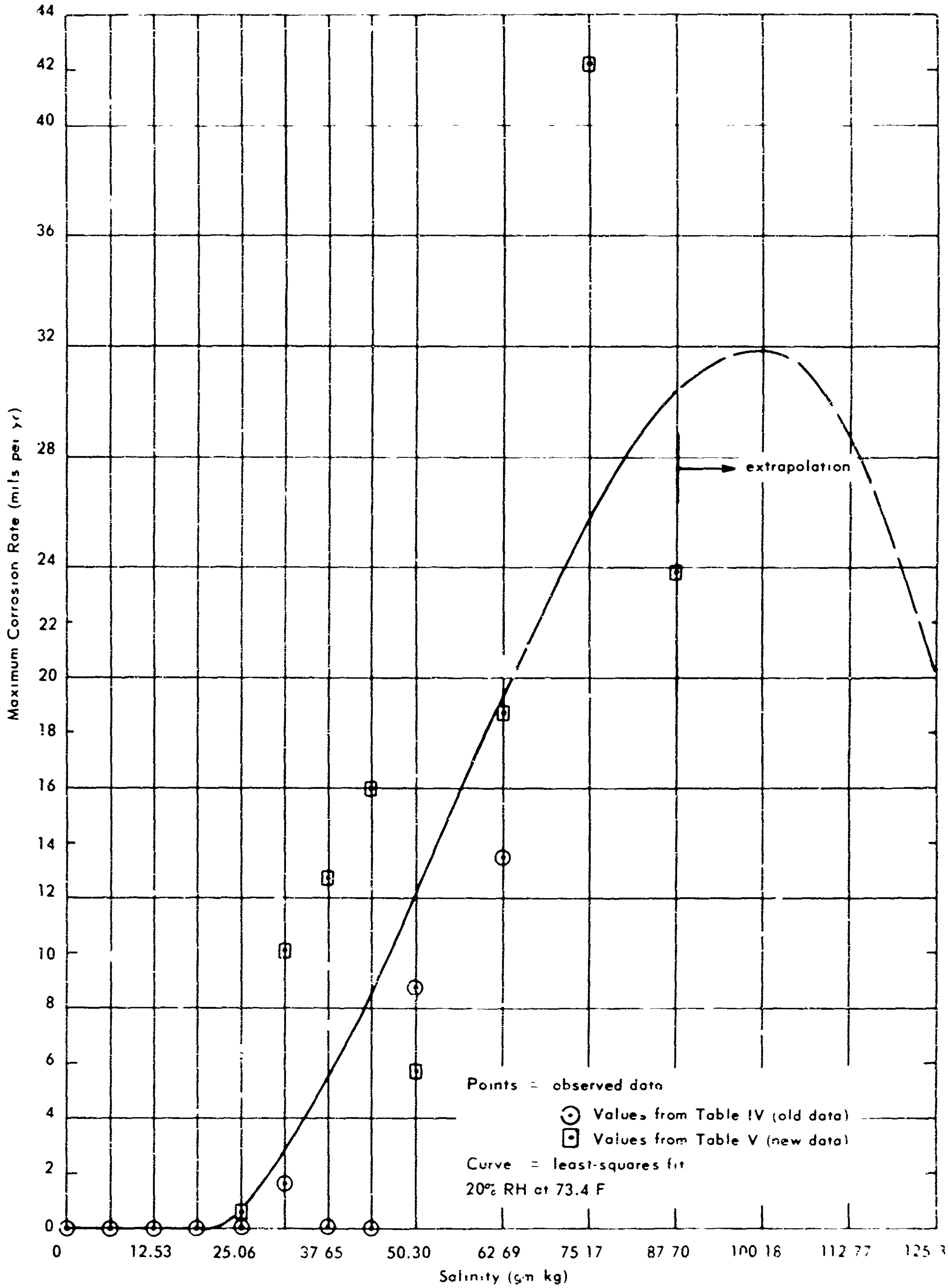


Figure 11. Corrosion versus salinity of mixing water — sea water and low-strength concrete (W/C = 0.702).

Figure 11 agrees with the findings as indicated in Figure 9 in that no significant corrosion occurs up to a salinity of approximately 25 gm/kg; thereafter the rate of increase of the maximum corrosion rate for the low-strength concrete is about 1 MPY for every 2 salinity units. Figure 8 shows that the rate of increase of the maximum corrosion rate for the high-strength concrete is about 1 MPY for every 5 salinity units. This indicates that the high-strength concrete offers 2-1/2 times more resistance to corrosion of mild steel than does the low-strength concrete. This indication also adds support to the extrapolated information that the point of maximum corrosion rate demands a higher salinity for the high-strength concrete (119 gm/kg) than for the low-strength concrete (100 gm/kg).

Depth-of-Cover Series

A series of 6-inch-thick concrete specimens of two different strengths (W/C = 0.444 and 0.702) were prepared as wet cups with probes. Measuring from the bottom surface of the specimen, the probes had concrete cover depths of 1, 2, 3, 4, and 5 inches. These cups have been stored in 50 percent RH at 73.4 F for approximately 1000 days with no significant or measurable corrosion occurring. This is to be expected since the salinity of the mixing water is zero.

Experimental Walls

A very complete description of the walls is reprinted in Appendix B. One side of the walls has been sprayed daily with sea water for approximately 3 years.

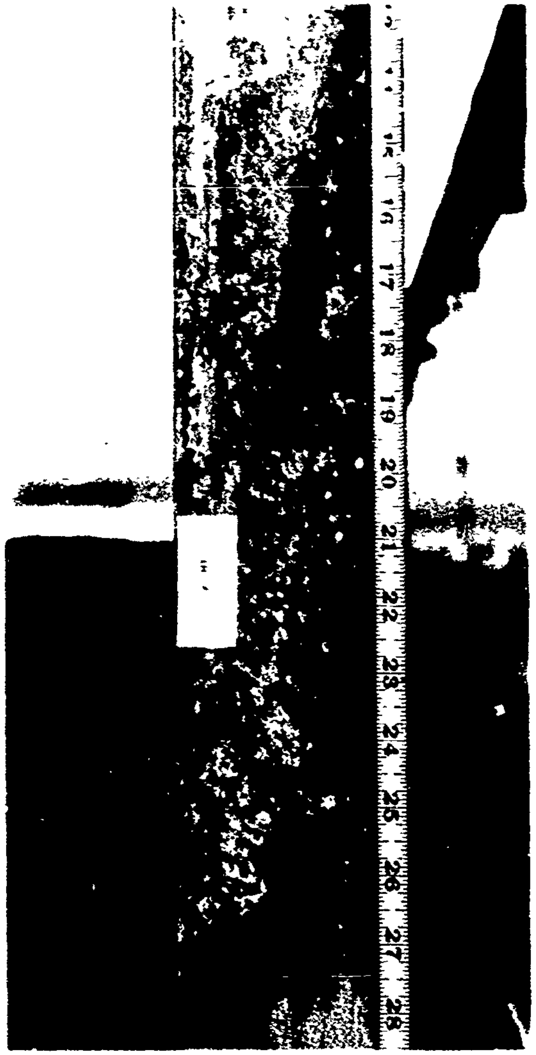
When the walls were 22 months old, the first observable cracks appeared in the vertical end faces of Walls 1, 2, and 10. Walls 2 and 10 are low-strength SG and ENR, respectively, with welded steel and 1-inch cover (thinner walls). Wall 1 is high-strength GMR with tied steel and 1-inch cover. Figure 3 shows that at the age of approximately 660 days the low-strength SG and ENR and the high-strength GMR concrete all had compressive strengths ranging from 4500 to 6000 psi. This range was even smaller at lower ages. Since these three designs began to show deterioration at approximately the same time, they appear to have somewhat the same resistance to deterioration. It would be expected that the thin-wall low-strength GMR design would show cracking also, but it has not. The thicker less-resistant and the thinner more-resistant designs have not shown cracking. The sprayed face of each of the walls has shown varying degrees of spalling.

Figure 12 shows the cracks in Walls 1, 2, and 10 after 34 months of testing, or 12 months after the cracks first formed. The sprayed surfaces of the walls face south. Over this 12-month period, the propagation rate of the crack development in each face shown in Figure 12 is:

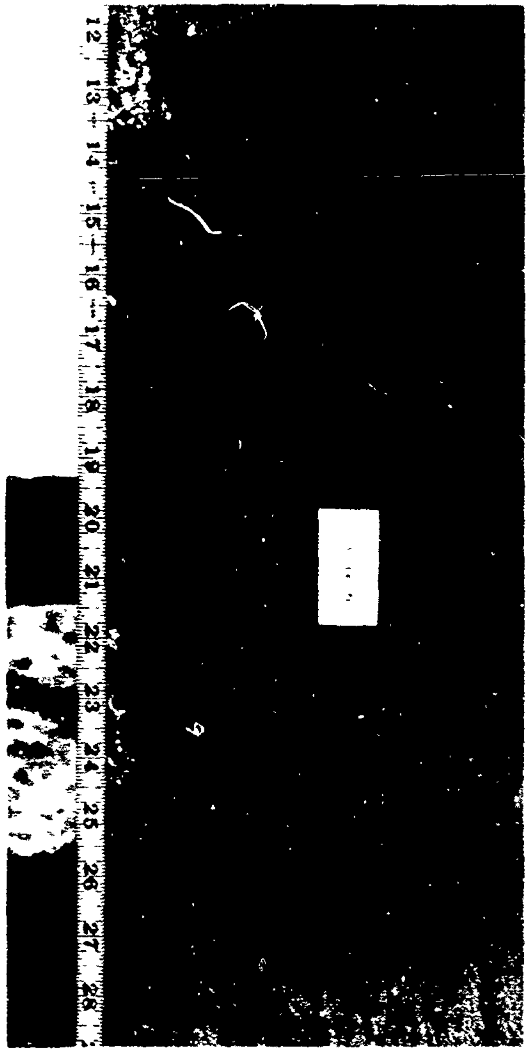
Wall	Propagation Rate (inches per month)
No. 1 East	3/4
No. 1 Front	1
No. 2 East	1
No. 2 West	1-1/16
No. 10 East	1-1/12
No. 10 West	1/3

Apparently the repeated wetting and drying of the wall surface sprayed with sea water assisted the movement of salts into the concrete, causing the steel to corrode. The basic cause of this corrosion (oxidation) is the instability of the iron in its refined form. The oxide of iron requires a greater volume than the original metal occupied thereby inducing tensile forces in the surrounding concrete causing the concrete to expand. Since concrete has a very low tensile strength, its failure by cracking serves to relieve the tensile forces. Even the smallest cracks propagate additional corrosion and further cracking by exposing the metal to outside elements. As would be expected, the cracks thus far developed in the end faces of the walls are in the same plane as the steel grid. One reason the top faces have not developed cracks, as the end faces have, is that each side face receives about half as much sunshine as the top face. The same is true of the front faces, except for the front face of Wall 1 adjacent to the east end face (Figure 12) which did develop a 10-inch-long crack.

These wall exposure tests and the compressive-strength tests (Figure 3) verify the finding that high-quality aggregate and high-strength concrete are essential if reinforced concrete structures are to resist the deteriorating effect of sea water and sea salts.



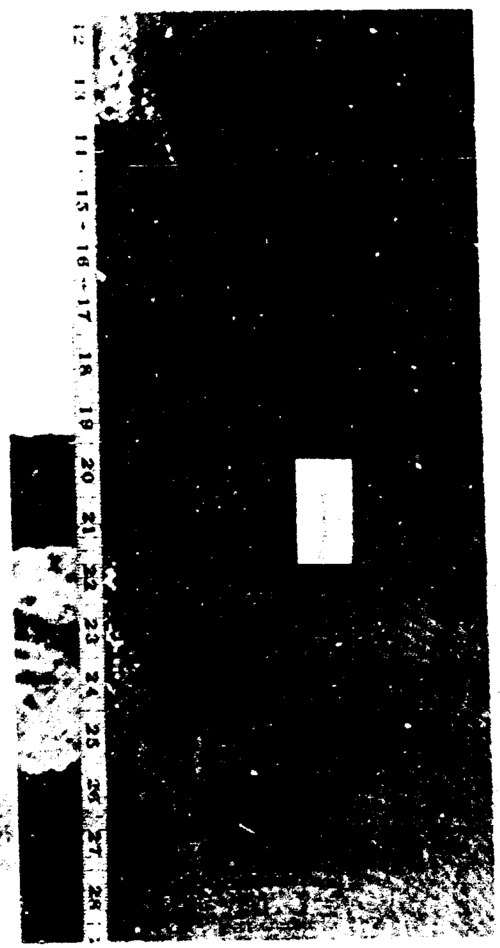
East end of Wall 1



Sprayed surface adjacent
to east end of Wall 1

East end of Wall 2

East end of Wall 1



Sprayed surface adjacent to east end of Wall 1

East end of Wall 2

GENERAL SUMMARY

The new and additional data as reported herein has not changed the findings last reported[1, 2] but has expounded and generally verified them, with the following exception.

Some salt may be beneficial to concrete in some respects. Although the new and additional data have not changed the findings about strength or WVT of concrete, the corrosion rates are affected slightly by the findings of the new high-strength series of specimens with probe . It is not possible to state what is or is not a significant corrosion rate in practice (actual construction), based on limited laboratory tests alone. But it can be stated that, within the bounds of this investigation, the higher the concrete strength the lower the salinity necessary to have a significant corrosion rate occur.

FINDINGS AND CONCLUSIONS

It is a general conclusion that a small amount of sea-water salts may be beneficial to concrete if rigid controls are exercised. At a mixing-water salinity of approximately 25 gm/kg, strength is improved, WVT is minimized, and for the low-strength concrete of this investigation corrosion of mild steel is negligible.

Compressive Strength

1. Maximum compressive strength occurs between salinities of 18 and 36 gm/kg for concrete incorporating NaCl in the mixing water.

2. Depending upon the age of concrete, relatively high salinities of NaCl can occur before strengths become less than those for zero salinity.

3. Natural sea water does not deteriorate plain concrete; strength is generally increased with salinities up to nearly three times that of natural sea water.

Water Vapor Transmission

1. With increasing NaCl salinity of mixing water up to 70 gm/kg, and of sea water up to a range of 19 to 31 gm/kg, WVT is reduced to its minimum for all thicknesses studied. With further increasing salinities, WVT is almost constant.

2. With increasing salinities, the effect of the thickness of the specimen becomes much more pronounced; thicker specimens show much greater reductions in WVT than do thinner specimens.

3. With or without salts (NaCl or sea-water salts), WVT varies inversely with compressive strength.

4. Without salts, thinner concrete specimens have somewhat higher WVT rates than do thicker specimens. Indications are that 6 inches may be a limiting thickness beyond which greater thicknesses do not further decrease WVT significantly.

5. Concrete made with poor-quality aggregate (GMR) shows higher WVT rates than concrete made with high-quality aggregate (SG and ENR), other factors being equal.

6. This investigation does not confirm nor does it completely negate an earlier finding[3] that an ambient RH of 20, 50, or 75 percent at 73.4 F has no significant effect on WVT compared with experimental variability.

Corrosion

1. The electrical-resistance corrosion-rate detection probe appears to be a reliable device for measuring the corrosive effect of concrete on mild reinforcing steel.

2. Concrete with NaCl in the mixing water shows an optimum salinity of about 70 gm/kg for maximum corrosion of mild steel.

3. When sea water is used in concrete mixing water, the salinity at which mild steel develops a significant corrosion rate appears to vary directly with the W/C. Within the limits of this investigation, for concrete with a low W/C, the maximum corrosion rate increased rapidly at lower salinities. For higher salinities the rate of increase of the maximum corrosion rate was 2-1/2 times slower for concrete with a low W/C.

REFERENCES

1. U. S. Naval Civil Engineering Laboratory. Technical Report R-217, The Effect of Salt in Concrete on Compressive Strength, Water Vapor Transmission, and Corrosion of Reinforcing Steel, by D. F. Griffin and R. L. Henry. Port Hueneme, California, 12 November 1962.

2. Griffin, D. F., and R. L. Henry. "The Effect of Salt in Concrete on Compressive Strength, Water Vapor Transmission, and Corrosion of Reinforcing Steel," American Society for Testing and Materials, Proceedings, Vol 63, 1963, pp 1046-1078.

3. U. S. Naval Civil Engineering Laboratory. Technical Report R-130, Water Vapor Transmission of Plain Concrete, by D. F. Griffin and R. L. Henry. Port Hueneme, California, 16 May 1961.

4. Griffin, D. F., and R. L. Henry. "Integral Sodium Chloride Effects on Strength, Water Vapor Transmission, and Efflorescence of Concrete," Journal of American Concrete Institute, No. 6 (December 1961), pp 751-772.

5. Cement and Concrete Association. TRA/374, The Workability and Compressive Strength of Concrete Made With Sea Water, by J. D. Dewar. London, December 1963.

6. ASTM Committee C-16, "Water Vapor Transmission Testing," Bulletin of ASTM, Materials Research and Standards, Vol 1, No. 2 (February 196'), pp 117-12'.

7. U. S. Naval Civil Engineering Laboratory. Technical Report R-057, Evaluation of Condensate Return Line Corrosion Testers, by J. M. Hayhoe and N. L. Slover. Port Hueneme, California, 13 March 1961.

Appendix A

MIX DESIGN DATA

Table A-1. Summary of Concrete Mix Design Data
(Reprinted From TR-217)

A. Characteristics of Materials

Cements: Colton, Type II; Colton, Type III; Victor, Type III.

Aggregates:
1. SG or San Gabriel, a reference aggregate from the Irwindale Pit of Consolidated Rock Products Co., Calif.
2. ENR or Eniwetok Atoll reef coral from Eniwetok Islet, Marshall Islands; a good-quality coral consisting of 95 percent coralline limestone.
3. GMR or Guam reef coral from Apra Harbor, Guam, the Marianas; a poor-quality coral consisting of 96 percent cellular coral or porous coralline rock.
4. Specific characteristics:

Item	SG		ENR		GMR	
	C	F	C	F	C	F
Bulk Sp Gr (oven-dry)	2.66*	2.63*	Overall Avg 2.49	2.49	Weighted Avg L: 2.16 H: 2.11**	
24-hr abs, percent	1.6*	1.8*	2.32	2.32	20.0*	7.6*
Comparative general physical condition, percent by weight	Reference aggregate, approx. 100 percent good		73 Good 73 25 Fair 25 2 Poor 2		49 Good 45 45 Fair 45 6 Poor 10	

*Determined by Task Engineers.
Data without (*) taken from NCEL TN-335A.

**L = W/C ratio of 0.702.
H = W/C ratio of 0.444.

Note: San Gabriel aggregate was used unless otherwise indicated.

Table A-1. Summary of Concrete Mix Design Data (Cont'd)

5. Grading: Pounds retained on each sieve per 6.0± cu-ft batch

Sieve	SG		ENR		GMR	
	H	L	H	L	H	L
3/4 in.	19.7	16.5	0	0	0	0
3/8 in.	197.6	196.6	202.0	193.8	179.6	172.4
No. 4	125.1	121.0	116.4	110.0	103.4	98.0
No. 8	72.5	85.9	67.2	77.8	59.8	69.6
No. 16	65.9	82.9	61.2	75.2	54.4	67.0
No. 30	72.5	83.5	67.2	76.0	59.8	67.6
No. 50	59.2	76.2	55.2	69.4	49.0	61.8
No. 100	33.1	38.7	30.6	35.4	27.2	31.2
Pan	13.3	18.7	12.2	17.0	10.8	15.2
Total	658.9	720.0	612.0	654.6	544.0	582.8

Water: Port Hueneme tap water at 73.4 F.

Chemical analysis (ppm): hydroxide (0.0); carbonate (0.0); bicarbonate (137.0); chlorides (62.0); calcium (38.0); magnesium (14.6); sulphate (465.0); sodium and potassium (219.0).

Slump: Designed for 3 in.

B. Batch of 1.0 ± cu ft consisted of the following:

Mix Design Factors	SG		ENR		GMR	
	H	L	H	L	H	L
W/C	0.444	0.702	0.444	0.702	0.444	0.702
Water (lb)a/	11.7	11.7	13.4	13.8	13.4	13.8
Cement (lb)	26.4	16.7	30.1	19.7	30.1	19.7
Cement Factorb/	7.62	4.81	8.65	5.66	8.65	5.66
Aggregate (lb)	109.9	120.0	102.0	109.0	90.7	97.2

a/ The quantity of water added at the mixer was corrected for moisture present in the aggregate and moisture required for absorption.

b/ Sacks per cu yd of concrete.

Appendix B

EXPERIMENTAL WALLS
(All the material in this appendix
is reprinted from TR-217)

Experimental Walls

In order to simulate a marine environmental exposure such as that encountered by a building on a tropic atoll, small walls of reinforced concrete were built and sea-water spray was applied daily. The details of the concrete mixes and their ingredients are shown in Table A-I, Appendix A.

The variables to be investigated are shown in Table B-I. The three types of aggregate described in Table A-I and two water-cement ratios (0.444 and 0.702) were used, and two different reinforcing-steel arrangements were employed. Each of the steel arrangements had the same grid spacings. The only difference in arrangement was that in one case the mild-steel bars were insulated from each other with electricians' plastic tape and tied together with nylon fishing cord, as shown in detail in Figure B-1. In the other arrangement all steel bars were tack-welded together at the point of contact. These arrangements were to permit the investigation of corrosion of individual steel bars as compared to the corrosion of an interconnected grid of steel bars. Two depths of steel cover were also included as variables, the depth being measured from the outer surface of the concrete to the nearest surface of the steel. It was desired to eliminate all external entries, such as tie-wire and bolt holes, through the concrete to the steel as unknown variables; therefore, concrete blocks for the particular kind of concrete to encase the steel grids were precast with wire loops embedded in each block for securing the steel grids. These are shown in Figure B-2. In this way there were no steel wires going from the steel grid through the concrete to the outside. The use of spacer blocks was avoided for the same reason, and the grids were held in place vertically by means of temporary holding devices which may be seen in Figure B-3.

It was desired to hold the W/C for the high-strength concretes equal throughout and that for the low-strength concretes equal throughout. It was further desired to design all concretes for identical consistencies as measured by the slump test. This required a greater amount of water and cement for the coral aggregate concretes than for the San Gabriel aggregate concrete.

Table B-1. Var

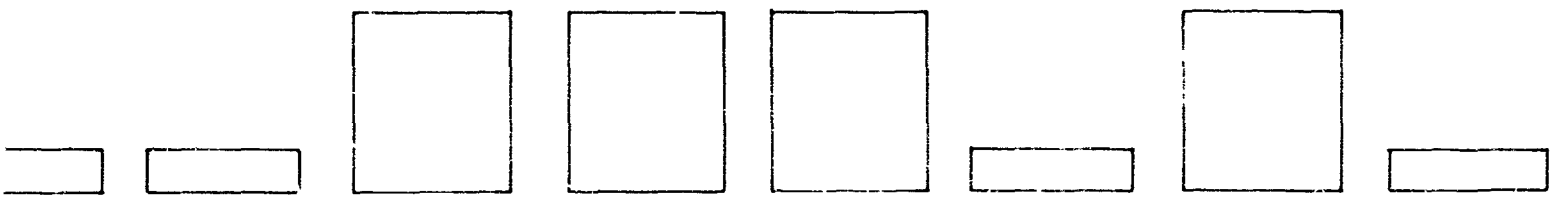
Statistical Factors \ Wall Configuration (Plan View)					
Wall Number	1	2	3	4	5
Aggregate[a]	GMR	SG	ENR	SG	ENR
Strength[b]	Hi	Lo	Lo	Hi	Hi
Steel Arrangement[c]	T	W	T	W	T
Depth of Steel (cover)	1"	1"	6"	6"	1"
Wall Thickness	3-1/2"	3-1/2"	13-1/2"	13-1/2"	3-1/2"
Casting Date (1961)	15 May	8 May	11 May	8 May	11 May
Cylinder Test Dates	10 Jul '61 5 Sep '61 26 Dec '61	3 Jul '61 28 Aug '61 18 Dec '61	6 Jul '61 31 Aug '61 21 Dec '61	3 Jul '61 28 Aug '61 18 Dec '61	6 Jul '61 31 Aug '61 21 Dec '61

a/ See Table I

b/ Hi = High W/C = 0.444
Lo = Low W/C = 0.702

c/ T = Tied (and steel contacts insulated)
W = Welded

-1. Variables for Experimental Walls



5	6	7	8	9	10	11	12
NR	GMR	ENR	SG	GMR	ENR	GMR	SG
Hi	Lo	Hi	Lo	Lo	Lo	Hi	Hi
T	W	W	T	T	W	W	T
1"	1"	6"	6"	6"	1"	6"	1"
1/2"	3-1/2"	13-1/2"	13-1/2"	13-1/2"	3-1/2"	13-1/2"	3-1/2"
May	15 May	11 May	8 May	15 May	11 May	15 May	8 May
l '61 ug '61 ec '61	10 Jul '61 5 Sep '61 26 Dec '61	2 Aug '62 11 May '63 10 Jan '64	30 Jul '62 8 May '63 8 Jan '64	6 Aug '62 15 May '63 15 Jan '64	2 Aug '62 11 May '63 10 Jan '64	6 Aug '62 15 May '63 15 Jan '64	30 Jul '62 8 May '63 8 Jan '64

While the slump test has for a number of years appeared to be an excellent measure of consistency for San Gabriel aggregate concrete, coral aggregate concretes do not respond quite so uniformly to the slump test. Although distribution of aggregate by percentage was approximately the same for all batches, the coral concretes appeared too harsh, compared with the San Gabriel aggregate concrete. The slumps in inches for the various concretes used in the walls are tabulated below:

Wall No.	Min	Avg	Max	W/C	Aggregate
6, 9	4-1/2	6-1/2	8	0.702	GMR
2, 8	1	2	3	0.702	SG
3, 10	1-1/2	5	6-1/2	0.702	ENR
1, 11	1/2	1-1/2	3	0.444	GMR
4, 12	3	3-1/2	4-1/2	0.444	SG
5, 7	3	4-1/2	6	0.444	ENR

The apparent harshness of the coral concretes was further demonstrated in the construction of walls when the forms were stripped at 7 days age. For example, Figure B-4 shows a typical San Gabriel concrete wall immediately after the forms were stripped and Figure B-5 shows a typical coral concrete wall after the forms were stripped. Although the apparent consistencies were the same, the coral concretes did not flow down into the foundation of the wall as well as did the San Gabriel concrete. This occurred even though the spud vibrator was applied internally to the concrete as well as externally to the form.

During the first 7 days of curing, the forms of each wall remained in place, and burlap was placed across the top of the walls for moisture retention. The burlap was kept wet by means of water spray. On the 7th day, after the forms were stripped from each wall, the voids, as shown in Figure B-5, were filled in with appropriately designed fresh concrete. On the 8th day each wall was rubbed down with mortar employing the same fine sand that had been used in the concrete mix for each particular wall. The walls were then completely wrapped in burlap, which was kept soaking wet. On the 28th day after placing, the walls were stripped of burlap, and 3-inch-diameter by 6-inch concrete cylinders for determining compressive strength were placed along the walls as shown in Figure B-6. When the last walls to be cast (GMR) had achieved an age of 30 days, the first application of sea water was made to the seaward side of each wall on 15 June 1961 for a 5-minute interval once each day at 0830. The sea water was obtained from the NCEL sea-water well.

When the GMR walls were 29 days old, the first set of monthly photographs was taken for the purpose of a statistical evaluation of time-dependent deterioration. On 7 July 1961, considerable efflorescence was noted on Walls 6, 9, and 11. Outside of this, no deterioration of the walls has been noted as of June 1962. The tests will continue.

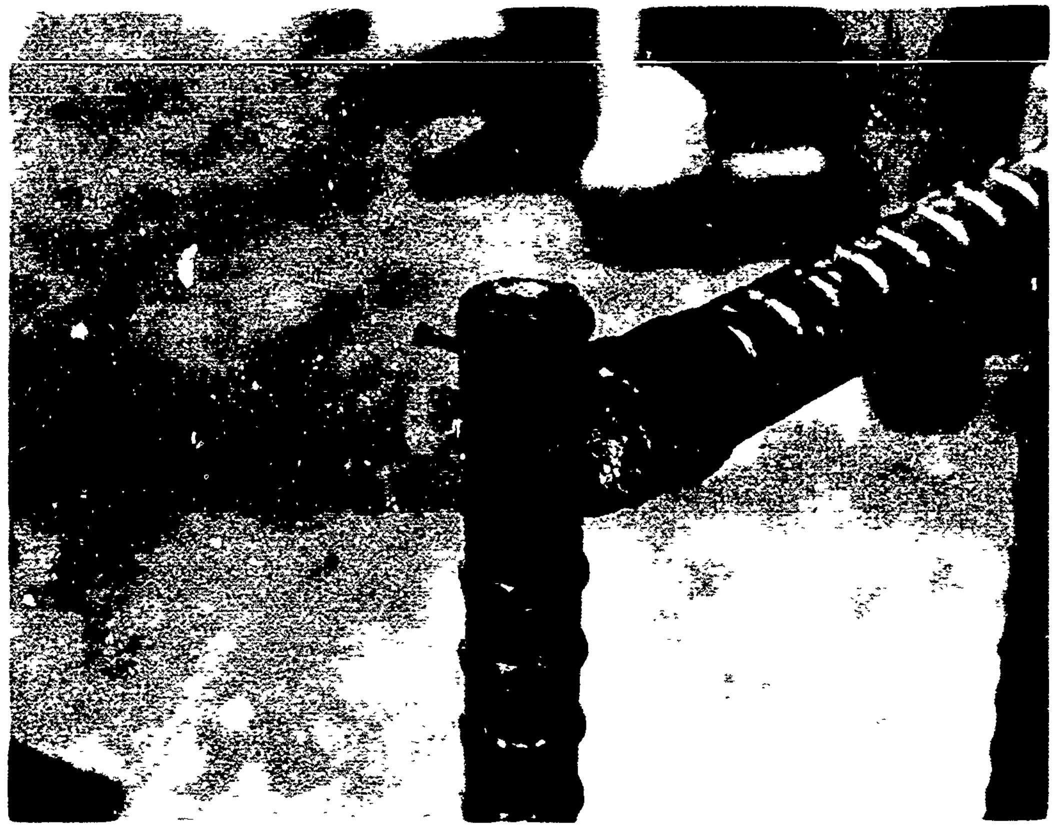


Figure B-1. Details of steel insulation.

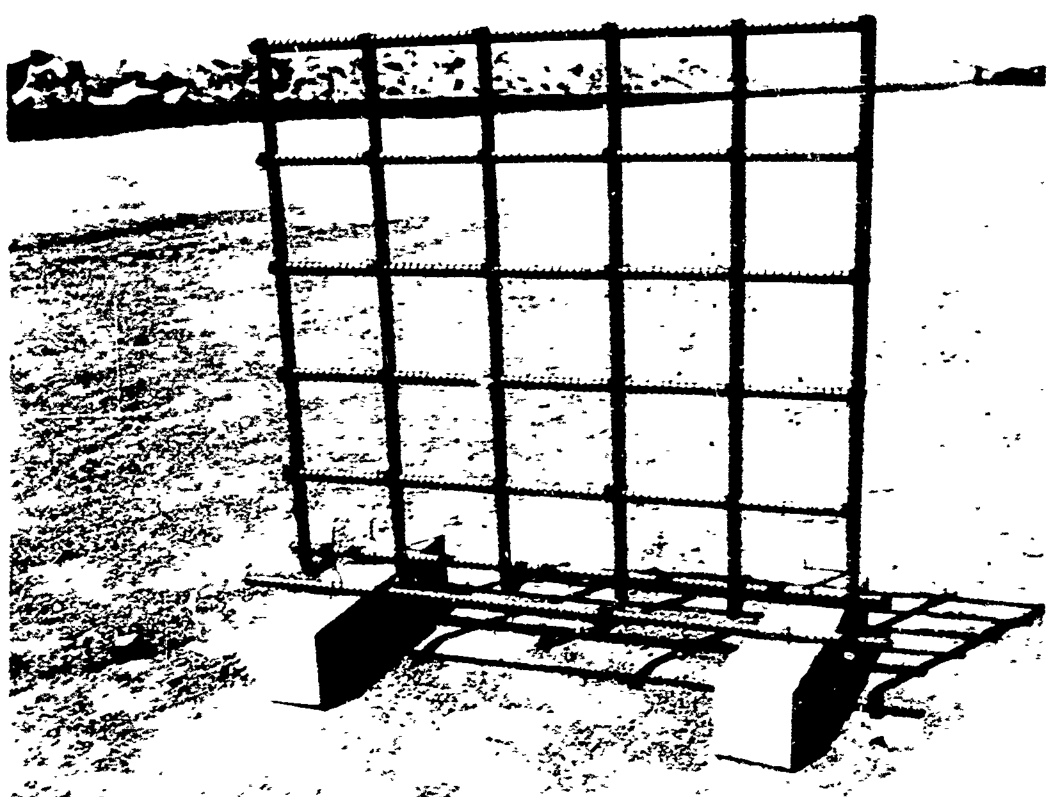


Figure B-2. Concrete supports for steel grids.

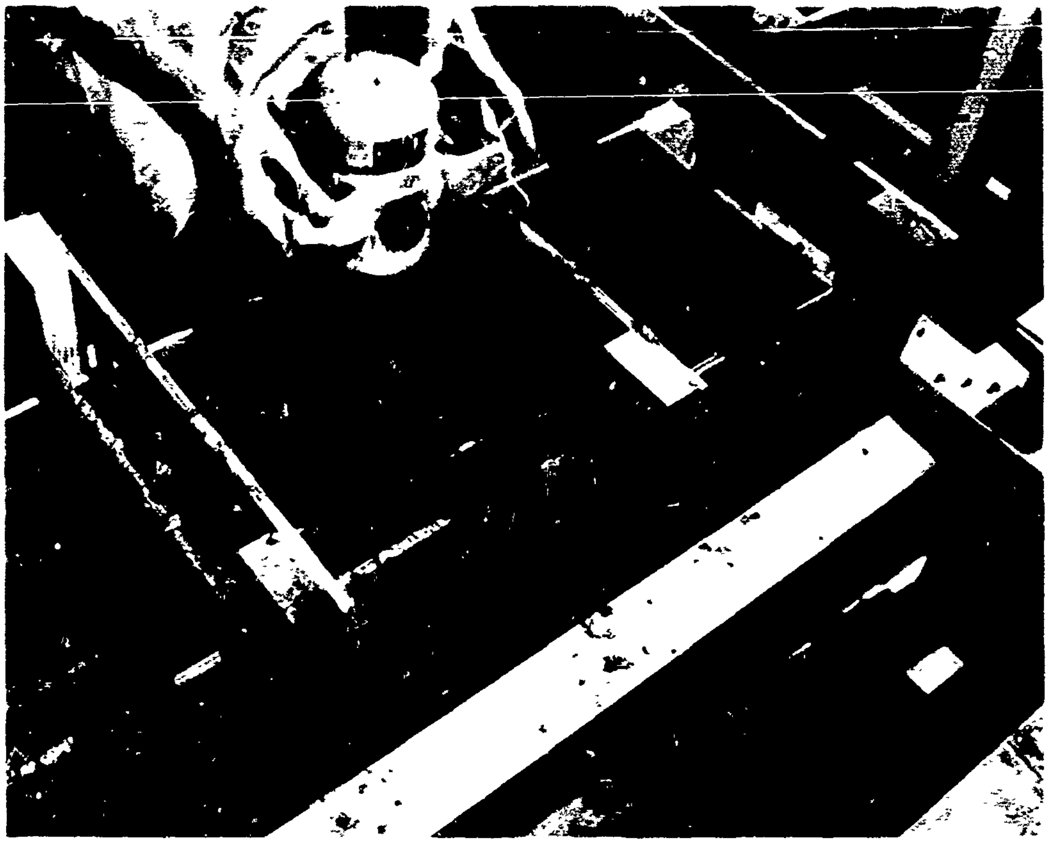


Figure B-3. Temporary devices for holding steel grids in forms.

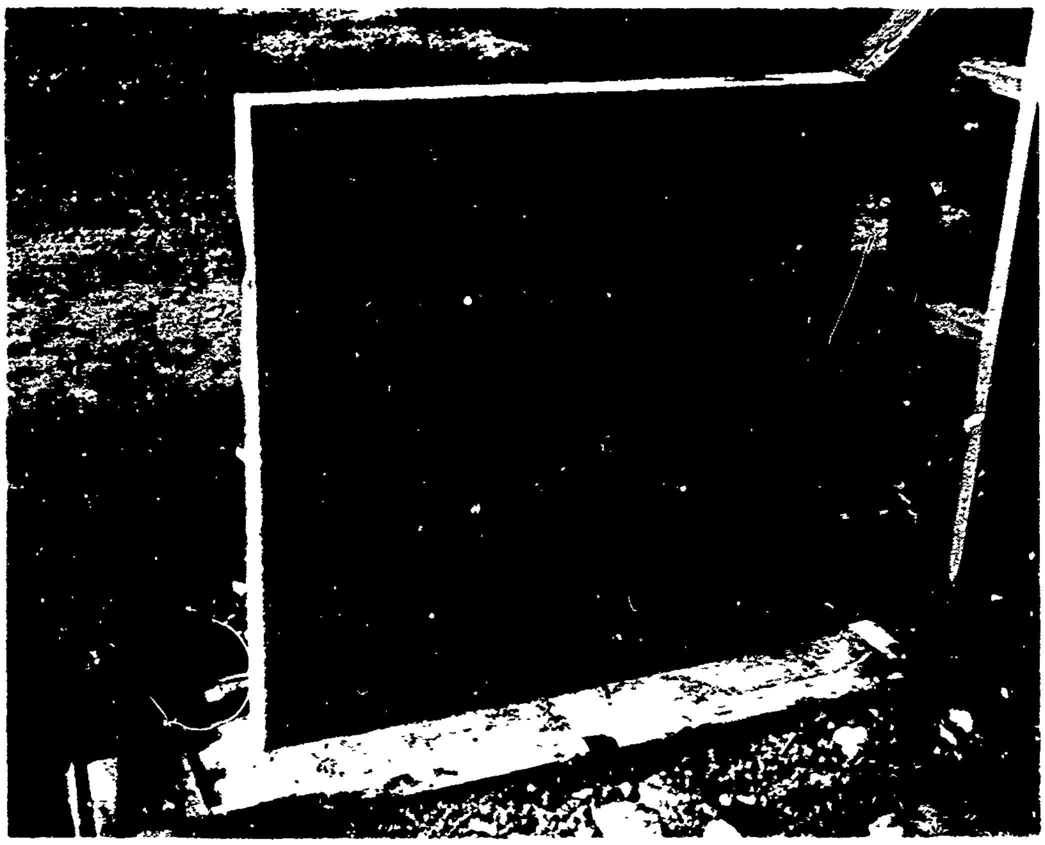


Figure B-4. San Gabriel wall after stripping forms.

Figure B-5. Coral concrete wall after stripping forms.

Figure B-6. Overall view of experimental walls.

Appendix C

CORROSION-DETECTION PROBE
(All the material in this appendix is reprinted from TR-217)

Corrosion-Detection Probe

The use of a Corrosometer, an electrical-resistance corrosion-rate detection probe and metering device, to measure directly the degree of corrosive environment of reinforcing steel occurred to the authors. The efficacy of the Corrosometer and probes as a corrosion-detection device had been investigated previously.[7] Consequently, probes fabricated of mild steel of the type shown in Figure C-1 were obtained for this purpose. The phenolic-resin potting cylinder shown in this figure was made just long enough to extend about 1/2 inch into the concrete. The probes were inserted into the fresh concrete through a hole in the wall of the cylindrical steel mold of the concrete cylinder. In order to do this, a cap was fastened across the top of the cylinder immediately after the concrete was cast. The cylinder was laid on its side, the plug unscrewed from the hole, and the probe inserted. The cylinder was then replaced in its as-cast position and placed in the fog room. The exterior end of the probe was protected by a small rubber sheet fastened around it.

The operation of the Corrosometer is based on the fact that the electrical conductivity of most metals is very great, while the conductivity of nonmetals is negligible by comparison. As the corrosion process converts metal into nonmetal, the electrical resistance of a piece of metal thus increases. The Corrosometer circuit utilizes this change of resistance to indicate the extent of penetration in an exposed metal specimen as corrosion proceeds on its surface.

The resistance of this exposed specimen is not measured directly by the Corrosometer circuit. Instead, a second specimen, made from the same metal or alloy, is connected in series with the first and the two specimens are made part of a bridge circuit. The second specimen is covered with a highly corrosion-resistant coating and so retains its original cross section and resistance. The ratio of the resistance of the exposed specimen to that of the covered specimen is then determined. Changes in this resistance ratio are translated directly into units of corrosion by the meter circuit.

By using a circuit which measures only the ratio of the two resistances, the measurements are made essentially independent of the current used to energize the bridge. Likewise, because temperature changes affect the resistance of both exposed

and covered elements in the same manner, the resistance ratio and the meter reading are independent of the temperature of the environment. It is this self-compensating feature of the bridge arrangement that permits the high degree of precision and sensitivity attained by the Corrosometer.

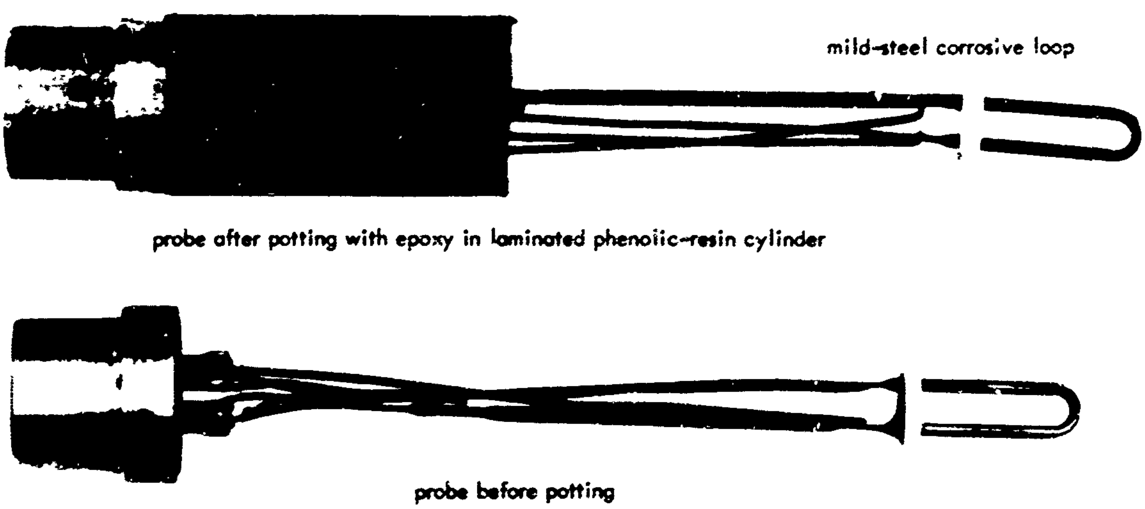


Figure 13. Electrical-resistance corrosion-rate detection probe.

Figure C-1. Electrical-resistance corrosion-rate detection probe.