WEATHERING STEEL GUARDRAIL
A MATERIALS PERFORMANCE EVALUATION

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Research Project 78 G-241
Research Report No. R-1155

Lansing, November 1980

ABSTRACT

Concern over the performance of weathering steels in general, and overlapping surfaces on these structures in particular, has prompted Michigan to evaluate some of its weathering steel guardrail. Principal areas of interest have centered on the deterioration of joint strength (performance) in relation to service age and environment. Data obtained from this investigation have been plotted and extrapolated to estimate the range of joint strength deterioration of guardrail in Michigan's range of corrosive environments.

Secondary items of interest have included an examination of:

1) Energy absorption of the guardrail joints in failure.

2) Several different methods of measuring thickness losses on pitted surfaces versus 'effective' thickness loss (calculated from loss in section strength).

3) Effects of pitting on section strength and energy absorption.

INTRODUCTION AND OVERVIEW

Beginning in the mid-1960's, Michigan began using unpainted 'weathering steels.' Initially, these steels were used only for bridges and later, in the 1970's, for guardrail, high-mast luminaires, and overhead sign structures. Weathering steels are, basically, a special composition steel product that, under specified weathering conditions, are promoted by the steel companies as developing a protective oxide coating that inhibits further corrosion. This protective 'patina' was hoped to make weathering steel structures maintenance-free by eliminating the need for periodic painting.

The appearance of Michigan's older weathering steel structures have not, in general, demonstrated the desired patina. In some cases, the thick rust scale present has prompted measurements and testing to verify the structural integrity. This was indeed the situation with some of Michigan's weathering steel guardrail (WSG).

Among the oldest weathering steel structures in Michigan is a test stretch of guardrail installed in February 1963. A cursory examination of this test site in 1978 revealed bulging lapped joints, expanding outward from the internal pressure of the corrosion products (Fig. 1). Ultrasonic thickness gage measurements indicated that some areas in the joint had lost up to 40 percent of their original thickness. Further testing and evaluation seemed definitely in order.

Studies performed by major steel companies with respect to the life of WSG have indicated typical thickness losses of from 'unmeasurable' in the freely exposed central portion of the beam up to 1.8 mil/year in the lapped joint area. The results of these studies are questionable in that they make thickness measurements using micrometers and/or ultrasonic thickness gages both of which can give deceptive measurements. Micrometers and ultrasonic thickness gages (UTG) will not only measure the thickness of the good metal but also any added oxide coating. While micrometers have the added disadvantage of giving no indication of the depth or extent of pitting, UTG's can even overestimate thickness on a pitted surface (ultrasonic wave transmission speed is slower in coupling agents used to seat the UTG probe than in the metal itself—thus, the deeper the pits, the greater the overestimate that can occur).

The possible problems in measurement technique combined with the physical appearance of our WSG pointed to the need for an independent assessment of the corrosion of these materials. Toward this end, thickness measurements combined with performance-related data seemed in order. Sections of rail were removed from several sites with the hope of

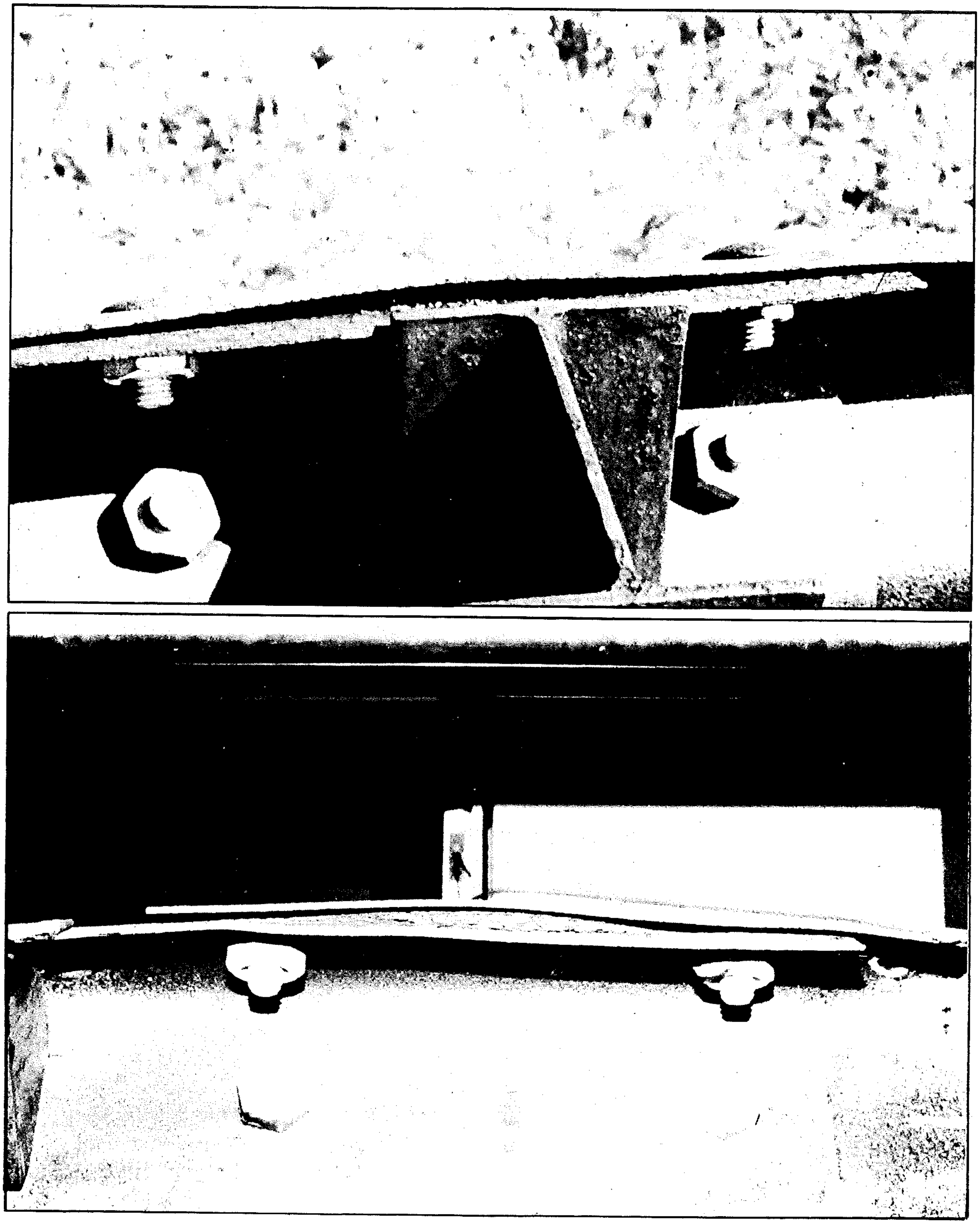


Figure 1. Weathering steel guardrail lapped joint after 15 years of exposure.

evaluating the performance of weathering steels as a function of age and environment and in relation to the performance of galvanized rail under similar conditions.

While Michigan is also extensively investigating the performance of its weathering steel bridges (preliminary results of the bridge study have been made available in a report recently published, and further work will be reported in the future) this report deals specifically with data obtained from guardrail. Even with this restriction in mind, there is much presented here that is relevant to weathering steel, in general, in a highway environment. Both the freely exposed rail 'interior' and the overlapped joint area are, for similar environments, representative of the performance of these same surfaces (freely exposed vs. overlapped) on other weathering steel structures.

PROCEDURE

Representative guardrail beams were obtained from a number of different sources and service environments. From these, an attempt was made to compare both WSG and galvanized guardrail (GG) for three different conditions—new as first installed, after exposure in a typical service environment, and after exposure in an extremely corrosive environment.

New guardrail of both the galvanized and weathering steel types, typical of that currently being installed in our highway systems, was obtained from the central warehouse which supplies our Maintenance Division.

Old WSG was obtained from several very different service environments. Guardrail sections were removed from a Lansing area test site in mid-1979 after 15-1/2 years of service (Fig. 2). Sections from this same site had also been removed and tested by A. J. Permoda in early 1970 after seven years of service. This site is situated on a freeway around the outskirts of Lansing. While situated close to the city limits, the Lansing site is still three to five miles from areas of industrial activity and other major sources of pollution. This section of freeway, with a moderately heavy traffic volume, is representative of an environment roughly intermediate between rural and urban. Guardrail at this site is freely exposed so that alternate wetting and drying should be easily obtained (except in lapped joints). While the guardrail is sufficiently close to traffic lanes that the beams were exposed to salt-laden vehicle spray, the boldly exposed nature of the site allows for periodic washing by rainwater thus minimizing the time interval and the concentration in which pollutants can act.

Weathering steel guardrail was also taken from a Detroit site after 4-1/4 years of exposure (Fig. 3). The guardrail taken from this heavily

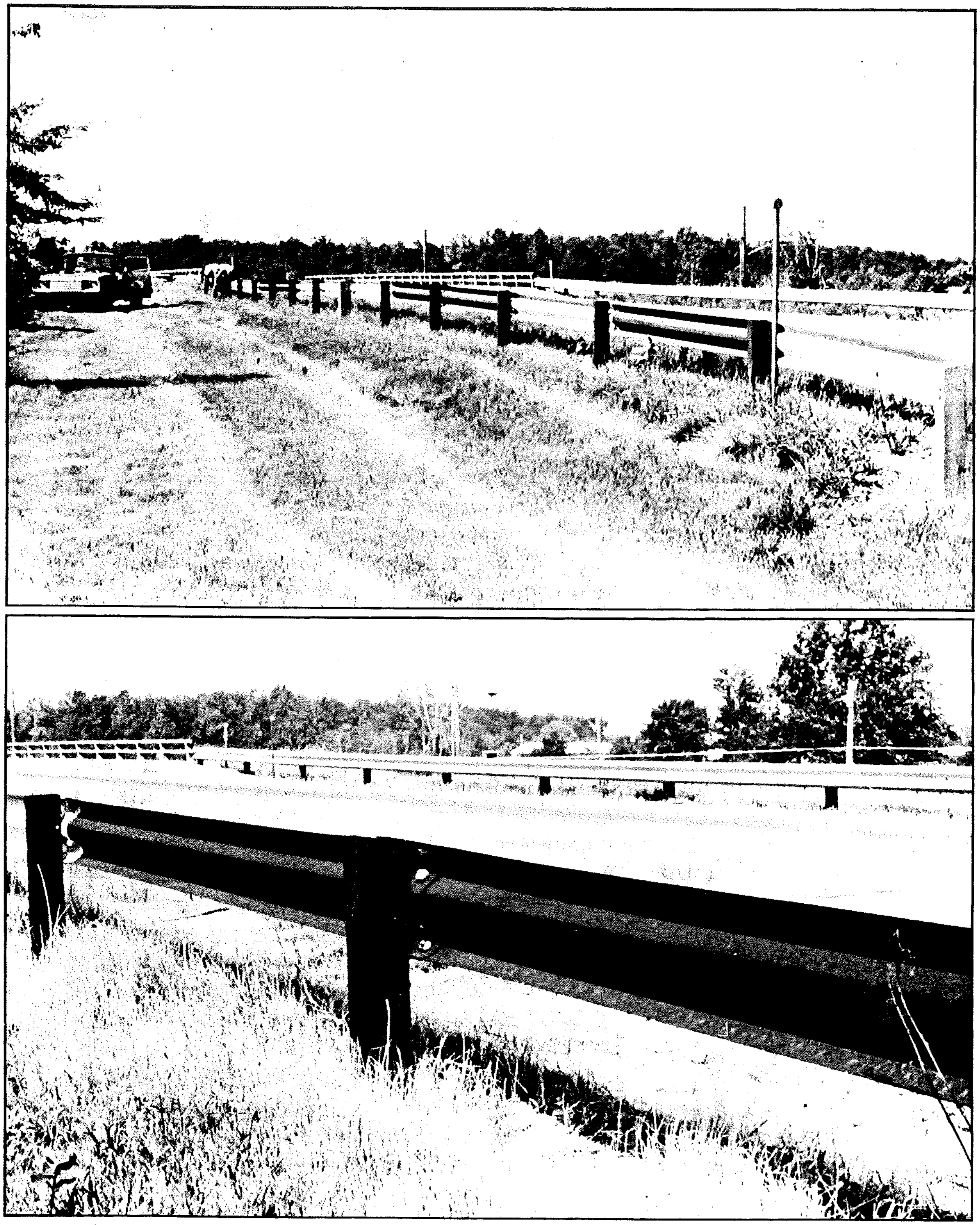


Figure 2. Lansing site—typical highway environment.

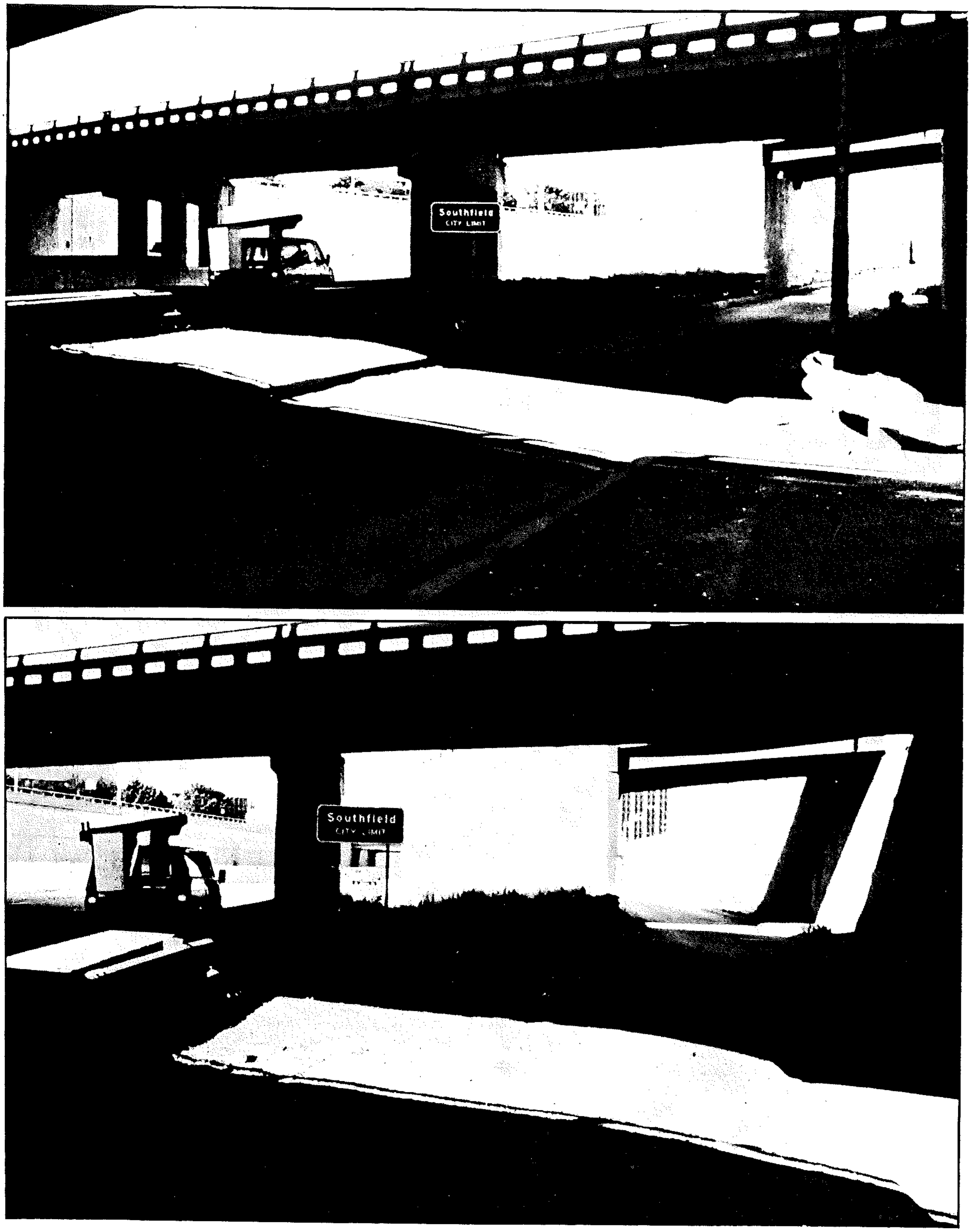


Figure 3. Detroit site—highly corrosive highway environment.

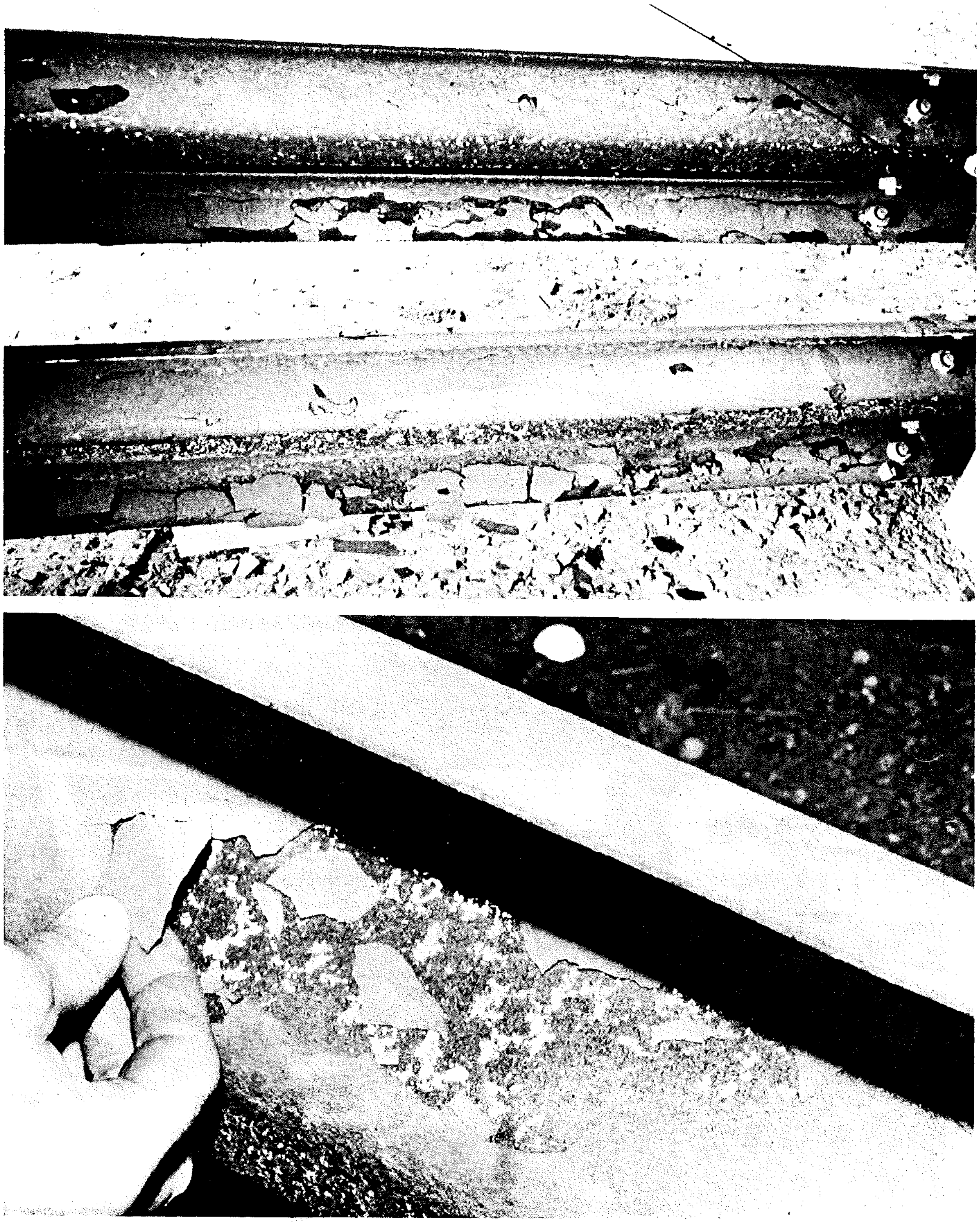


Figure 4. Close-up of weathering steel guardrail at Detroit site.

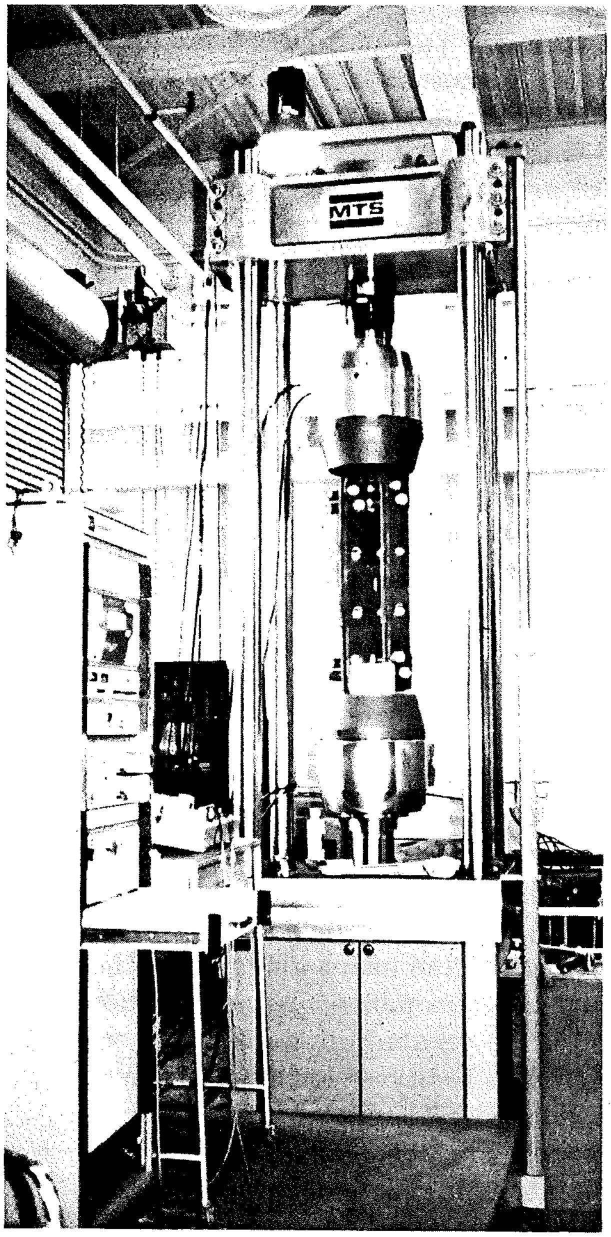
traveled urban site should be representative of what can be expected under extremely poor conditions. Of the two beams taken from this 'tunneled' freeway site, one was partially covered with trash and debris. Both were located under overhead bridges such that neither rain nor direct sunlight was allowed to aid in creating the alternate wetting and drying cycle deemed necessary to create a protective oxide coating. The WSG beams were also subject to all manner of roadway pollutants (most notably salt—the other major detrimental condition for weathering steels) distributed by passing vehicles. Without rainwater to wash the guardrail beams, pollutant deposits continue to build up (Fig. 4) and act in higher concentrations when moisture is present (dew, traffic spray, etc.).

Galvanized guardrail sections were also removed from the Lansing site after 15-1/2 years of service. The galvanized guardrail, being taken from the same location as the Lansing WSG, was exposed to essentially the same service environment. The galvanized coating was relatively well preserved with no red rust spotting.

An unsuccessful attempt was made to locate some more severely corroding GG in an effort to properly assess strength reductions possible in GG subjected to extremely corrosive environments. Although time and money did not permit further sampling, the major groupings of interest have been represented for WSG—Lansing, roughly a typical Michigan highway environment and Detroit, an extremely non-ideal environment—and reasonable estimates can be made for GG.

For the Lansing site, three of the WSG joints were removed intact by sectioning the guardrail beams several feet either side of the joint. This was deemed necessary to evaluate the effects, if any, of the prestressed condition of the joint (all WSG joints at this site were packed solid with corrosion products (Fig. 1) and most were obviously bulging from the internal pressure). The ends of the rails opposite the intact joints were disassembled in a conventional manner and later combined to create a few additional joints for testing. Of the two joints created in this fashion, one was constructed from end pieces that had both been lapped with galvanized guardrail for the last 9-1/2 years (galvanized rail replaced the WSG removed in 1970). The other joint involved end pieces from WSG to WSG lapped joints. The GG and Detroit WSG beams were removed whole and joints were later formed from the two ends of a single rail.

Joints were also constructed from the interior sections of the guardrail for all of the groups of guardrail. This was done with the old weathered guardrail to gain insight into how differently the two types of exposure of a guardrail beam (freely exposed vs. lapped joint) respond to the same service environment. This same procedure was also performed with the new



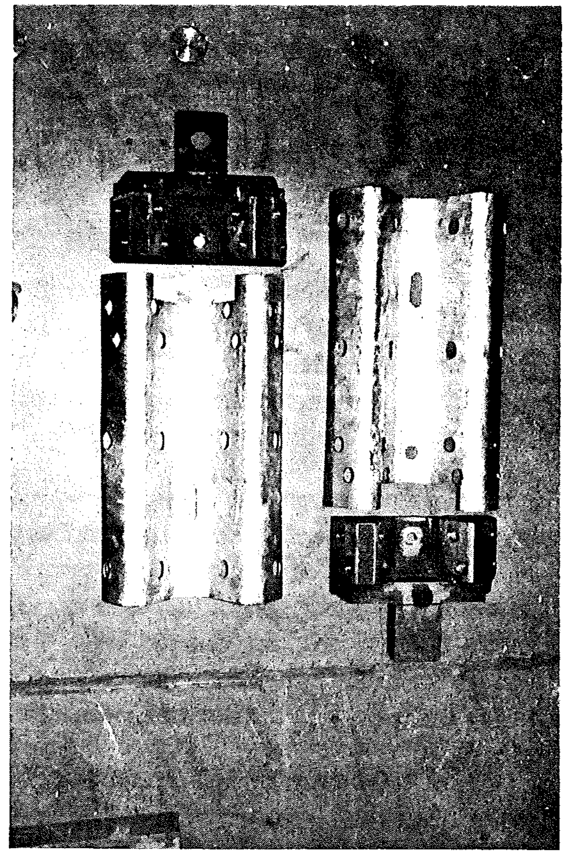


Figure 5. The 200 kip MTS set-up for pulling guardrail joints (left) and fixtures (above) for gripping guardrail with prepared guardrail ends.

beams to verify that there was not a significant difference in test strength between the original guardrail ends and our constructed guardrail ends.

All full-size guardrail joints were statically pulled apart (in tension) until failure (Fig. 5). Major results for the full-size guardrail are given in Table 1.

Failure criteria were developed for several possible modes of failure (see Appendix). Based on this 'simplified' analysis, failure in shear (tearing of the beam metal around the joint bolts) would be expected to occur most frequently. With a known failure load, the Appendix calculations can be reversed to solve for beam thickness giving an empirically based number that is representative of the 'effective' joint thickness at the specific location(s) of failure (i.e., the metal actually sheared). The derived failure equations also allow a means of relating beam thickness to load sustained at failure (i.e., joint strength). Thus a starting thickness, known beam properties, and known corrosion rate for a particular environment would provide sufficient information to plot, say, joint strength vs. age (Fig. 6).

For the exposed WSG, no data were available regarding the initial beam thickness; however, estimates could be made. The thickest micrometer measurements (peak-to-peak measurement of the pitted surface) represent a figure that is less than or equal to the original starting size. The uncertainty in this initial data and values calculated therefrom is reflected in the data tables by an appropriate inequality sign.

Small sheet tensile specimens were cut from all of the various groups of guardrail. Both the rail interior and lapped joint areas are represented for the WSG. While the small specimen tensile data, in most instances, do not directly relate to the full-size guardrail joint performance, they do supply valuable information on the performance of weathering steel in general. The small tensile specimens were used to give a performance-related evaluation of the corrosion losses that would include not only the effect of micrometer or UTG measured loss, but also additional losses due to pitting as well as any stress concentration effects possibly resulting from the pitting. Several of the small tensile specimens were milled to remove all pits so that the actual tensile properties of the guardrail material could be assessed. This could then be used to determine the 'effective' cross-sectional area and hence 'effective thickness' of the remaining pitted tensile specimens. From this information, an 'effective corrosion rate' could, in turn, be calculated. Data from the small tensile specimens are presented in Table 2.

Thickness measurements were taken for the small tensile specimens of all the various groups of guardrail via both micrometer and ultrasonic

TABLE 1
FULL SIZE JOINT STRENGTH AND THICKNESS

Guardrail Group	Age, years	No. of Joints Tested	Average Joint Strength, 1,000 lb	Average Beam Thickness, Micrometer, in.	"Effective" Joint Thickness, in.	Average Thickness Loss, Micrometer Based, mil/year	Effective Thickness Loss, Based on Strength, Pitting Effects Included, mil/year
New Galvanized Guardrail							
"Interior" Constructed Joint	new	2	107.25	0.109			
Lapped Joint	new	6	113.50	0.109	0.109		
New Weathering Steel Guardrail							
"Interior" Constructed Joint	new	2	110.00	0.106			
Lapped Joint	new	5	98.75	0.106	0.106		
Lansing Site Galvanized Steel Guardrail							
"Interior" Constructed Joint	15.50	2	119.00	0.111			
Lapped Joint	15.50	2	120.00	0.111	0.114		
Detroit Site Weathering Steel Guardrail (Initial Beam Thickness ≥ 0.114; Initial Joint Strength ≥ 104,000 lb)							
"Interior" Constructed Joint	4.25	2	96.75	0.106		≥ 1.88	≥ 3.06
Lapped Joint	4.25	2	92.25	0.101	0.101	≥ 3.06	≥ 4.48
Lansing Site Weathering Steel Guardrail (Initial Beam Thickness ≥ 0.112; Initial Joint Strength ≥ 113,000 lb)							
"Interior" Constructed Joint*	7.00	1	≥ 101.00	0.109		≥ 0.51	
Lapped Joint*	7.00	1	≥ 104.00	0.105	≥ 0.103	≥ 1.09	
"Interior" Constructed Joint	15.50	2	114.00	0.104		≥ 0.52	≥ 0.97
Original Lapped Joint	15.50	4	90.75	0.091	≥ 0.090	≥ 1.35	≥ 1.86

* Previously reported, March 1970.

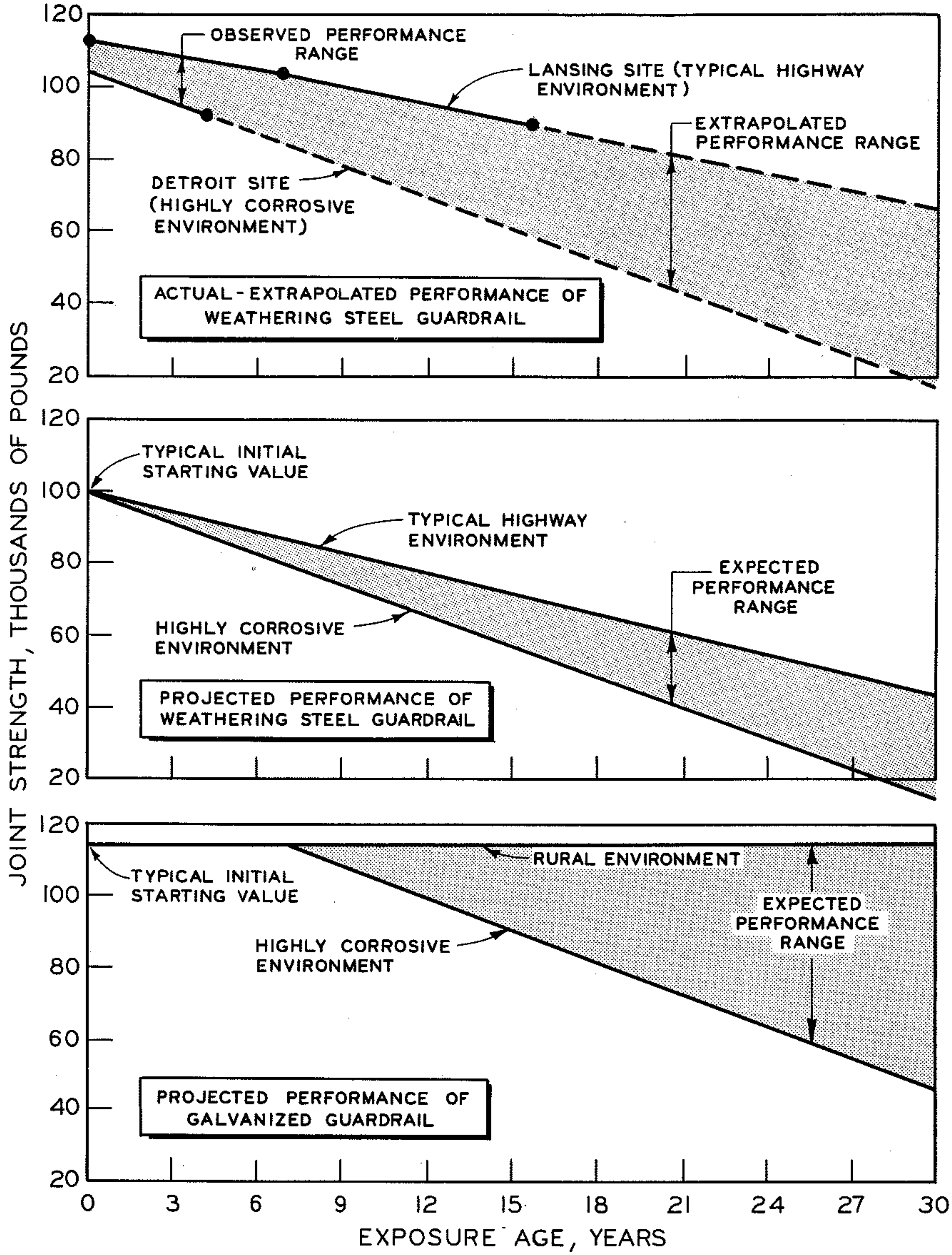


Figure 6. Guardrail joint strength actual and projected vs. exposure age.

TABLE 2
STRENGTH AND THICKNESS DATA FROM SMALL SHEET TENSILE SPECIMENS

Guardrail Group	Exposure Age, years	Yield, ksi	Tensile, ksi	Width, in.	Load Sustained at Failure, thousands of pounds	Initial Thickness, in.	Effective Thickness, in.	Percent Strength Reduction From New
New Galvanized (average)	0	62.8	95.8	0.497	5.17	0.108	0.108	--
New Weathering Steel (average)	0	57.5	76.7	0.501	4.05	0.105	0.105	--
Lansing Site -- Galvanized Guardrail -- "Typical" Michigan Environment								
(average)	15.50	66.9	87.7	0.498	4.83	0.110	0.110	--
Detroit Site -- Weathering Steel Guardrail -- Highly Corrosive Environment								
Milled (average)	4.25	55.500	74.500	0.501	3.17	0.085	0.085	--
Beam Interior								
No. 1	4.25	*	*	0.502	3.67	0.114	0.098	13.8
No. 2	4.25	*	*	0.502	3.87	0.114	0.104	9.1
Interior Avg.							0.101	11.5
Lap Joint								
No. 1	4.25	*	*	0.498	3.67	0.114	0.099	13.2
No. 2	4.25	*	*	0.495	3.38	0.114	0.092	19.5
Lap Avg.							0.095	16.4
Lansing Site -- Weathering Steel Guardrail -- "Typical" Michigan Environment								
Milled (average)	15.50	64.250	82.500	0.500	3.55	0.086	0.086	--
Beam Interior								
No. 1	15.50	*	*	0.503	4.15	0.112	0.100	10.8
No. 2	15.50	*	*	0.502	4.10	0.112	0.099	11.7
No. 3	15.50	*	*	0.502	4.07	0.112	0.098	12.2
No. 4	15.50	*	*	0.502	4.12	0.112	0.100	11.2
Interior Avg.							0.099	11.5
Lap Joint								
No. 1	15.50	*	*	0.503	3.47	0.112	0.084	25.3
No. 2	15.50	*	*	0.503	3.30	0.112	0.080	29.0
No. 3	15.50	*	*	0.500	3.75	0.112	0.091	18.8
Lap Avg.							0.085	24.4

* Beam material properties are "directly" measurable only for the milled specimen.

thickness gage. These data, in turn, could be compared with the effective thickness to assess the shortcomings of each method. Some special surface preparation was used for each method. For the micrometer, loose rust was removed until the peaks of the pitted surface exposed bare metal. At this point, at least no portion of the oxide coating is included in the thickness measurement. For the ultrasonic thickness gage the surface was lightly ground, leveling off the tops of the peaks and leaving the valleys filled with iron oxide. Since the ultrasonic wave transmission speed of iron oxide closely approximates that of iron, this type of surface preparation (as opposed to, say, sandblasting) minimizes the couplant associated over-readings that can occur on a pitted surface. The comparison of these thickness measurements with the effective thicknesses are reported in Table 3.

TABLE 3
THICKNESS MEASUREMENTS OF PITTED GUARDRAIL SURFACE COMPARED WITH EFFECTIVE THICKNESS

Guardrail Group	Exposure Age, years	Effective Thickness, in.	Micrometer		Ultrasonic Thickness Gauge	
			Measured Thickness, in.	Deviation From Effective Thickness, percent	Measured Thickness, in.	Deviation From Effective Thickness, percent
Detroit Site Weathering Steel Guardrail						
Beam Interior (average)	4.25	0.101	0.106	-5.0	0.098	2.0
Lap Joint (average)	4.25	0.095	0.101	-6.3	0.094	0.2
Lansing Site Weathering Steel Guardrail						
Beam Interior (average)	15.50	0.099	0.104	-4.8	0.100	-0.8
Lap Joint (average)	15.50	0.084	0.091	-7.4	0.087	-3.1

Load vs. displacement plots were recorded for both the full size joints and the small tensile specimens during failure. From these, total energy absorption to failure was calculated where possible. Results are reported in Table 4.

Chemical analyses were performed for the major groupings of guardrail. Results are tabulated in Figure 7 and discussed further in the results section.

TABLE 4

ENERGY ABSORPTION OF FULL SIZE GUARDRAIL JOINTS AND GUARDRAIL BEAM MATERIAL IN STATIC TENSILE FAILURE

Guardrail Group	Exposure Age, years	Energy Absorption		Elongation, percent	Projected New Energy Absorption For Sheet Tensile Specimens, ft-lb	Percent Reduction In Energy Absorption Of Pitted Tensile Specimens From Projected New Values
		Full Size Joint, ft-lb	Small Sheet Tensile Specimen, ft-lb			
New Galvanized (average)	0	12,600	171.0	22.6	---	--
New Weathering Steel (average)	0	13,000	167.0	27.0	---	--
Lansing Site - Galvanized Guardrail						
(average)	15.50	10,500	159.0	23.0	---	--
Detroit Site - Weathering Steel Guardrail						
Milled	4.25	--	146.0	29.1	196	--
Beam Interior	4.25	--	161.0	26.9	196	17.9
Lap Joint	4.25	10,900	113.0	22.3	196	42.3
Lansing Site - Weathering Steel Guardrail						
Milled	15.50	--	150.0	28.0	198	--
Beam Interior	15.50	--	161.0	25.2	198	18.7
Lap Joint	15.50	10,600	85.5	16.6	198	56.8

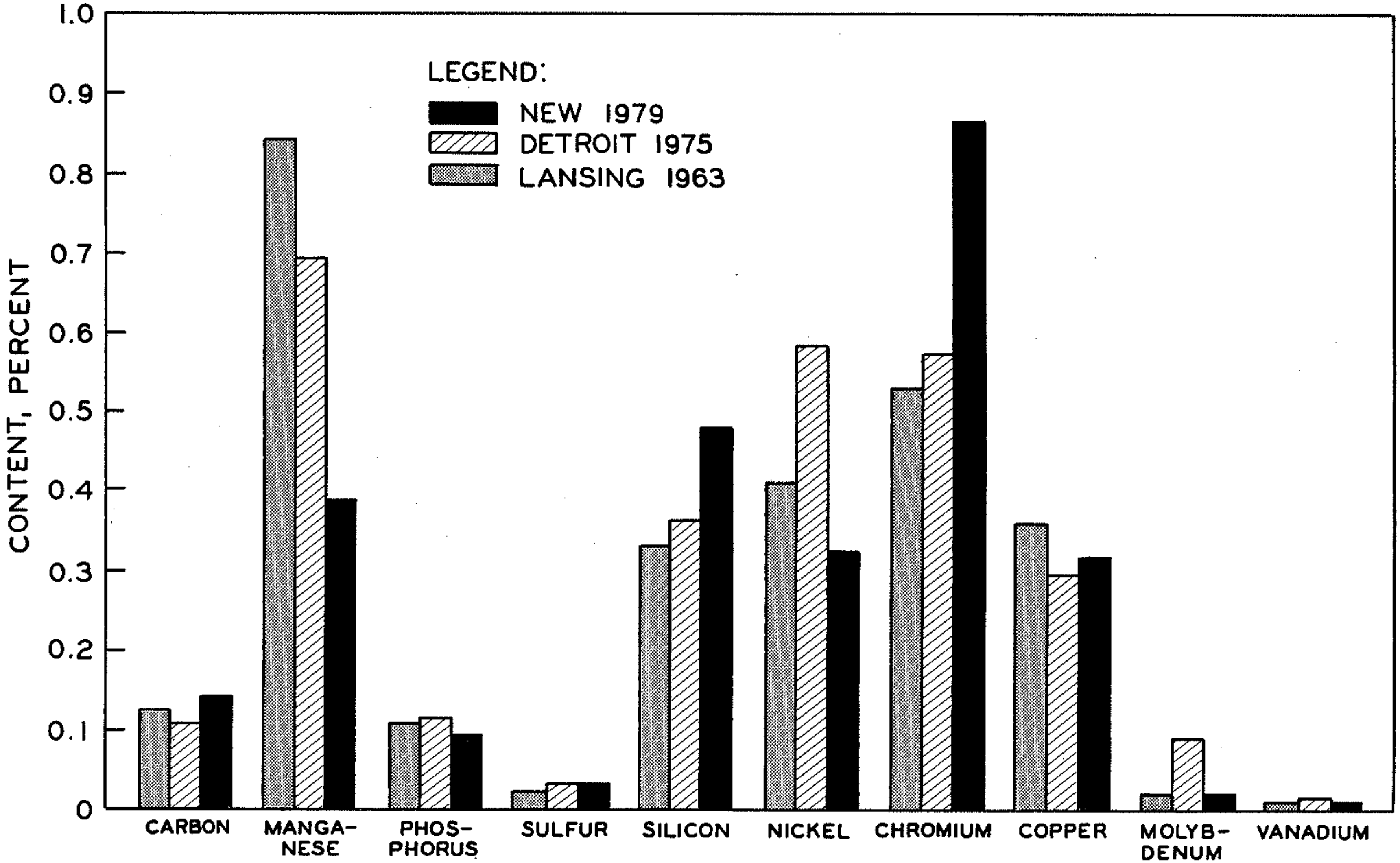


Figure 7. Chemical analysis of weathering steel guardrail (WSG).

RESULTS

Although limited time, funds, and initial data have restricted the thoroughness and completeness of this investigation, a concerted effort was made to extract as much useful information as possible from what was available.

While the research results of the steel companies have dealt principally with thickness measurements, our primary concern has been with respect to the actual strength deterioration of the weathering steel guardrail. With this in mind, special attention was given to relating accurate thickness measurements to the actual performance of the guardrail. Additional concerns have centered on evaluating the extent to which the pitting has influenced the guardrail performance as well as differences between the performance of the central (interior) portion of the beam and the lapped joint area.

Typical thickness measurements are reported in Tables 1 and 3 for all of the representative groups of guardrail. Several important trends can be noted from these data. The substantial differences in corrosion rate between the freely exposed surfaces of the guardrail and the lapped surfaces exemplifies the effect of surface exposure on the resulting corrosion rate. While noticeably present at both sites, these surface exposure differences were greatest for the Lansing site. For this site, the average effective corrosion rate between the lapped surfaces (1.40 mil/year/surface) was found to be almost three times that for the freely exposed surface with some large areas (several square inches) exhibiting six to seven times the corrosion rate of the freely exposed surface (this represents a single surface corrosion rate of slightly over 3 mil/year). This high corrosion rate for the lapped surfaces places some doubt on the ability of weathering steels to outperform steels lacking "enhanced corrosion resistance" (at least for lapped surface exposures).

Several factors probably play a part in the increased corrosion rate observed in the lapped joints. Moisture, when present, is more likely to be retained in the joint area creating conditions similar to submerging the weathering steel in water—a condition not recommended by the steel companies, since this can promote a leaching away from the surface of the specific elements required for the formation of the protective oxide coating.

Lapped surfaces, when sufficiently close together, are also susceptible to concentration cell corrosion—an enhanced corrosion process created by differences in the concentration of either oxygen or metal ions (or both) in the water in the crevice and the area external to it. For the oxygen ion concentration cell, corrosion is accelerated between the lapped surfaces.

The metal ion concentration cell, more likely to occur when pollutants such as salt are present to aid in the formation of metal ions, on the other hand, does its damage just exterior to the area of contact. The concentration cell reaction is often referred to as crevice corrosion.

While the guardrail splice can start out as a fairly loose area of contact, corrosion products building up within the joint will eventually create the tight contact necessary for crevice corrosion. Corrosion products would be expected to deposit outward from areas of first contact until eventually the whole joint forms a tight crevice. The bolt holes would aid in the drying of the areas immediately surrounding the bolts, thus possibly helping to preserve these areas longer.

The significant difference between the micrometer-based thickness measurements and the empirically derived effective thickness helps to point out the inadequacies of a micrometer for measuring a pitted surface. This inadequacy is most noticeable for the Detroit site WSG. For this highly corrosive environment, micrometer measurements indicated average corrosion rates of from 0.94 mil/year/surface for the freely exposed surface and 1.53 mil/year/surface for the lapped surface with some areas (less than 1 sq in.) hitting 3.41 mil/year/surface. The average effective thicknesses of these exposures, however, reveal actual corrosion rates of 1.53 and 2.95 mil/year/surface, respectively.

Also very noticeable is the difference in corrosion rate between a 'typical' (Lansing site) environment and a highly corrosive (Detroit site) environment. These differences can be used to establish estimates of the limits of joint strength performance for the possible range of corrosion rates that can be expected to occur in our multitude of service environments (Fig. 6).

As the full-size guardrail joints were pulled apart, in tension, several different modes of failure (and combinations thereof) were observed. Failure most commonly occurred as the metal surrounding the lap joint bolts sheared and bent until one end of the bolts could freely pass through the deformed holes of the lapped beams Some of the joints failed in tension as one of the beams pulled apart, usually along a line through the bolt holes. Although both types of failure occurred for both types of guardrail, the shear and bending mode was most representative of the WSG. Both modes occurred with approximately equal frequency in the galvanized guardrail with bolt shearing occasionally complicating the picture.

Reductions in strength of the full size joint after approximately 16 years were negligible for the galvanized guardrail. Although no initial data were available for the GG, this represents a fairly safe assumption since the galvanized coating remained intact. The weathering steel guardrail, on the other hand, showed a 20 percent reduction in strength for the same time interval at the Lansing site. The more corrosive environment of the Detroit site resulted in an 11 percent reduction in strength after only 4-1/4 years.

The one joint (Lansing site) constructed from WSG that had been in contact with GG for the last nine years was 11 percent stronger than the WSG to WSG contact joints from the same environment. Or stated in a slightly different manner, the joint strength of this galvanically protected WSG joint was only 3 percent below the averages for the WSG from the same site tested nine years earlier.

No significant difference in strength (or energy absorption) could be discerned between those joints removed whole and those later reassembled in the laboratory. Apparently, any differences in the performance of the WSG joints due to the rust-bulging prestress were negligible (at least with respect to the tests performed).

Differences between the joint strength of the original guardrail joints and interior beam constructed joints were found to be significant and are reported in Table 1. Care must be taken, especially for WSG, in the interpretation of these results since the differences in the new WSG indicates that our procedure for constructing the interior joint was sufficiently different from the original joint construction to account for a fair portion of the difference. The primary difference between our constructed joints and the original joints appears to be in the method of producing the bolt holes. Our constructed joints were drilled as opposed to the punching procedure that is normally used in mass production. Although time did not permit a thorough investigation of this phenomenon, it is suggested that the joint strength differences may be due to possible differences in the condition of the metal surrounding the bolt holes (even slight bending in the metal surrounding the punched out area could contribute to the ease with which the shear and bending mode of failure proceeds) as well as possible alignment differences in the bolt hole pattern influencing the engagement sequence of the bolt and surrounding metal interface.

A significant further drop in the strength of the weathering steel joints could very well occur. The manner in which the guardrail fits together and in which corrosion products are initially deposited appears (at least in our sampling) to act to initiate crevice corrosion at first far removed from the bolt hole areas so crucial to the joint strength (shear and bending mode of failure). But as corrosion products continue to deposit outward from areas of first contact, it is only a matter of time before the bolt hole areas are affected also. If the present areas of greatest corrosion were shifted to the areas surrounding the bolt holes, the projected strength reduction would be on the order of 40 to 50 percent after only 16 years for the Lansing guardrail. This same type of shift would produce a 20 to 26 percent strength reduction in the Detroit guardrail after only 4-1/4 years. While it is true that the preferential drying around the open exposed bolt holes would very likely help to protect this area in most instances, there are undoubtedly other situations in which these same circumstances could work to a disadvantage. This might occur when, say, vehicle-borne salt spray makes direct contact with the face of the guardrail, putting corrosive pollutants right through the bolt hole area into the joint crevice. While the freely exposed face of the guardrail would dry faster and wash more easily via rainwater, the areas within the crevice subjected to vehicle spray would probably continue to build up salt concentration and soak up moisture via capillary 'wicking' action and corrode at a much faster rate.

Results of the thickness measurements are reported in Table 3 These results are, of course, largely dependent on the methods of surface preparation and the data presented are valid only for the specific procedures used. (Refer to page 13 for a review of the procedures followed.) Thickness measurements made by both micrometers and ultrasonic thickness gages are compared to an effective thickness (derived from beam material property considerations). The micrometer consistently overestimated the effective thickness by 5 to 8 percent. The ultrasonic thickness gage both over and underestimated the effective thickness but never by more than approximately 3 percent and usually considerably less.

Measurements of the energy absorbed by both full-size guardrail joints and small tensile specimens are recorded in Table 4. Results of the full size beams provide minimal insight since no direct comparisons were available between the weathered guardrail and the values it would have had when new. Results of the small tensile specimens were more instructive. For these a new comparison was effected by milling both surfaces until all pits had been removed and applying a corrective factor (the original thickness divided by the milled thickness), to bring the result back to the original gage thickness. The small tensile specimens (top and bottom surfaces unmachined) taken from the interior (freely exposed section) of the beams indicated only an 18 percent reduction in energy absorption while the specimens taken from the lapped area averaged a 57 percent reduction in energy absorption for the Lansing site after 15-1/2 years of weathering. Results for the Detroit site show similar reductions after only 4-1/4 years of exposure. Most of this reduction in energy absorption was the result not so much of the beam material to sustain a load but rather a significant decrease in the apparent ductility of the beam material. Percent elongation was almost halved for the tensile specimens taken from the lapped joint area. High stress concentrations produced by significant changes in cross-sectional area (large dishpan-like pits occur on the lapped surfaces) are undoubtedly involved here. While stress concentration effects produced by the same pit depths would be less for thicker materials, the possible problems are still apparent.

Chemical analysis was performed for all of the major groups of guardrail. The results of these analyses are shown in Figure 7. The primary item of interest here is the difference in composition between the three groups (new; Detroit site, 4-1/4 years; and Lansing site, 15-1/2 years) of WSG. While it is very likely that the composition plays some role in influencing the corrosion rate of the freely exposed surface, there is reason to doubt that this same influence would be as significant in the lapped joint areas.

Since the beneficial metallic elements can be leached away in long term aqueous environments, the moisture retention abilities of the joint area would minimize, if not eliminate, the beneficial effects of the corrosion resistant elements. In other words, the performance of the section of the guardrail beam most likely to fail is possibly independent, or nearly so, of the weathering characteristics of the component steel (at least for the inner contact surfaces) within the limits of variation in chemistry that are present in these steels.

The major results of this investigation are summarized in Figure 6. In these graphs, joint strength deterioration vs. age is plotted for both a typical highway environment and a highly corrosive environment. In this manner, a representative performance range is established by upper and lower boundaries. The uppermost graph of Figure 6 depicts and extrapolates results actually observed for WSG. In the middle graph the observed deterioration rates for WSG are projected from a current typical initial joint strength.

Figure 6 establishes the same type of performance boundaries for typical GG as currently used in Michigan (AASHTO M 180-74, Class A, Type 2, 3.6 oz zinc coating). The upper limit is established by extrapolating a rural environment case, where the galvanized coating should remain intact for the full 30 years. The lower limit is developed for a severe industrial environment where the galvanized coating should be starting to spot with red rust after approximately seven years, (based on the American Society for Metals, predicted coating corrosion rate for a severe industrial environment). From this point on, the deterioration rate of the joint strength is assumed to approximate that of the WSG from the Detroit site—an extremely corrosive environment. This would appear to be a reasonable assumption. Studies have been performed by one of the major steel companies on weathering steel and plain carbon steel corrosion specimen suspended under the same bridges that sheltered the Detroit site guardrail (1, 2). For this particular highly corrosive environment, weathering steels do not apparently demonstrate a great advantage over plain carbon steels. The ratio between the average corrosion rates for weathering vs. plain carbon steel after seven years was found to be 0.92. Combining this knowledge with the known protective effects that will result from the remaining zinc coating, this lower boundary probably presents a slightly worse picture than what would actually occur for galvanized rail. Even for this conservative estimate, the worst case for the GG just begins to approach the upper limit (typical highway environment) of the WSG performance band after 27 to 30 years (Fig. 6).

Guardrail sections of weathering steel remain at both the Detroit and Lansing sites so that additional work could be done in the future to better refine the joint strength vs. age plots or obtain additional information as required.

CONCLUSIONS AND RECOMMENDATIONS

Since Michigan has already ceased to use unpainted weathering steel in new construction, the desirability of this point is not reiterated here. It should be mentioned, however, that the data presented in this report have done much to influence this decision, especially for guardrail. Some of the major considerations involved in the guardrail decision include:

1) Current lack of a significant cost differential between galvanized guardrail and weathering steel guardrail,

2) No loss in strength of the GG as long as the galvanizing remains intact, and

3) The projected drop in WSG joint strength below the minimum level specified guardrail strength during the expected service life and the possible liability this places on the state in traffic accidents involving guardrail.

REFERENCES

1. Zoccola, J. C., et al, "Performance of Mayari R Weathering Steel (ASTM A 242) in Bridges at the Eight Mile Road and the John Lodge Expressway in Detroit, Michigan," Bethlehem Steel Corporation.

2. Zoccola, J. C., Interoffice Memo dated April 5, 1976, Eight Year Corrosion Test Report, Eight Mile Road Interchange, Bethlehem Steel Corporation.

APPENDIX

GUARDRAIL BEAM FAILURE CRITERIA

Beam Data

Thinnest Width (TW) = beam width - combined width of bolt holes

= 19 in. - 4 x 0.875 in. = 15.5 in.

Beam Thickness (BT) for 12 gauge = 0.105 ± 0.009

= (upper thickness limit) 0.114

= (lower thickness limit) 0.096

Smallest Beam Cross-Section (SBCS) = TW x BT

= (upper thickness limit) 1.767 sq in.

= (lower thickness limit) 1.488 sq in.

Tensile Strength of Beam Material (TS) $\geqq$ 70 ksi

Yield Strength of Beam Material (YS) $\geqq$ 50 ksi

Tensile Strength Failure

Failure in this instance usually involves a pulling apart of a beam rail along a line running through the bolt holes.

Tensile Load at Failure = SBCS x TS

$\geqq$ 123,700 lb at upper thickness limit

$\geqq$ 104,200 lb at lower thickness limit

Shear and Bending Failure

Failure in this case is by a rotational motion of the bolt, tearing through the two beams. An empirical expression for this failure mode is shown below.

Beam Tensile Load at Failure = (area of metal sheared by bolts) x (shear strength of beam material)

where: (area of metal sheared by bolts) = BT x 2 (no. of rails) x 8 (no. of bolts) x 2 (beam-metal is sheared on both sides) x (length of shear in failure = 0.763 for WSG and 0.683 for GG)

The length of the shear in failure is empirically derived from the results of the new beam guardrail tests. Slight differences in the exact failure mechanism involved for the WSG and the GG have necessitated separate shear lengths for these two types of guardrail.

and: (shear strength of beam material) $\cong$ 1/2 x (tensile strength of beam material)

For WSG

$\doteq$ 97,400 lb at upper thickness limit

$\doteq$ 82,000 lb at lower thickness limit

= 12.2 x BT (in.) x TS (ksi)

or BT (in.) = Joint Strength/12.2 x TS (ksi)

Similarly for GG

$\doteq$ 87,000 lb at upper thickness limit

$\doteq$ 68,500 lb at lower thickness limit

= 10.9 x BT (in.) x TS (ksi)

or BT (in.) = Joint Strength/10.9 x TS (ksi)