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Foreword

This publication contains three papers and three abridgments of papers which should be of interest to materials engineers, bridge engineers in both design and construction, and structural maintenance engineers.

Coburn reviews the uses made of high-strength low-alloy steel at various locations. The paper compares the composition and mechanical properties to those of structural carbon steel and reviews the advantages of a steel requiring no painting. The steel may be of interest to landscape architects as well as structural designers because it develops a dark-colored oxide film, permitting it to blend into the natural background of trees and shrubs.

Main reviews the case for galvanized bridges, beginning with the use of galvanized guardrail and bridge rail and culminating in the construction near Quebec of the first galvanized conventional steel bridge in the world.

The paper by Schwerdtfeger discusses corrosion data based on 4,500 specimens of ferrous materials buried by the National Bureau of Standards at 86 sites under various conditions of soil resistivity and pH for periods up to 17 years. It then relates laboratory and field data to the determination of current densities necessary for cathodic protection.

A summary of welding practices employed by state highway departments in constructing steel bridges is an interesting feature of the Illinois cooperative research report and is included in the paper by Sanders and Munse. An even more important inclusion than the survey of the uses of welding is the record and explanations of the inspection and testing methods that have been developed and used in the control of high-quality welding.

Smylie discusses the fabrication of orthotropic deck sections for the Port Mann Bridge in British Columbia. This is probably the largest major structure in North America to employ this technique of floor construction and the Dominion Bridge Company of Vancouver, B.C., Canada, displayed great ingenuity in adapting their shop procedures to the requirements of the imported isotropic design. Though in use in many parts of Europe for many years, this type of bridge is new on this continent because high wage scales as compared to European labor costs had inhibited its use. This paper illustrates how the development of production-line methods and equipment designed to achieve efficiency and economy in this field has closed the gap in construction costs. By such developments, orthotropic construction has been made competitive with other conventional types in the field of long-span bridge design and construction.

The Benjamin and Walsh paper discusses a survey of 31 bridges in Illinois to determine if there were any significant differences between the condition of surfaces or quality of concrete in bridge floors poured on conventional wood forms and floors poured on "stay in place" metal forms. Twenty-one of the decks had utilized the metal forms and ten of the structures had used wood forms. The authors had the cooperation of the Illinois Division of Highways in the selection of the test structures and the accumulation of the test data. The authors evaluated the data and found no significant differences in any of the characteristics of concrete poured into either type of form. After 5 years of service, the metal forms were bonded to the bottom of the deck slab.

Contents

A Low-Cost Maintenance-Free Structural Steel for Highway Applications

S. K. COBURN
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Highway engineers and landscape architects have indicated a need for a structural material that is low in initial cost and requires a minimum expenditure of money and labor for maintenance. Such a product is required for use in roadside rest area buildings, highway guardrails, median barriers, signposts and structures, lighting standards, snow fences, bridges, and utility poles and towers. There is also a desire in some instances to reduce the amount of light-reflective materials employed in highway structures and thus minimize distraction from the natural scenery. The use of unpainted high-strength low-alloy steel containing certain alloying elements to improve atmospheric corrosion resistance appears to fulfill this need. The superior mechanical properties and enhanced resistance to atmospheric corrosion of one of these steels in various environments are described. Examples are given illustrating the current use of this steel in such applications as transmission towers, large buildings, light standards, and bridges.

•THE GROWING body of literature describing the numerous studies conducted by municipal, state, and Federal highway agencies concerning the various facets of highway construction, operation, and maintenance attests to the engineering complexity of this 20th century phenomenon—the highway.

Over the years a wide variety of materials has been employed by highway and bridge engineers and by landscape architects to implement their design concepts. Such terms as strength, durability, resistance to impact, resistance to atmospheric corrosion, cost of maintenance, and esthetic appeal are constantly being used to describe the performance of the materials utilized in highway construction practice. Often materials that have performed effectively in another industry may be transferred to a highway application with excellent chances for success. But the best test for evaluating a construction material is service performance over extended periods of time.

When a corrosion engineer surveys the highway research literature, he encounters comments and observations such as the following:

1. The increasing use of highways to serve as rights-of-way for communication and power line poles and towers is a fact that must be accepted as an economic approach to lower operating costs and rates, in spite of the fact that the presence of such installations is not always esthetically pleasing (1, 2);
2. The most objectionable esthetic feature of many new urban freeways is the excessive amount of shiny metal—guardrails, fences, railings, lampposts, sign standards, etc. (3);
3. Roadside rest areas should contain buildings and structures that require little or no maintenance and employ brick, tile, or stone so as to be of a permanent nature (4); and

Paper sponsored by Committee on Metals in Highway Structures.

TABLE 1

COMPOSITION RANGES AND TYPICAL COMPOSITION FOR HIGH-STRENGTH LOW-ALLOY STEEL[a]

Element	Composition (%)	
	Range	Typical
C	0.12 max.	0.09
Mn	0.20-0.50	0.38
P	0.07-0.15	0.09
S	0.05 max	0.033
Si	0.25-0.75	0.48
Cu	0.25-0.55	0.41
Cr	0.30-1.25	0.84
Ni	0.65 max.	0.28

[a]COR-TEN, U.S. Steel Corp.; alloy of Cr, Si, Cu, Ni, and P.

4. A typical urban area in Iowa contains 52 signs and 43 signposts per mile, and two-man paint crews waste from 35 to 50 percent of their time waiting for the posts to be repaired before they can paint them (5).

It is evident from such comments that highway engineers are actively seeking alternative materials of construction that are low in initial cost, require little or no maintenance, involve a minimum expenditure for labor, and are esthetically pleasing for the specific application. A final requirement is that a structural material, in addition to fulfilling economic and engineering requirements, should alter the natural scenery as little as possible.

These are an imposing set of requirements for any single structural material; however, a specific type of steel produced in a variety of shapes and forms is available to meet these needs. This product is known as a high-strength low-alloy steel and is characterized in mechanical properties by a minimum yield point of 50,000 psi in accordance with Specification A 242 of the American Society for Testing and Materials (ASTM). (ASTM A 242 describes the properties for structural shapes and bars, for welded, riveted, or bolted construction; ASTM A 374 and A 375, respectively, deal with cold-rolled and hot-rolled steel sheets and strips in cut lengths and coils.) The particular high-strength low-alloy steel discussed herein (COR-TEN, U.S. Steel Corp.) meets the requirements of the ASTM A 242 specification in thicknesses up to and including ½ in., provides five to eight times the resistance to atmospheric corrosion that can be obtained with structural carbon steel with low residual copper content, and can be used in the unpainted condition for many boldly exposed applications. The applications discussed hereafter specifically cover the use of unpainted high-strength low-alloy steel. Because ASTM A 242 does not specify the alloy elements required to obtain enhanced resistance to atmospheric corrosion, a structural design engineer should inform his supplier of the intended usage and indicate the need for a steel with a chemical composition that includes the elements Cr, Si, Cu, Ni, and P in the composition range shown in Table 1. It should be recognized that although this paper is devoted to a discussion of a specific steel composition, active research programs are being pursued towards the development of other steels with enhanced atmospheric corrosion resistance, equivalent mechanical properties and weldability. Thus, it is expected that in the near future additional steel compositions will be available for use in the highway applications discussed herein.

CHARACTERISTICS OF HIGH-STRENGTH LOW-ALLOY STEEL

It is evident from the composition of the high-strength low-alloy steel given in Table 1 that the elements Cr, Si, Cu, Ni, and P are present in larger than just trace quantities. These elements exert a significant influence on the corrosion resistance and mechanical properties of this steel. The mechanical and engineering properties of this steel are described in Table 2. For comparison, Table 3 contains a listing of the properties of ASTM A 36 structural carbon steel.

Atmospheric Corrosion Resistance

Before discussing the unique properties of this high-strength low-alloy steel, it would be well to consider briefly why and how steel rusts in the atmosphere. Iron

TABLE 2

MECHANICAL PROPERTIES (MINIMUM) OF HIGH-STRENGTH LOW-ALLOY STEEL

Steel Thickness (in.)	Yield Point (psi)	Tensile Strength (psi)	Elongation (%)	
			In 8 In.[a]	In 2 In.
≤½	50,000	70,000	19	22
>½-1½[b]	47,000	67,000	19	–
>1½-3	43,000	63,000	16	24

[a]This steel can be joined by riveting or bolting in all thickness and can be welded in thicknesses up to and including ½ in.
[b]0.180 in. and heavier.

TABLE 3

COMPARATIVE PROPERTIES AND ENGINEERING DATA

Mechanical Properties (at ≤ ½ in. thick)	High-Strength Low-Alloy Steel	Structural Carbon Steel (ASTM A 36)
Yield point, min, psi	50,000	36,000
Tensile strength, psi	70,000 min	58/80,000
Elongation in 2 in., min, percent	22	23
Elongation in 8 in., min, percent, 0.180 in. and heavier	18	20
Cold bend	180° D = 1T	180° D = 1/2T
Resistance to atmospheric corrosion (comparative)	4 to 6	1 (or 2 with copper 0.20% min)
Compressive yield point, psi	Tensile Y. P.	Tensile Y. P.
Shearing strength, psi	0.70 T. S.	0.70 T. S.
Modulus of elasticity, psi	28/30,000,000	28/30,000,000
Charpy impact, keyhole notch (as rolled, room temperature, average), ft-lb	40	25
Coefficient of expansion per degree F, -50 to 150 F	0.0000065	0.0000065

and other metals oxidize because they have a considerable affinity for oxygen. This strong potential for uniting with oxygen is related to the large amounts of energy used to extract the iron from its ore. To prevent oxidation, which occurs only in the presence of moisture on the surface, a barrier must be inserted between the steel and the atmosphere. With stainless steel a thin, transparent, almost impervious oxide film forms naturally and is extremely effective in preventing corrosion. Oxide films also form on carbon steel; however, because they tend to be voluminous and porous, they are usually incapable of sealing the surface effectively. Therefore, carbon steel tends to oxidize continuously.

It has been established that the corrosion process that occurs on steel in contact with the necessary film of moisture is an electrochemical one similar to the action in a battery. At any given instant, some areas are corroding and are called anodic sites; areas that are not corroding serve as cathodic sites. Small electrical currents flow between the anodic and cathodic sites. Initiation of these so-called corrosion currents is stimulated by the presence of small amounts of dissolved salts, alkalis, or acids that are almost always present in the atmosphere and which deposit on the metal surface.

The pioneering work of Buck, (7) and the early publications and interpretations of the results of ASTM tests conducted from 1916 through 1951 (8) have demonstrated that the introduction of small amounts of copper into carbon steel increases its atmospheric corrosion resistance appreciably (9). The ASTM tests show that a minimum of 0.20 percent copper added to carbon steel doubles its atmospheric corrosion resistance. The results of numerous atmospheric corrosion tests conducted during the past 30 yr confirm these observations and also show that the high-strength low-alloy steel under discussion is at least four and often as much as eight times as resistant to corrosion as carbon steel.

Manner of Conducting Exposure Tests

The standard ASTM testing method for obtaining corrosion data is illustrated in Figure 1. Test specimens of various compositions are exposed on a rack inclined 30 deg to the horizontal and facing south. The specimens are cleaned and weighed before exposure, and individual specimens are removed at scheduled intervals for evaluation and then discarded. The reduction in thickness is calculated from the loss in weight. A time-corrosion curve is established from data developed during several exposure periods. The slope of this curve in the linear portion indicates the corrosion rate.

Results of Exposure Tests

Time-corrosion curves describing the performance in an industrial atmosphere of the high-strength low-alloy steel in comparison with a copper-bearing steel and a carbon steel are shown in Figure 2. For these curves, the calculated reduction in thickness in mils of test specimens is plotted against the time of exposure in years. It is evident that the high-strength low-alloy steel is far more resistant to corrosion in an industrial atmosphere than are either the copper-bearing steel or the carbon steel. The narrowness of the bands shows that the thickness of the respective test specimens did not influence the corrosion rates and indicates that corrosion is a surface phenomenon. Similar results illustrating the superiority of the high-strength low-alloy steel over carbon steel in a semi-rural atmosphere are shown in Figure 3.

Figure 1. Typical arrangement of test specimens on exposure racks, 800-ft lot, Kure Beach, N.C.

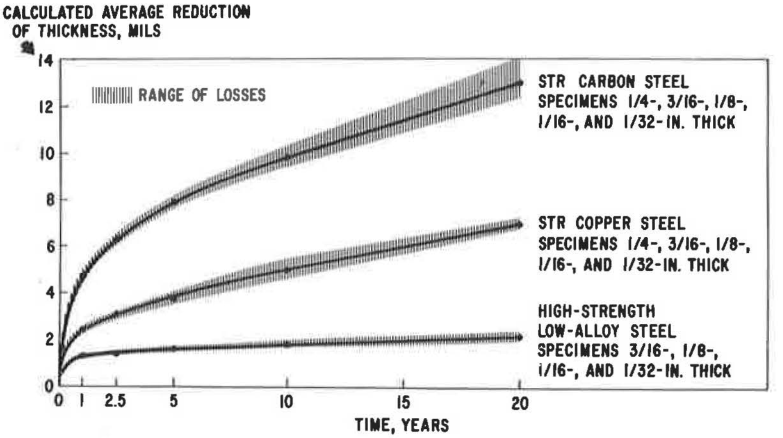


Figure 2. Comparative corrosion of steels (1-ft square specimens) in industrial atmosphere, Kearny, N.J.

Figures 2 and 3 also show that approximately 80 percent of the corrosion occurring on the high-strength low-alloy steel in both the industrial and the semi-rural atmospheres occurs within the first 2 to 3 yr of exposure. This amounts to a reduction of about 1. 5 mils in metal thickness. The corrosion that occurs in the next 20 yr reduces the thickness by only about another 0. 5 mil. Thus, it is clear that the structural integrity of the high-strength low-alloy steel is not endangered by corrosion when it is properly used in the boldly exposed condition.

Figure 4 illustrates the comparative corrosion losses for specimens of carbon steel and the high-strength low-alloy steel at 80 and 800 ft from the Atlantic Ocean at Kure Beach, N. C. Although the performance of the high-strength low-alloy steel at the 80-ft lot is superior to that of carbon steel, the corrosion rate is so high that its use in the unpainted condition would be inadvisable in severely corrosive marine atmospheres. Specimens located 800 ft from the ocean corrode at a considerably lower rate, and the superior performance of the high-strength low-alloy steel over carbon steel is again evident. Applications close to seawater may warrant consideration of galvanizing or painting the high-strength low-alloy steel.

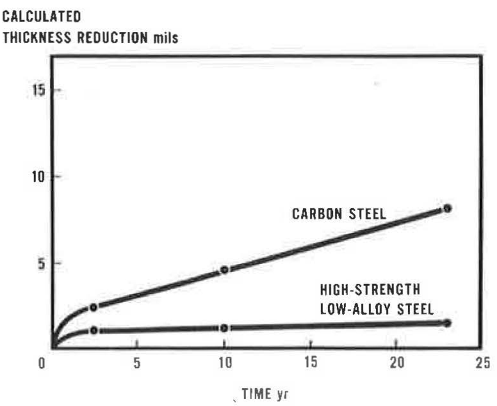


Figure 3. Comparative corrosion of steels in semirural atmosphere.

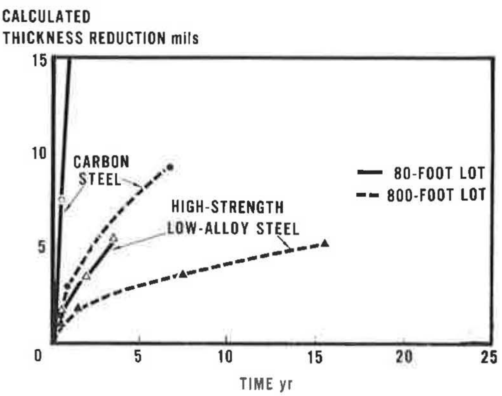


Figure 4. Comparative corrosion of steels in marine atmosphere, Kure Beach, N.C.

SURFACE PROTECTION

Formation of Protective Oxide Film

The satisfactory atmospheric corrosion resistance of the unpainted high-strength low-alloy steel is dependent on the chemical composition containing the unique combination of alloying elements necessary for the development of the thin, tightly adherent, protective oxide film. The interaction between these elements and various environmental constituents has not been completely established; however, several factors are apparent. The kind and amount of contamination in the atmosphere are significant. Other critical factors are sunlight, rain, and dew. As mentioned previously, moisture must be present for corrosion to occur. The drying effect of sunlight assists in the formation of the protective oxide film from the corrosion product. Rain is helpful in washing away water-soluble iron compounds that form during the corrosion process. Dew is considered to be especially important because it remains on the steel surface for a substantial period during the night and absorbs atmospheric pollutants that can assist in the formation of the oxide film.

The extent of corrosion of steel in a particular locality can be influenced by the type of fuel being burned and the kind of combustion products released to the atmosphere. Although many industrial effluents are responsible for corrosive attack, these same contaminants, particularly the sulfur-containing compounds, assist in the formation of the protective oxide film on the surface of the high-strength low-alloy steel.

Characteristics of Rust Film

In the as-rolled condition, the appearance of the high-strength low-alloy steel surface is very similar to that of carbon steel. The color and texture of the rust film that forms on the high-strength low-alloy steel, however, differs markedly from that which forms on carbon steel. Figure 5 shows the appearance of the rust film on steels exposed in an industrial atmosphere for 15 yr. The rust film that forms on carbon steel is voluminous and reddish brown in color. That which forms on the high-strength low-alloy steel is very thin, no more than several mils thick. The color of this thin rust film changes progressively, depending on the time of exposure and the kind and amount of atmospheric contamination present, and ranges between a dark reddish brown and a warm purple gray.

Furthermore, the rust film on the high-strength low-alloy steel, unlike that on carbon steel, is not scaly and loose but fine grained and tightly adherent. Whereas large patches of rust fall from a bare carbon steel structure as it weathers, the initial rust film on the high-strength low-alloy steel slowly dusts away through the erosive action of wind and rain and after a time the formation of loose rust virtually ceases. In rural and semi-rural atmospheres, where the amount of air pollution is relatively

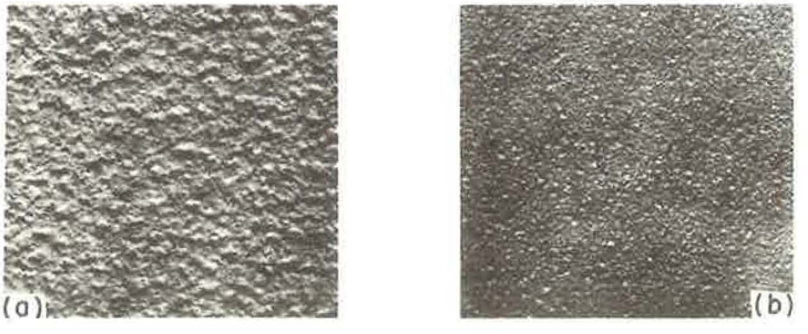


Figure 5. Appearance of rust after 15 yr in industrial atmosphere: (a) carbon steel, and (b) high-strength low-alloy steel.

low, the change in color of the rust film will be extended over a relatively longer period of time. At any one time, the color of the rust film will be essentially uniform and the change in color will be almost imperceptible.

Performance of High-Strength Low-Alloy Steel in Different Locations

Another series of exposure tests has been conducted in widely separated parts of the country to determine the effect of surface condition of the test specimens at the time of exposure. Welded specimens of the high-strength low-alloy steel and of carbon steel were exposed in each of six metropolitan areas and one seacoast location. These sites were selected on the basis of conditions peculiar to their location. San Francisco's nightly fog, Birmingham's lack of prevailing wind, and Newark's chemical industry are typical of the variety of local conditions sought for this investigation. For each location, one pair of specimens were sandblasted before exposure, a second pair were acid-pickled, and a third pair had the mill scale intact. The carbon steel specimens were welded with carbon steel electrodes; the high-strength low-alloy steel specimens were welded with a 2 percent nickel electrode.

The results after 2 yr of exposure indicate that with all three initial surface conditions, the high-strength low-alloy steel performs well in each of the test areas: in the Newark-New York area; the Gary and Chicago areas; in Birmingham, Washington, San Francisco, and Seattle; and at the marine site, 800 ft from the ocean, at Kure Beach, N. C. In each location, a dark tightly adherent oxide film formed. Furthermore, regardless of the surface condition, the characteristic homogeneous-appearing rust film formed during the first 2 yr of exposure. Figure 6 shows the respective specimens exposed on a test rack in Chicago. The darker appearing specimens on the right are of high-strength low-alloy steel. Although in all cases the oxide film formed on the specimens from which mill scale was not removed, mill scale should be removed by pickling or blasting to obtain optimum appearance during the initial exposure period.

Effect of Sulfur in Atmosphere

As indicated earlier, the protective rust film forms most readily on the high-strength low-alloy steel when it is exposed in localities where industrial and home-heating exhaust gases from the combustion of coal and oil contribute sulfur compounds to the atmosphere. However, sulfur compounds are also present in the rural areas of the country as a consequence of the decomposition of field crops, the burning of diesel fuel in farm equipment, and the general movement of weather. Because of the widespread interest in sulfur as a crop nutrient, soil and rainwater have been analyzed for sulfur in about 20 states. Some of the more recent work has been summarized in a United States Department of Agriculture technical bulletin (10). Table 4 gives a few of the results reported. It is evident from these data that sufficient sulfur is widely available in the atmosphere to promote the formation on the steel of a tightly adherent, protective oxide film that gradually assumes a reddish to gray-brown appearance.

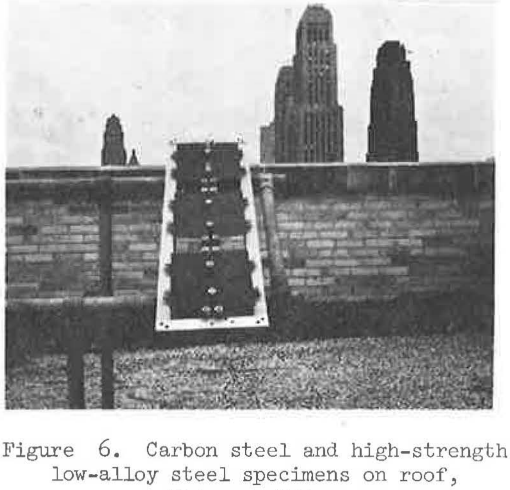


Figure 6. Carbon steel and high-strength low-alloy steel specimens on roof, Chicago, Ill.

Staining

During the formation of the protective oxide film, a small amount of water-soluble corrosion products will form. These will be washed down the members of a structure by rain or other sources of moisture and will cause staining on concrete

TABLE 4

SULFUR COLLECTED IN RAINWATER, DECEMBER 1952 TO NOVEMBER 1955

State	No. Locations	Avg. Ann. Accretion (lb/acre)
Ala.	2	6
Fla.	6	2-7
Ga.	5	4-8
Ky.	6	8-14
Miss.	7	3-7
N. C.	16	4-14
Tex.	5	5-11
Va.	19	12-31

footings, galvanized hardware, and unglazed porous brick.

If the rust-bearing water flows over concrete surfaces, the alkali in the concrete causes precipitation of brightly colored iron salts which can adhere and permanently discolor the footing. If this discoloration is objectionable, it can be minimized by designing the footing so that water will not stagnate but will drain away. Another approach is to apply a dark-colored paint with a glossy finish that will prevent stains from being easily seen. The concrete may also be coated with a silicone-type preparation that will prevent the iron-bearing water from reacting with the alkali in the concrete. If the rust-bearing water should contact galvanized hardware, the alkaline surface will cause iron salts to precipitate on the surface of the zinc in the same manner. These superficial deposits, however, will not in any way impair the effectiveness of the protective zinc coating.

Safety

Safety must be an important consideration when new equipment and new materials of construction are proposed. Because of the granular texture of the high-strength low-alloy steel surface, linemen and other workers are afforded a more positive footing on both wet and dry surfaces.

Because of the absence of any coating, metal-to-metal contact can be obtained in bolted connections; thus, the possibility that the connections will loosen under vibration is minimized.

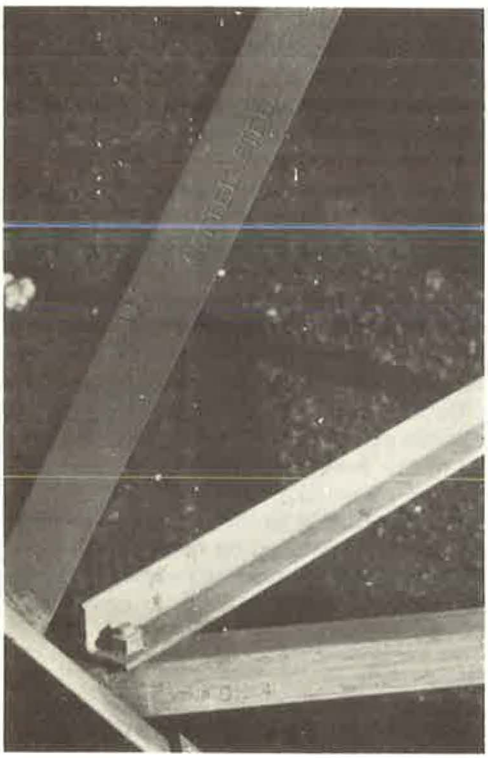


Figure 7. High-strength low-alloy steel angle on galvanized steel transmission tower after 15 yr of exposure, Gary Steel Works.

SOME APPLICATIONS FOR UNPAINTED HIGH-STRENGTH LOW-ALLOY STEELS

Transmission Towers

Although the high-strength low-alloy steel has been available since 1932, utility engineers have recognized its potentialities only in recent years. In 1948, an unpainted angle member (Fig. 7) was installed in a galvanized carbon-steel tower at the Gary Steel Works of the U. S. Steel Corp. Periodic thickness measurements have been made and to date no significant changes have occurred. It might be expected that severe corrosion would first occur in the bolted connection, so the angle was removed and examined. There was almost no indication of attack in the area of contact between the unpainted and the galvanized members. The zinc surface was still bright and shiny, and the bare steel had only a superficial coating of rust.

Since that time, several other installations of this steel, in the unpainted condi-

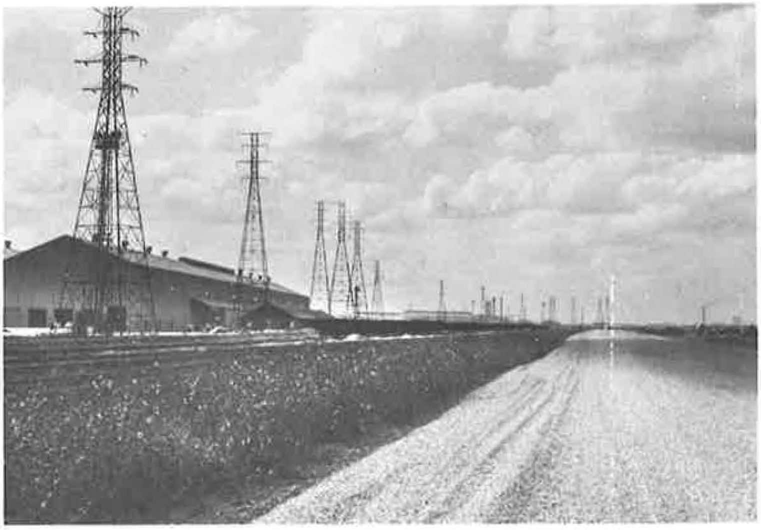


Figure 8. High-strength low-alloy steel 69-kv transmission towers, Gary Steel Works.

tion, have been made and are under programmed surveillance for future reporting. A brief description of some of the more interesting of these is given in the following.

Figure 8 shows a few of the seventeen 69-kilovolt (kv) double-circuit transmission towers located on the south shore of Lake Michigan within the Gary Sheet and Tin Works of the U.S. Steel Corp. The towers were erected early in 1960. Atmospheric conditions at this location—frequent lake breezes and mist, sunshine, rain, and proximity to an industrial plant—are optimum for the rapid formation of the protective oxide film on the unpainted steel. Recent examinations of these towers have shown that the rust film is tightly adherent to the base metal and has assumed a dark brown color.

Figure 9. Appearance of high-strength low-alloy steel in Project EHV after 2 yr near Pittsfield, Mass.

Transmission tower No. 18 in General Electric Company's Project EHV (11) was erected of high-strength low-alloy steel in a semi-rural area near Pittsfield, Mass., in 1960. Figure 9 shows the surface after 2 yr of exposure. The texture of the rust is still slightly granular but is generally tightly adherent to the base metal. The structure has assumed a deep brown color.

At a new highway crossing in a rural area near Brookville, Pa., the West Penn Power Co. in January 1962 erected two 138-kv H-frame transmission towers. Figure 10 shows one of these structures. Eight months after erection, all surfaces, except the base of the tower legs, had developed a generally homogeneous reddish-brown protective oxide film. At the base of the tower legs, two light-colored streamers developed as a result of the nightly formation and condensation of low-level ground fog. Moisture condensed on the lower braces and drained down the

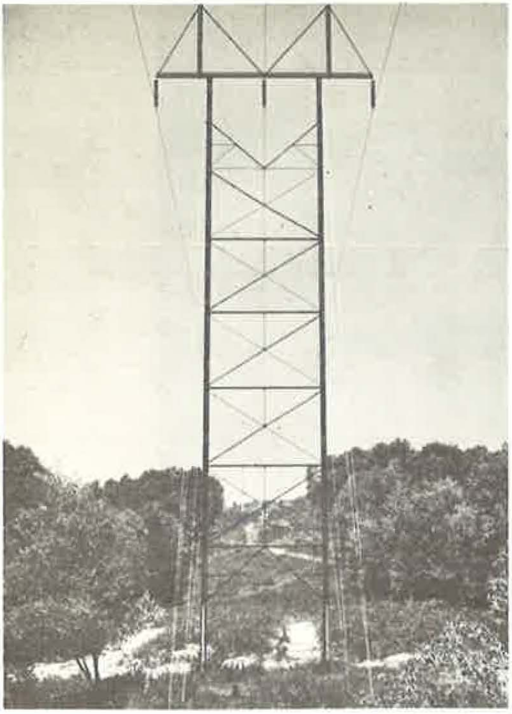


Figure 10. High-strength low-alloy steel transmission towers after 8 mo near Brookville, Pa.

tower legs, thus maintaining a wet condition that caused development of the color in the wash area to lag behind that in the balance of the tower. If appearance is an important factor, the water flow on this type of structure can be diverted by designing to facilitate drainage.

Figure 11 shows a high-strength low-alloy steel wide-flange beam (6 by 6 in., 15.5 lb/lin ft) used as a crossarm on wooden poles in a 115-kv transmission line. With the formation of the protective oxide, the general color of the arm tends to blend in with the color of the pressure-creosoted poles. In addition, the high-strength low-alloy steel arm is lighter and less expensive than the wooden arm it replaces, even with the cost of an additional insulator per string to make up for the insulation lost by the substitution of steel for wood. Many installations of this type have been in operation for about 6 mo.

The Virginia Electric and Power Co. of Richmond, Va., encouraged by the good performance of the aforementioned installations, is using high-strength low-alloy steel for all towers in its 350-mi 500-kv transmission line presently being erected in West Virginia and Virginia (12).

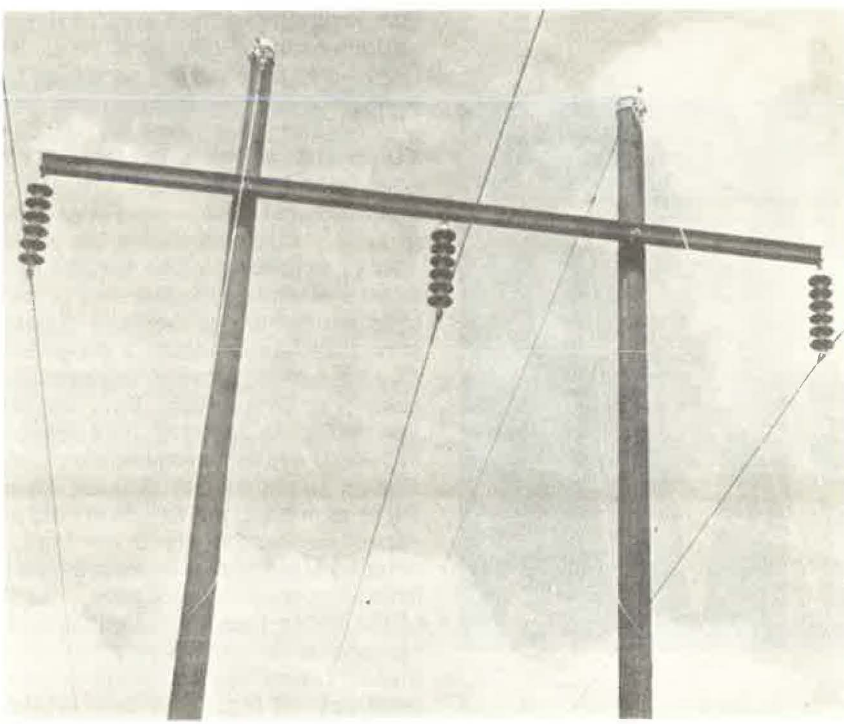


Figure 11. High-strength low-alloy steel crossarm application on creosoted poles.

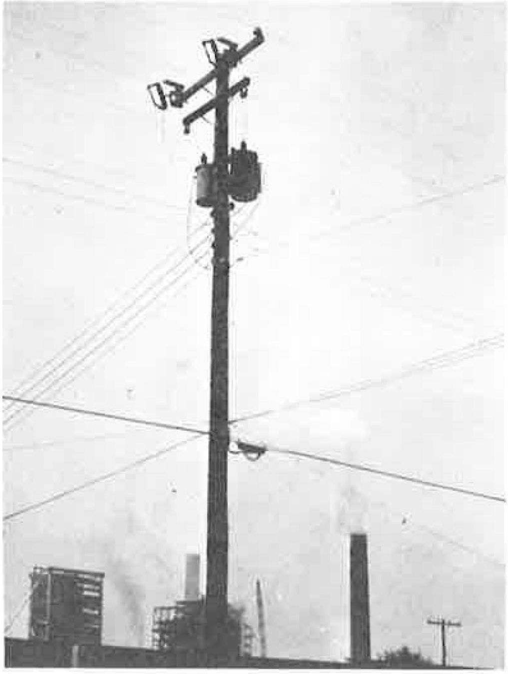


Figure 12. Effluent from smoke stack deposits on transformer tank of high-strength low-alloy steel and painted carbon steel, Georgetown, S.C.

In Georgetown, S. C., about 10 mi from the ocean and just outside a large paper mill, a 25-kva distribution transformer is incased in an unpainted high-strength low-alloy steel tank and cover. A painted carbon steel unit is mounted next to it. After 3 yr of exposure, a pleasing dark brown rust film has formed on the surface of the high-strength low-alloy steel tank and its cover. It has been the experience of the Georgetown utility company engineers that the painted carbon steel tanks require touch up maintenance at the end of 3 yr of service and complete repainting after 5 yr. When the 3-yr inspection was made in June 1964, large areas of paint were observed flaking from the carbon steel tank cover and base. The high-strength low-alloy tank was in good condition (Fig. 12). It is evident that this essentially rural area in close proximity to the paper mill actually is quite corrosive. These particular atmospheric conditions, however, effectively promote the formation of the dark tightly adherent oxide film.

Buildings

During the summer of 1962, a two-story building was erected on the campus of the University of South Dakota at Vermillion. Approximately 36 unpainted H-beam col-

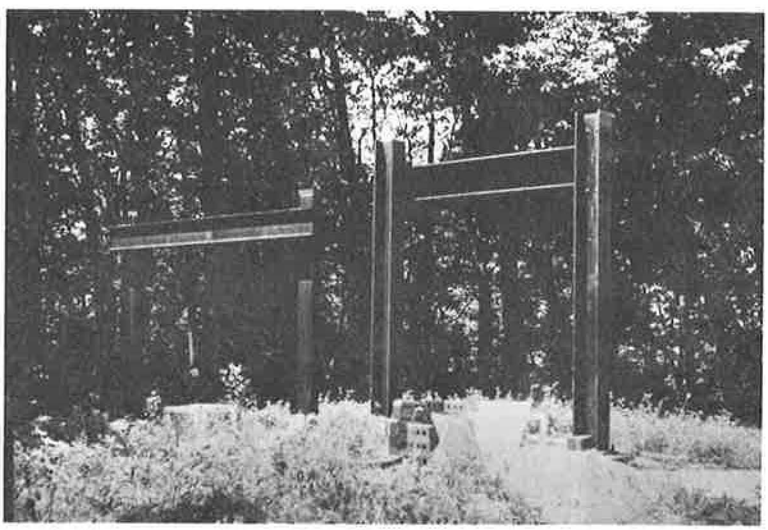


Figure 13. Mock-up of high-strength low-alloy steel in atmospheric exposure test, Moline, Ill.

umns were used as part of the exterior wall treatment. The rural location of the building will afford an excellent opportunity to observe and compare the development of the rust film on the H-beams with that on the transmission towers near Brookville, Pa.

Located on the outskirts of Moline, Ill., is one of the finest creations of Eero Saarinen Associates—the new Deere and Co. administration center. This complex includes a bridge leading to a seven-story administration building which in turn is connected with a two-story exhibition building. This group of structures represents the first instance in which unpainted high-strength low-alloy steel has been employed in the construction of bridge members and all exterior building columns, posts, beams,

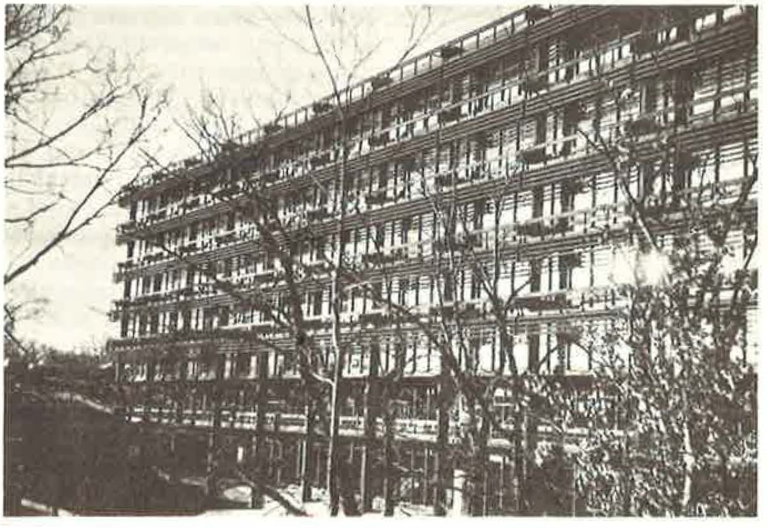


Figure 14. Exterior structural features of Deere and Company administration building, Moline, Ill.

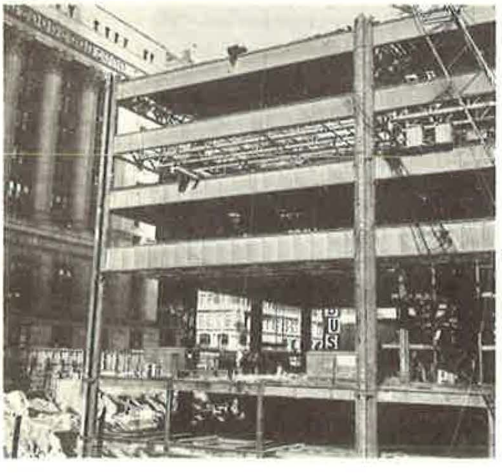


Figure 15. View of early stage of construction of Chicago Civil Courts Building.

girders, and sun control devices. Before this steel was specified, exposure tests of the type shown in Figure 13 were made over a 2-yr period beginning in October 1958 to establish the performance of the steel in the Moline environment. Figure 14 illustrates some of the exterior treatment of the main building which is a rigid frame all-welded structure. Additional construction details have been described in reports appearing in the Architectural Record (13, 24), the Architectural Forum (14), and Time (15).

In Chicago, where many architectural innovations have been introduced, the construction of the Chicago Civil Courts Building marks the first introduction of this steel in a multistory high-rise building in the heart of a metropolitan business center. This 680-ft, 32-story building described in the Architectural Forum is characterized by its exterior curtain walls of unpainted high-strength low-alloy steel spandrels and window sashes (16). Figure 15 is a view of an early stage of construction. Test panels of this high-strength low-alloy steel, exposed about ½ mi from the structure, indicate that the steel should reach a terminal color in about 2 to 3 yr. In each of these buildings the maintenance-free feature was a factor in the final choice of materials.

Other smaller buildings are being erected in several cities using high-strength low-alloy steel for beams, columns, and fascia. These sections are being used to serve as decorative as well as functional members.

Guardrails

Currently a number of high-strength low-alloy steel guardrail sections have been placed in service on the highways of 12 states. In some instances the material has been used as median barrier rail. The test sites are located in midwestern states as well as in states bordering the Atlantic and Pacific Oceans and the Gulf Coast. The specific exposure sites can be classified as being highly industrial, urban, and rural. Figure 16 shows the appearance some 6 mo after erection of a guardrail employed as median barrier. A rich brown oxide film has already begun to form.

The Michigan State Highway Department presently has under test a guardrail installation, similar in composition to the steel being discussed, on the approaches to one of the Interstate highways near Lansing. This test section has been exposed over two winters and has been subject to deicing salt applications and splashing from melted snow. The color and texture of the oxide film has not been affected, and the structure is continuing to exhibit its typical rustic color.

Light Standards

A high-strength low-alloy steel light standard was installed in 1960 in Monroeville, Pa., some 15 mi east of Pittsburgh. The oxide film in this semi-industrial location is developing at a satisfactory rate. One of the most impressive observations is that the light standard remains quite unobtrusive when compared with an adjacent light standard constructed of conventional materials. Figure 17 is a view of the two light standards taken during a picnic period. The high-strength low-alloy steel standard blends into the forested background, and the conventional light standard presents a contrasting

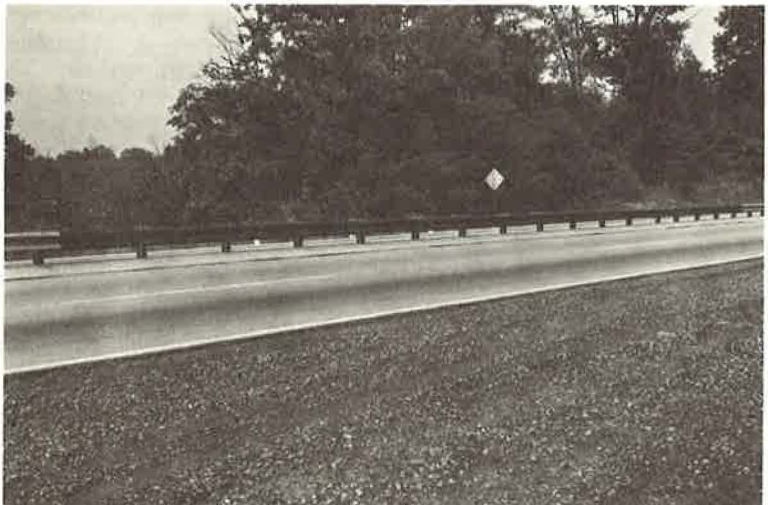


Figure 16. Experimental use of high-strength low-alloy steel in median strip.

Figure 17. High-strength low-alloy steel light standard in background.

appearance. Figure 18 is a view of a high-strength low-alloy steel light standard in a midwestern state, one of a group of 20 such test units that have been in service in this vicinity for about 2 yr.

Figure 18. High-strength low-alloy steel light standards in midwestern city.

Signposts

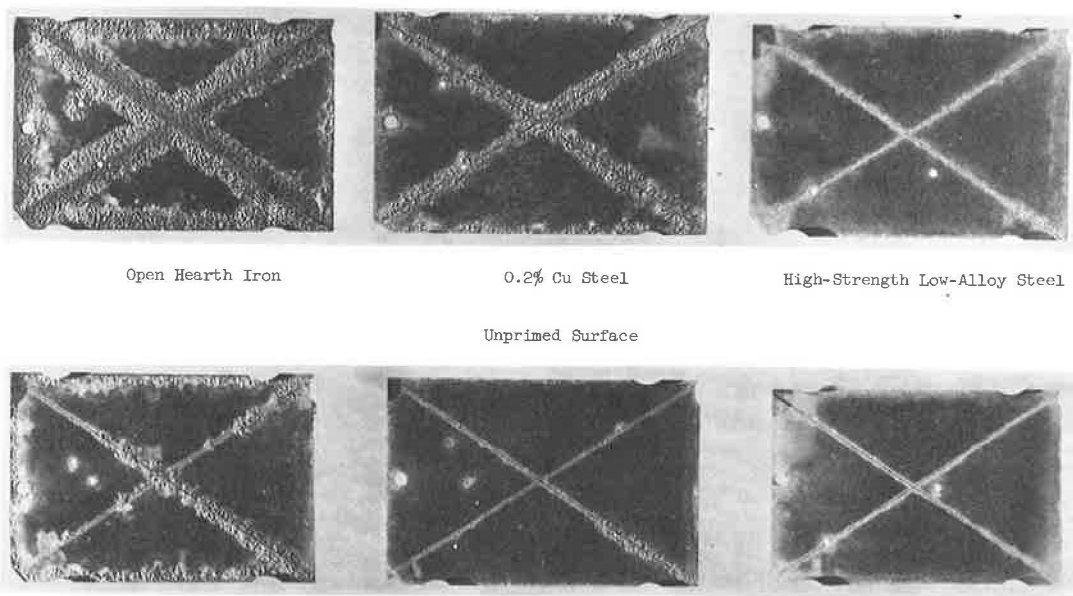
Highway signposts represent another large area where maintenance economies can be achieved through the use of corrosion-resistant high-strength low-alloy steel (5). In addition, a safety factor can be gained through its use. The results of numerous surveys designed to learn the best way of presenting information and instructions to speeding motorists have been reported in the literature. These studies included color combinations, sign configuration, sign height above ground level, letter size with respect to sign dimensions, and number of legs supporting the signs (17). It might also be desirable to conduct a study in which motorist-attention delay time in reading signs is correlated with attention-gaining colors used for signposts, legs, braces, and supporting arms. Making these structural members of maintenance-free dark-colored, high-strength low-alloy steel could contribute to highway safety and eliminate costs for painting and repainting.

Snow Fences

Snow fences represent an application where unpainted high-strength low-alloy

Figure 19. View of new snow fence design.

steel might be considered. The Roadway and Ballast Committee of the American Railway Engineering Association (18), in a report on methods of protecting roadways and track against drifting snow and sand, has evaluated the performance of perforated corrugated steel panels. This type of fence has been installed by several railroads, and comments on initial cost of materials and erection are favorable. The cost of installation and removal is comparable to those incurred with wood lath. The fence may be installed in the same manner as temporary lath fence and removed during the off-season. Because of the corrugations and perforations, stacking is simple, and the slots can be chained together to prevent pilfering. Presently, these fence panels are supplied painted and galvanized. However, they could be fabricated from high-strength low-alloy steel, in which case painting and touchup maintenance costs as well as galvanizing costs can be eliminated. Figure 19 illustrates the fence in service.



Open Hearth Iron | 0.2% Cu Steel | High-Strength Low-Alloy Steel

Unprimed Surface

Open Hearth Iron | 0.2% Cu Steel | High-Strength Low-Alloy Steel

Bonderite Primed Surface

Figure 20. Effect of surface treatment on painted steels after 8 mo of exposure in marine atmosphere 80 ft from Atlantic Ocean at Kure Beach, N.C.

Bridges and Overpasses

A major engineering consequence of the construction of the Interstate Highway System as well as of the expanded local highway systems is the increasing need for additional bridges and overpasses. This multiple use of space has resulted in a need for numerous medium- and short-span bridges. Together with the initial material and erection costs for such structures comes frequent paint maintenance to retain structural integrity and an esthetically pleasing appearance.

The Michigan State Highway Department has been actively seeking a steel for which painting and paint maintenance could be eliminated. Recently they selected a high-strength low-alloy steel, similar in composition to the one discussed herein, for use in the construction of a major overpass in the Detroit area. This unpainted overpass structure is now open to traffic. The performance of the steel in this application should demonstrate to bridge designers the versatility inherent in the high-strength low-alloy steels with enhanced atmospheric corrosion resistance.

The desire by engineers for steels possessing such properties as high strength and enhanced corrosion resistance will probably lead to the development of additional steels for bridge applications.

Another highway bridge built of unpainted high-strength low-alloy steel was erected in late 1964. This all-welded 3-span 110-ft long 2-lane bridge replaces an old 5-span

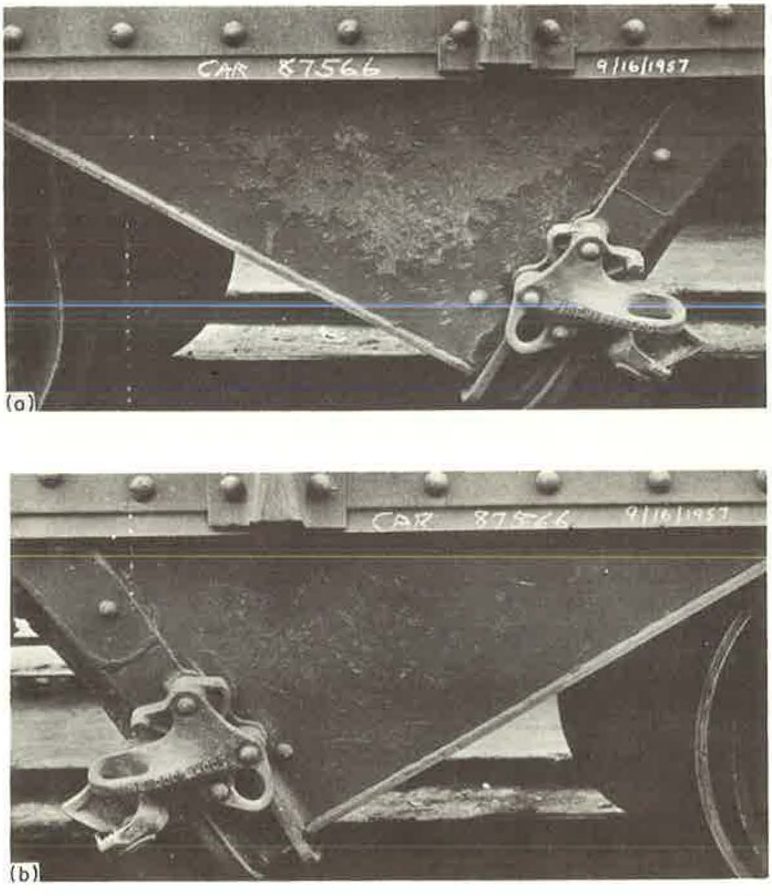


Figure 21. Appearance of painted hopper sheets: (a) copper-bearing steel, and (b) high-strength low-alloy steel.

bridge that crosses the Brush Creek in Miami County, north of Dayton, Ohio. A 4-span simple beam bridge of composite construction will be erected in 1965 as an adjunct to the New Jersey Turnpike near Morristown, N.J. This 192-ft long bridge will employ some 77 tons of a high-strength low-alloy steel in the unpainted condition. It is expected that the appearance of this bridge will be similar to that of high-strength low-alloy steel test specimens exposed in industrial areas. It should be noted, however, that these two bridges are considered experimental in the sense that if the unpainted structures are not acceptable to the bridge engineers, they may be painted.

Also, because there may be some objection to the appearance of rust stains on concrete abutments visible to the public, bridge designers employing high-strength low-alloy steels should provide for drainage of rainwater and melted snow or for painting of the affected surfaces.

Paint Performance

Hayes and Maggard of Purdue University, after evaluating the use of high-strength low-alloy steels in bridge construction, contended that the use of such steels is economically justified. They also indicated that additional economies could be achieved through a 50 percent increase in paint life (20, 21, 22). Studies by LaQue and Boylan show that the performance of an automobile paint finish on the high-strength low-alloy steel, when scribed, was very similar to that on a steel that has been bonderized before painting. Figure 20 illustrates the performance of specimens exposed 80 ft from the ocean at Kure Beach, N.C. Pretreatment of steel with phosphate compositions insures against undercutting of the paint film when it is damaged. Because the oxide film on the high-strength low-alloy steel is thin, it does not undercut paint films and thus permits the paint to perform its primary task of protecting the steel surface. Copson and Larrabee (23) also present evidence concerning the extra durability of paint on high-strength low-alloy steel. They demonstrated the performance of paint on sheets of copper-bearing steel and high-strength low-alloy steel on the sides of a railroad hopper car. After 5 yr of service, the paint on the copper-bearing steel sheets had come off relatively large areas, whereas the paint on the high-strength low-alloy steel sheets continued to give adequate protection. Figure 21 shows the performance of the paint in this service test.

SUMMARY

It is evident from the foregoing discussion concerning the mechanical and corrosion-resistant properties of the high-strength low-alloy steel that a structural material fulfilling the requirements of highway engineers and landscape architects is available and proven for use in highway applications. Because this steel develops a dark oxide film, permitting it to blend into the natural background of trees and shrubs, the landscape architect has at his disposal a material that minimizes marring of the natural scenery when used for roadside rest area buildings, signposts, etc. Engineers requiring poles, towers, and light standards that must border highways and freeways have a strong durable product that remains unobtrusive while serving a utilitarian purpose. Bridge engineers who may be seeking new means for introducing strength and economy into their bridge designs have a new construction material available on an experimental basis. Safety engineers concerned with improving sign performance also have a material that can be used selectively to solve their problems. Finally, those engineers and planners concerned with the economic phases of initial construction costs and subsequent expenditures for maintenance and appearance may choose this material because they can "build it and forget it."

REFERENCES

1. Lemly, J. H. Non-Vehicular Benefits from Utility Use of Streets and Highways. Highway Research Board Sp. Rept. 75, pp. 1-33, 1962.
2. Nelson, C. E. Economic Implications of Utility Use of Highway Locations in Utah. Highway Research Board Sp. Rept. 75, pp. 33-51, 1962.

3. Pushkarev, Boris. Highway Location as a Problem of Urban and Landscape Design. Highway Research Record No. 23, pp. 7-18, 1963.
4. Garmhausen, W. J. Roadside Rest Requirements on the Interstate Highways. In Roadside Development 1962, pp. 40-44, 1962.
5. Iowa State Highway Maintenance Study. Highway Research Board Sp. Rept. 65, Supp. I, pp. 150-151, 1961.
6. Specifications for High-Strength Low-Alloy Steel, ASTM Designation: A 242-63T. ASTM Standards 1963, Pt. 4, pp. 257-259.
7. Buck, D. M. Influences of Very Low Percentages of Copper in Retarding the Corrosion of Steel. Proc. ASTM, Vol. 19, Pt. II, pp. 224-237, 238-246, 1919.
8. Reports of Committee A-5, Corrosion of Iron and Steel. Proc. ASTM, 1916-1951.
9. Larrabee, C. P. Corrosion Resistance of High-Strength Low-Alloy Steels as Influenced by Composition and Environment. Corrosion, Vol. 9, pp. 259-271, 1953.
10. Jordan, H. V., et al. Sulfur Content of Rainwater and Atmosphere as Related to Crop Needs. USDA Tech. Bull. No. 1196, Superintendent of Documents, U.S. Govt. Print. Off., Washington, D.C.
11. Alpert, S. D., et al. Metal Towers for 650-kv Operation on Project EHV. Paper presented at AIEE Winter Gen. Mtg., New York, Feb. 1961.
12. Rawls, J. A. VEPCO 500-kv Line Advances New Design Ideas. Electrical World, Vol. 158, pp. 44-46, 72-73, Nov. 26, 1962.
13. Dinkaloo, John. The Steel Will Weather Naturally. Architectural Record, pp. 148-150, Aug. 1962.
14. John Deere Sticks of Steel. Architectural Forum, Vol. 121, No. 1, pp. 76-85, 1964.
15. Architecture: The Plowman's Palace. Time, Vol. 84, No. 6, pp. 64-65, Aug. 7, 1964.
16. Architectural Forum, Chicago Civic Center, Vol. 116, pp. 110, 129, May 1962.
17. Burg, A., and Hulbert, S. F. Predicting the Effectiveness of Highway Signs. Highway Research Board Bull. 324, pp. 1-11, 1962.
18. Metal Fence to Check Drifting Snow and Sand. Amer. Railway Eng. Assoc., Bull. 584, pp. 502-504, Feb. 1964; Ann. Proc. Amer. Railway Eng. Assoc., Vol. 65, 1964.
19. Hayes, J. M., and Maggard, S. P. Economic Possibilities of Corrosion-Resistant Low-Alloy Steel in Short Span Bridges. Proc. Amer. Inst. of Steel Constr., Nat. Eng. Conf., pp. 59-68, 1960.
20. Hayes, J. M., and Maggard, S. P. Economic Possibilities of Corrosion-Resistant Low-Alloy Steel in Welded I-Section Stringer Highway Bridges. Highway Research Board Proc., Vol. 41, pp. 125-162, 1962.
21. Hayes, J. M., and Maggard, S. P. Economic Possibilities of Corrosion-Resistant Low-Alloy Steel in Box Girder Highway Bridges. Highway Research Record No. 76, pp. 102-139, 1965.
22. LaQue, F. L., and Boylan, J. A. Effect of Composition of Steel on the Performance of Organic Coatings in Atmospheric Exposure. Corrosion, Vol. 9, pp. 237-241, 1953.
23. Copson, H. R., and Larrabee, C. P. Extra Durability of Paint on Low-Alloy Steels. ASTM Bull., Dec. 1959.
24. Dinkaloo, John. Bold and Direct Using Metal in a Strong Basic Way. Architectural Record, pp. 135-142, July 1964.

The Lizotte Bridge

A Review and Discussion of "The Case for Galvanized Bridges" by J. R. Hall *

ROGER MAIN, Thomas Gregory Galvanizing Works, Maspeth, New York

ABRIDGMENT

•THE FIRST hot-dip galvanized conventional steel bridge in the world was recently opened to traffic near Deschaillons, outside of Quebec City, Canada. Problems investigated by the Quebec Department of Public Works included plant and kettle capacity of local galvanizers, possibility of warpage, acid entrapment, physical effects of pickling and galvanizing, and the slip resistance of galvanized connections.

It was found that the use of high-tensile bolts, designed to develop full friction at the contact surfaces, can be recommended. No effects on the physical properties were observed as a result of galvanizing. No prohibitive undesirable effects on the physical properties of the steel will result from a properly controlled galvanizing process. Excessive warpage of large members of roughly symmetrical cross-section will not occur. Proper design and good fabricating technique will prevent acid entrapment. Current capacity is up to 80 ft, with larger galvanizing facilities awaiting demand.

The galvanized bridge is a three-hinged arch structure joining river banks 400 ft apart. It is designed for H-20 loading, weighs almost 400 tons, and has a clear midspan of 200 ft. It cantilevers 60 ft from the piers carrying 45-ft suspended box girders. The deck-type truss is composed of rolled wide-flange shapes and some welded H-sections. The longest members are the box girders and the 49-ft bent chord sections. The box girders are 45 ft long, have a cross-section of 2 by 3½ ft, and weigh 4½ tons each.

The added cost of galvanizing was about $10,000, or 3 percent of the cost of the entire project. If a painting program is instituted before the evidence of first rust, this bridge will remain in as-new condition structurally and esthetically, and will cost the taxpayers less than any other combination of materials or coatings. Considering the numerous advantages of steel as a structural material, it may be logically concluded that the use of galvanized steel for small bridges and highway structures offers advantages unchallenged by other materials.

* (Bond Metal Finishers Co., Ltd., 1565 Cabot St., Montreal, Quebec)

Paper sponsored by Committee on Metals in Highway Structures.

Soil Resistivity as Related to Underground Corrosion and Cathodic Protection*

W. J. SCHWERDTFEGER, Corrosion Section, National Bureau of Standards

ABRIDGMENT

•CORROSION data are based on measurements made on about 4,500 specimens of commonly used plain ferrous materials which had been buried in backfilled trenches at 86 National Bureau of Standards sites for periods up to 17 yr. The metals exposed consisted of open hearth iron, hand-puddled and mechanically puddled wrought irons, and open hearth and Bessemer steels, all without alloying constituents. The soils ranged in resistivity from 50 to 54,000 ohm-cm and in pH from 2.6 to 10.2.

For periods of exposure up to 5 yr, the maximum pit depths are on the average deeper in soils with resistivities up to 500 ohm-cm than in soils with higher resistivities. In the higher resistivity soils (> 500 ohm-cm), there appears to be no regular variation between maximum pit depth and soil resistivity. However, for periods of

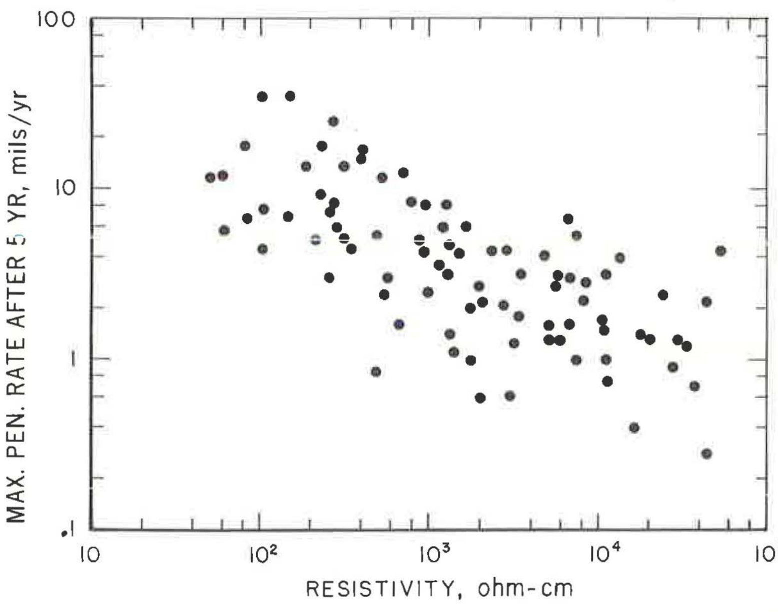


Figure 1. Rates of penetration of specimens based on maximum pit depth-time curves, from 5 yr to from 12 to 17 yr (for majority of soils).

*The complete paper, on which this abridgment is based, has been published in the Journal of Research of the National Bureau of Standards, Vol. 69C, No. 71, Jan.-Mar. 1965.

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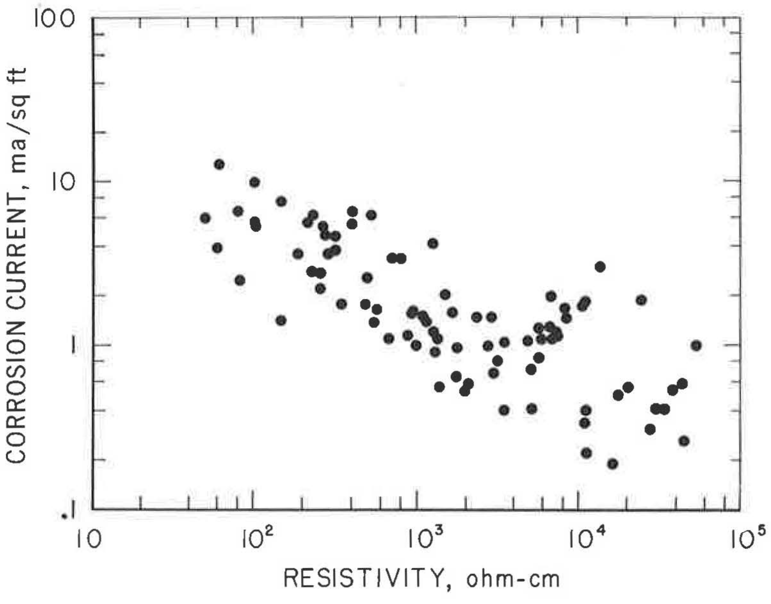

Figure 2. Calculated corrosion current densities based on weight loss rates between 5 and 12, and between 5 and 17 yr of exposure for majority of soils.

exposure longer than 5 yr, the rate of maximum penetration, in general, decreases as the soil resistivity increases, as shown in Figure 1. According to an empirical equation, increasing the area of metal exposed from that of the specimens (0. 4 sq ft) to that equivalent to the exterior surface (45. 16 sq ft) of a bare 20-ft length of 8-in. diameter pipe approximately doubles the maximum penetration rates given in Figure 1.

The weight losses on the same specimens were converted to corrosion current densities. The corrosion rates after 5 yr are shown in Figure 2. The pattern is similar to that of Figure 1, indicating a correlation between weight loss and maximum pit depth.

The current densities required for cathodic protection are usually greater than the current densities associated with corrosion. The data (Fig. 2) can be adjusted to give an estimate of the current densities probably necessary for the cathodic protection of bare metal surfaces similarly exposed. This does not imply that current density is being substituted for polarization requirements as a criterion for cathodic protection, but that current densities might be estimated to fulfill the polarization requirements. A factor of 1. 5 is suggested for soils with resistivities up to 5, 000 ohm-cm, 2. 0 for soils from 5, 000 to 10, 000 ohm-cm, and a factor of 3. 0 for soils with resistivities greater than 10, 000 ohm-cm. The higher corrosion rates existing during the first few years of exposure indicate that it is probably economically feasible to wait for about 2 yr before measuring current densities necessary for cathodic protection.

Study of Inspection Methods and Quality Control for Welded Highway Structures

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W. H. MUNSE, Professor of Civil Engineering, University of Illinois

•THE PRIMARY purpose of weld inspection in highway structures is the control of weld quality. This purpose can be fulfilled, in part, by the detection and elimination of flaws which may cause failure.

There are numerous methods of inspection available to detect flaws in a weld. However, no matter what method is used, it is necessary to determine if the severity of the observed flaw is sufficient to warrant correction. Numerous recommendations and discussions are available to assist in this determination, but the inspector must make the final evaluation and decision. If these flaws are objectionable, a satisfactory weld can generally be obtained by proper incentive for the welder or by changes in the welding procedure.

In general, weld flaws can be classified into five categories: lack of penetration, cracks, slag inclusions, porosity, and harmful surface defects. The acceptance or rejection of the weld is usually determined by limits specified in each of these categories. Although various specifications require rejection of welds based on different relative severities for the different types of flaws, nearly every specification requires rejection for lack of penetration or the presence of a crack. The bases for rejection for the other types of flaws are so varied that often only general indications of them can be presented.

The available methods of inspection may be divided into three general categories: destructive tests, proof tests, and nondestructuve tests. Normally, the destructive tests, such as tension and bend tests, are mechanical and are used to determine whether the welder is qualified to fabricate the actual structural weldment or if the welding procedure will yield a satisfactory welded joint. Since the part is usually tested to failure, its usefulness as a component of the structure is destroyed. Therefore, these tests are generally conducted on a standard series of test specimens or on a sample specimen of one of the structural components.

Proof tests are another type of mechanical test and consist of applying to the structure or member a load or pressure equal to or exceeding that expected in service. The load or pressure applied to the structure should not, however, be great enough to damage the structure or to cause failure later at the service load or pressure. The selection of the proof load, required to indicate whether there are defects that might result in adverse behavior or failure in service of the structure or component part, is a matter of experience and judgment. Although proof tests are required for many types of welded structures, their use for welded highway structures is very limited because of the nature of the loads and design criteria for these structures. Since their use is limited, no further discussion of proof tests will be presented in this report.

The third, and the most widely used, type of inspection method is the nondestructive test. The methods of nondestructive inspection may be divided into the following classifications: visual, trepanning, radiography, dye penetrant, magnetic particle, and ultrasonic. None of these inspection methods completely fulfills the requirements for

Paper sponsored by Committee on Steel Superstructures.

the inspection of all types of weldments. It is necessary, in most cases, to supplement a basic inspection procedure with one or more of the other inspection methods. The types of tests required for inspection of a specific structure or component of a structure may vary considerably, depending on many factors associated with the design and loading of the structure. For example, if failure of the weld will cause severe damage to the structure and possibly loss of life, a searching sensitive inspection procedure will be necessary to insure adequate weld quality. The selection of the inspection methods and the acceptance criteria are generally based on the judgment and experience of the designers and owners of the structure.

In addition to loading and design criteria for a structure, there are numerous other factors which should be considered when selecting inspection methods. In certain cases, some of the methods mentioned above cannot be used because the necessary equipment is not portable enough to be available at the point of inspection. If this limitation does not exist, the decision as to the type of inspection method is generally based on one or more of the following considerations: sensitivity or resolution required, material to be inspected, geometry of material, method of fabrication, and types of defects possible or expected. Because of the differences in relative importance of the various welds in a structure, any of these factors may determine or control the method of inspection for the different welds.

A survey was conducted in 1960 of all state highway departments to determine which of the inspection methods they use for the various components and members of welded highway structures. A number of changes occurred in the welding practices of the highway departments after this survey and in 1963, the American Welding Society (AWS) issued a new specification (1) which incorporated a number of changes in the requirements and recommendations for weld inspection for bridges. Also, since the original survey, the amount of welding in highway bridges had substantially increased and a number of highway departments had changed their inspection program. To bring the information in the original survey up to date, requests for a review of the weld inspection information were again sent to all state highway departments in November 1963. Their replies to these surveys indicate the overall judgment and experience of a number of engineers and inspectors, and provide excellent guidance for the selection of the inspection methods and for the development of an inspection specification. A summary of both surveys and a detailed listing of the replies of the highway departments to the 1963 survey are presented later in this report. With a study of the sensitivity and uses of the different inspection methods and an examination of the procedures used by the various highway departments, inspection procedures necessary for the proper control of weld quality in highway structures can be better selected. Detailed inspection specifications are not presented in this report; however, references are made to a number of currently available inspection codes.

METHODS OF INSPECTION

Welding inspection has become an integral part of any weld fabrication. Through the years, experience has been gained which has helped considerably in defining the requirements necessary for welding inspection. The inspection methods selected are, however, still largely dictated by judgment and experience.

In this report, the principles underlying the operation of each of these inspection methods are briefly explained and an attempt is made to evaluate their adaptability to fulfill the requirements necessary for welding inspection in welded highway structures.

Visual Inspection

It should be pointed out that in the broad sense, all methods of welding inspection may be considered visual. In this report, however, visual inspection is considered inspection performed without the use of auxiliary equipment other than small hand pieces, such as a flashlight or a magnifying glass.

Because of the expense involved and the time required, it is generally impossible to examine thoroughly the internal and external condition of every weld in a highway structure. Therefore, it is necessary to have a comprehensive visual inspection method available during and after completion of welding. Since the only equipment commonly used in visual inspection is a light weld-size gage and magnifying glass, the cost of a complete visual inspection program is generally limited to the cost of inspection personnel. Numerous guides are available to assist the inspector in making decisions concerning the reliability or quality of a weld (2-7).

Visual inspection should begin on the component parts and be performed at various stages during fabrication. With a knowledge of the quality of the weldment at the different stages of fabrication, the decision to accept or reject the weld can then be based on a number of factors.

At the time of fit up, the material should be inspected. A check of the size and shape of the pieces should be made and all heavy scale, grease, paint and oil should have been removed. After root chipping or gouging, inspection is recommended because of the importance of the initial weld pass in the overall behavior of the weldment. The initial pass tends to cool very quickly thereby trapping gas. Also, because of the rapid cooling, this pass is highly susceptible to cracking. Thereafter, every stage of welding should be checked because when the welding is completed, only the surface can be inspected visually. As each layer of the filler metal is deposited, the inspector should, if possible, check the layer for defects. Before the subsequent layer is deposited, visible defects, such as cracks, are accessible and can be remedied. On completion of welding, a visual examination should be conducted to determine conformance to specifications, weld appearance, external flaws (cracks, crater cracks, overlap, undercut, etc.), and dimensional accuracy of the weldment.

In many instances, visual examination of each layer of filler metal is not possible. In this case, however, an indication of the internal condition can be obtained from the external appearance of the weld. In general, if a welder has taken care to place the final passes properly, he has done so also on the other passes.

Although the final acceptance of most structural welding is determined mainly from the appearance and visual inspection of the weld, critical welds require supplemental internal and external inspection in conjunction with a thorough visual examination. It should be emphasized, however, that although in many instances these other methods are necessary, they are principally used to supplement a well-conducted visual inspection program.

Visual examination, it should be pointed out, is limited to surface imperfections, and sensitivity depends on width of defect, light reflection, degree of surface smoothness and, most of all, the skill and judgment of the inspector. However, the following quality factors can usually be determined by visual means (3):

1. Dimensional accuracy of the weldment (including warping);
2. Conformance of the finished weld to specification requirements regarding the extent, distribution, size, contour and continuity of welds;
3. Uniformity of weld appearance; and
4. Surface flaws, such as cracks, overlaps, undercuts, surface porosity and unfilled craters.

It should be emphasized that the correct evaluation and interpretation of any discrepancies in the appearance of a weldment is the essential part of visual inspection. To provide a proper evaluation, it is necessary to have a sound knowledge of the service requirements of the weldment and of the welding processes involved in its fabrication, as well as the judgment and experience required to evaluate the quality of the weldment.

Trepanning

Trepanning (3, 8) consists of the removal and examination of a small disc or ring containing the weld by means of a tubular tool with sawteeth around its end. Although classified as a method of nondestructive testing, trepanning does destroy a portion of the weldment. In many cases, such as locations where brittle fracture or fatigue behavior is critical, the removal and subsequent rewelding may be more damaging than the initial defects. However, a trepanning plug is sometimes necessary since it provides an excellent sample for metallurgical studies.

The equipment necessary for trepanning is relatively inexpensive. In addition to a large air or electric drill, the only tools necessary for removing the plug are a holder for the drill, a pilot drill, and a hole saw. The pilot drill, usually ¼ in. in diameter, is used first to drill the guide hole. The hole saw is then used to remove the sample. If necessary, the hole may then be rewelded.

Although trepanning is not used very extensively in present engineering practice, it is the most suitable method under several conditions. It is sometimes the most satisfactory inspection method when only one side of a specimen is available, when a metallographic test is necessary, or when spot sampling is desired.

If the sample is to be used for a metallographic test, the specimen is usually either cleaned by acid or ground, polished, and etched until there is a clear definition of the weld structure. The metallurgical structure may be examined for number of layers of filler metal, microstructure, weld profile, weld contour or hardness, and fusion.

If trepanning is used as a spot sampling method, a sample is generally taken at about every 50 ft of welding. When a defect is located, additional specimens are taken until the limit of the defective welding has been established. Where a single defect in a weld might result in a serious failure, a complete subsurface examination (radiographic or ultrasonic) should be used. Spot checking or sampling is only useful in controlling the average quality of the welds.

Radiography

Radiography (3, 4, 5, 9, 10) includes a number of inspection methods for the determination of the internal quality of a weld. The three basic methods employ X-rays, gamma rays or fluoroscopy. Although variations of all three methods are used for inspection purposes, this discussion is limited to X-ray and gamma-ray radiography because of their extensive use. The use of fluoroscopy to examine welds in a welded highway structure is normally not feasible.

Basically, radiography is the passing of rays through an object; the rays land on a film or screen, revealing or recording the internal structure of the test object. X-rays and gamma rays are electromagnetic waves of short wavelength capable of penetrating materials opaque to longer light waves. Some of the radiation passing through the object is absorbed, depending on the wavelength, the density of the material and its thickness. However, more radiation passes through a void in a uniform thickness of material than through the material itself. Since radiation affects photographic film in proportion to intensity and time, the area of film under the void receives more radiation and appears darker as a shadow image.

The basic operation of both radiographic methods is essentially the same; however, the operation, cost and construction of the equipment varies greatly. X-rays are produced by electrical means, are generally limited to one direction, and occur only when the power source is operating; gamma rays are radiated from isotopes and are continuously transmitted in all directions. Because each may be best suited for different applications, it is not easy to compare their merits. The longer wavelength of X-rays results in higher contrast radiographs; therefore, where high contrast is needed, X-ray radiography is more suitable. Isotopes radiate gamma rays in all directions and allow the simultaneous inspection of a number of specimens. However, the principal advantage of isotope radiation is that it is very portable and does not require any additional power source, but extreme care must be taken in operation to guard against radiation hazards.

Some of the numerous factors that affect the selection of the type of radiography are material density, thickness of material, time available, accessibility and economics. For example, the initial cost for X-ray equipment is relatively large, whereas the initial cost for a gamma-ray source is less but repeated replacement is necessary because of its decay.

X-ray machines are available in sizes from 10 to 1,000 kv or more. However, highway structure inspection is generally limited to 200-kv machines because the larger capacity units are very difficult to transport.

There are four radioactive sources commonly used in gamma radiography—cobalt-60, iridium-192, thulium-170 and cesium-137. The choice of the source generally depends on the type and thickness of material to be inspected, maximum allowable exposure time, cost, sensitivity and frequency of exposure. For thicknesses larger than about 2 in., it is almost a necessity to use an isotope source because the size of X-ray machine required is large and difficult to transport, whereas a small change in the size of the radioactive source is all that is necessary in isotope radiation.

After selecting the radiographic source to be used, the specimen is exposed to this source. The radiographic film, which has been previously placed against the specimen on the side away from the source, is also exposed. The amount of radiation received by the film depends on the density of the specimen material. A defect within the material will cause an increase or decrease, depending on the type of defect, in the radiation reaching the film. For example, a slag inclusion in a weld will appear as an irregularly shaped light shadow on the radiograph. This defect can be readily distinguished from porosity and cracks, which appear as rounded dark shadows of varying sizes and as a fine straight or wandering dark line, respectively.

Gamma-ray and X-ray radiographic inspections are usually conducted according to the AWS specification (1, Appendix E), ASTM Specification E 94-52T, or the ASME Boiler Code for Unfired Pressure Vessels.

After obtaining an indication of the defects in a radiograph, the inspector must decide if they are severe enough to require correction. Several sets of reference radiographs are available to assist in this determination: ASTM Specification E 99-63, U.S. Navy Bureau of Ships Radiographic Standards, and the International Institute of Welding International Collection of Reference Radiographs of Welds. These reference radiographs show typical examples of the various defects encountered in welding and indicate the relative severity of each one. The limits of acceptability are defined in the applicable specifications.

The selection of one of the methods of radiographic inspection as the prime or supporting method of inspection in any welded structure is based on the evaluation of the advantages and disadvantages of the inspection method. The advantages are as follows:

1. Permanent record of inspected weldment, thus making it the most positive inspection method within its range;
2. Positive identification of defects; and
3. Good sensitivity in that defects with thicknesses of less than 2 percent of the thickness of the base material can be found.

The disadvantages (depending on the type of radiographic source) may be summarized as follows:

1. Health hazard from radiation requiring precautionary measures during operation;
2. Cost of equipment;
3. Size and weight of equipment and time loss for exposures (directly related factors since size and/or weight may be reduced in many instances if additional time is allowed for exposure and vice versa);
4. Applicable to only a limited number of joint types and only when both front and back of weld are accessible; and
5. Results available only after film has been exposed and developed (not true of fluoroscopy).

In determining the relative value of the advantages or disadvantages, the type of structure to be inspected should be considered. The many disadvantages generally preclude the use of radiography for 100 percent inspection of a part of a structure unless a service failure of that part would endanger the life of the structure and the lives of individuals. It should be remembered that radiography is not the final answer in inspection methods, but rather an aid to the control of weld quality.

Dye Penetrant Tests

Dye penetrant tests (4, 11) are limited to the detection of surface defects and subsurface defects with surface openings. These tests are highly sensitive and are useful in detecting very small surface discontinuities. Dye penetrants are especially useful in inspecting nonmagnetic materials where magnetic particle tests cannot be used. The flow properties of the penetrants enable detection of defects that would not be seen by visual or other means of inspection. This method is applicable to all homogeneous materials except those of a generally porous nature where the penetrant would seep into and drain from the pores in the surface.

The cost of inspection with dye penetrant is relatively small since the only materials necessary are prepared solutions or powders of penetrant, emulsifier, and developer. The basic steps of the operation are as follows:

1. Liquid penetrant is applied to surface of object;
2. Time is allowed for liquid to penetrate defects;
3. Excess penetrant is removed by emulsifier;
4. Absorbent powdered material (or liquid) is applied to the surface;
5. Developer acts as a blotter and draws out penetrant in defects; and
6. Penetrant diffuses in developer indicating location of defects.

After the defects are located by the dye penetrant, additional examination by other methods of inspection may be desired to indicate further the extent of the defect.

Magnetic Particle Tests

Magnetic particle tests (4, 6, 12) may be used to detect, in ferromagnetic materials, discontinuities at the surface, and under certain conditions, those which lie completely under the surface. Nonferromagnetic materials or any other material which cannot be strongly magnetized cannot be inspected by this method. However, with suitable materials, magnetic inspection is highly sensitive to surface defects.

There are three basic operations in a magnetic particle test:

1. Establishing a suitable magnetic field in the test object;
2. Applying magnetic particles (in either dry form or solution) to the surface of test object; and
3. Examining the test object surface for accumulation of the particles (indication of defect) and evaluating the defect.

The tests are usually conducted in accordance with ASTM Specification E 109-63.

The only appreciable cost for this type of inspection is for the equipment to produce the magnetic field. In general, one of the following types of equipment is used: alternating current, direct current, half-wave rectified current or permanent magnets. In some cases, motor generator welding machines may be used as a source of power and the only additional cost is the magnetizing prods.

For the detection of surface cracks, an a. c. magnetizing current should be used; for subsurface defects, a half-wave rectified current with dry magnetic powder is necessary. Therefore, equipment is needed that will produce either half-wave or alternating current. The selection of the type of current is determined by the depth of penetration desired by the magnetic field.

When the inspector is operating the equipment, he may obtain indications of surface discontinuity, subsurface discontinuity, or nonrelevant magnetic disturbance. With little experience, he can readily differentiate between them. However, to differentiate between the various types of subsurface discontinuities, such as slag inclusions, inadequate penetration, and incomplete fusion, requires considerable experience.

The indication of a surface defect is usually a sharp line (orientation) of magnetic particles since the magnetic field is broken at the defect. The indication of a subsurface defect is formed by the defect forcing the magnetic flux lines to break through the weld surface and appears as a slight orientation or "gathering" of the magnetic particles above the defects at the surface of the weld. As the depth of the defect increases, the size of the defect must increase, so that even a slight orientation of the particles will occur.

A permanent record of the defect may be obtained by making a line sketch or photographing the magnetized particles. The magnetic particles may also be transferred in their magnetized position from the weld to a permanent record sheet by transparent adhesive tape.

This method of inspection of welded highway structures has found its main use in the examination of non-critical but load-carrying welds. In the case of welded built-up girders, it is commonly used to inspect the flange-web fillet welds and as an alternate method for compression and web butt welds. In each of these instances, its major purpose is to locate severe surface defects, such as cracks, rather than subsurface defects.

For normal field applications and many shop uses, the magnetizing current available is too low for detection of subsurface defects. Its use, therefore, should be limited to welds where subsurface defects will not be critical in determining the behavior of the weldment. This method, as all other methods of inspection, should not be used as the sole inspection technique but as a supplement to a complete visual inspection program.

Ultrasonic Inspection

Although ultrasonic inspection (3, 5, 13, 14) has been used in other fields and in some phases of structural inspection for a number of years, it has only recently been introduced in the inspection of welded highway bridges. Because of its undeveloped potential, ultrasonic inspection offers possibilities for reliable weld inspection. Not only can ultrasonic inspection be used for flaw detection, but also for thickness measurements and study of the metallurgical structure.

This method of inspection makes use of an electrically timed wave of the same type as sound waves but of a higher pitch (1 to 25 megacycles per second). The signal wave is propagated into the test piece and a portion of the signal is reflected by any discontinuities. The original and reflected signals are shown on a cathode-ray tube by a series of "pips" or vertical indications. Since the length of time (or distance between pips) is proportional to the distance traveled, the distance to the discontinuity, if any, can be determined. By taking readings at several locations and interpreting the width and height of the pip, an indication of the relative size and shape of the discontinuity can also be obtained. The procedures outlined in ASTM Specification E 113-55T are usually the bases for the inspection.

The use of ultrasonic inspection is limited, though, by the following basic deficiences (3):

1. Lack of a permanent photograph of weld defects;
2. Great dependence on the skill of the operator; and
3. Difficulty in establishing a standard of acceptance.

A photograph of the screen of the cathode-ray tube can be taken to give a type of permanent record. Some of the advantages of ultrasonic testing are high sensitivity (greater than radiography), greater penetrating power (detects flaws in steel thicknesses up to 20 ft), fast response, need for access to only one surface of specimen, and portability of equipment (units available weighing only 35 lb). Although there are several difficulties encountered in the use of ultrasonics for inspection of welded highway structures, these are rapidly being overcome, yielding an accurate, rapid and relatively inexpensive method of inspection.

At present, ultrasonics is used extensively by engineers in the inspection of welds in several types of structures, for example, oil storage tanks and gas pipelines. The transfer of this method to extensive use in structural applications and the inclusion of this method as an acceptable inspection procedure in the specifications seems only a matter of time.

Destructive Testing

Destructive tests give a numerical measure of the property under consideration and are tests to failure or destruction. They are usually limited to qualification tests conducted to indicate the ability of a welder to fabricate a weldment that he will later be required to fabricate in an actual structure or to obtain a comparison between two or more welding procedures. In welded highway structures, normally the only type of destructive tests required are those indicated in the AWS Specifications (1, Section 5, Appendix D).

It can readily be seen that destructive testing of any component part of a highway structure would not be feasible and, most likely, the repairs required by removal of the part from the structure would be detrimental and more damaging than any defects found in the component.

WELDING INSPECTION IN STATE HIGHWAY DEPARTMENTS

As stated previously, changes in welding procedure and inspection methods necessitated, in November 1963, the updating of the 1960 survey of inspection procedures of each of the 50 states, Puerto Rico, and the District of Columbia. Forty-six of the departments replied to the original inquiry, whereas all 52 departments replied to and are included in the 1963 survey. A summary of the significant information in the replies to the 1963 survey is given in Table 1. In several instances, specific information is not shown in the summary table because only general information was included in the highway department's reply.

It should be noted that although the methods of inspection used by the highway departments vary widely, the number of states permitting shop and/or field welding is increasing rapidly. In 1960, only 78 percent of the highway departments replying indicated use of welding as a primary fabrication method but, by 1963, 50 of the 52 highway departments, or 96 percent, were using welding as a primary fabrication method. Although not listed in the summary table, the U. S. Bureau of Public Roads permits welding on Interstate highway projects and, on occasion, has encouraged the states to use welding in cooperative projects. The Bureau specifies on welded bridges in which it cooperates that the state require thorough visual inspection and magnetic particle inspection of 1 ft in every 10 ft of fillet welds (ASTM Specification E 109). In addition, it specifies for field welds radiographic inspection of 100 percent of all tension and compression splices, including splices subjected to stress reversal, and for shop welds radiographic inspection of 100 percent of all tension splices subjected to reversals of stress and 25 percent of each compression and shear splice. If 10 percent of the 25 percent of the latter splices are defective, then the remaining 75 percent must be radiographed.

Table 1 indicates that the majority of states permitting welding require radiographic inspection in addition to visual inspection of the tension butt welds. However, for compression butt welds, the use of radiographic inspection is not so extensive. For fillet welds, magnetic particle inspection is used to a large extent because of its immediate response and low cost.

It should be pointed out that each state follows the present AWS Specifications (1), which require only visual inspection of the welds with provisions included for radiographic, magnetic particle, dye penetrant or ultrasonic inspection. The current specifications do, however, encourage the use of radiographic inspection, especially for groove welds carrying primary tensile stress. In addition, in the AWS specifications there are standard qualification tests for welds and for welders. It should be noted that before welded structures of steel other than ASTM A 373, A 36 or A 441 structural steel can be designed on the basis of the AWS specifications, modifications both in inspection and welder qualification must be made in the specification requirements.

An analysis of the replies from the 46 highway departments to the 1960 inquiry shows that 36 of the groups were using welding as a method of initial fabrication or were, at the time of their reply, planning a welded highway structure. Of these, 24 (67 percent) indicated they were requiring some radiographic inspection. The remaining states either used a comprehensive program of visual inspection or the specific types of inspection were not designated. In several instances, magnetic particle inspection techniques were specified for fillet welds. However, none of the departments

TABLE 1

SUMMARY OF INSPECTION METHODS OF STATE HIGHWAY DEPARTMENTS

State	Date of Information	Welding Fabrication			Inspector[a]	Inspection Specified[b]				Remarks
		Shop or Field	Repair Only	None		Tension Butt Welds	Comp. Butt Welds	Web Splice Welds	Fillet Welds	
Ala.	11/29/63		x							Weld reinforcing bars only.
Alaska	12/2/63	x[c]			S	R[1]	25% R	25% R	10% MP	[1]All splices subjected to stress reversal, but not more than one-third of each splice beginning at point of maximum tension.
Ariz.	11/21/63	x			S	R				R additional welds as specified on plans.
Ark.	12/16/63	x			S	R	R[2]	25% R	10% MP	[2]Shop welds—25% R.
Calif.	12/9/63	x			S	R[3]	R[3]	R[3]	MP or DP[4]	[3]R 80% of all primary stressed shop welds, 100% of all primary stressed field welds; [4]MP used occasionally, DP used most often.
Colo.	11/20/63	x			S	R	25% R	25% R[5]	10% DP	[5]Of tension side.
Conn.	11/22/63	x			F	R or MP[6]				[6]As specified on drawing.
Del.	11/21/63	x			S[7]	R	R[8]	R[8]	10% MP[9]	[7]Commercial field inspection; [8]tension only if span less than 100 ft; [9]tension only.
Fla.	11/21/63	x			F	R	R	R	MP	
Ga.	11/19/63	x			S	R	R	R		
Hawaii	11/21/63	x			S	R	R	R	MP	Currently designing two steel bridges to determine cost of steel structures.
Idaho	11/26/63	x			S	R	R	R	MP	
Ill.	11/18/63	x[c]			S	R	R	R		
Ind.	11/20/63	x			S	R	R	R	MP	Built-up girders shop welded; secondary members may be field welded.
Iowa	12/9/63	x			S	R	R			
Kan.	11/26/63	x			S	R[10]	R[10]	R[10]	MP[10]	[10]When required by specifications or when extra shop splices permitted on long girders.
Ky.	11/26/63	x			S	R	R[11]	R[11]	MP	[11]Not to exceed 25% of weld length.
La.	11/19/63	x			S	R	R	33% R[12]	10% MP	[12]Web tension side.
Me.	11/21/63	x[c]			S	R	R	R	MP	
Md.	12/4/63	x			F	R	R[13]	R[14]		[13]On plate girders, not on rolled beams; [14]when butt-welded.
Mass.	11/26/63	x[15]			S[16]	R	R	R	MP	[15]Welding of primary stressed members limited to shop; [16]state inspector present.
Mich.	11/29/63	x			S	R	R[17]	R[17]		[17]Portion only.
Minn.	12/2/63	x			S	R	R	R[18]	2% R[19] 15% MP	[18]Tension side and top 14 in. of compression side (vertical); sonic and R spot check; [19]fillet welds of T-1 are 100% R.

Neb.	11/19/63	x		S	R[21]	R[21]	R[22]	MP[22]	[21]In shop; [22]only when inspected by independent laboratory.
Nev.	11/26/63	x[c]		S[23]	R	25% R	33% R	10% MP	[23]Commercial inspectors.
N. H.	1/2/64	x[c]		S	R	R	33% R[24]	10% MP	[24]Nearest tension flange.
N. J.	12/10/63	x		S	R	25% R[25]	25% R	10% MP	[25]All field splices and splices subjected to stress reversals—100% R.
N. M.	11/21/63	x		F	R			10% MP or 10% DP	
N. Y.	12/6/63	x		S[26]	R	R[27]	33% R[28]		[26]R by commercial inspectors; [27]all field welds—shop welds only when sufficient objectionable tension shop welds found; [28]all field splices—100% R.
N. C.	11/21/63	x		S[29]	R				[29]Commercial shop inspection.
N. D.	11/20/63	x[c]		S	R	R[30]	R[30]	MP	[30]Portion only.
Ohio	11/20/63	x		S	R	R	R[31]		[31]1 ft from each end.
Okla.	12/27/63	x		S	R[32]	25% R[33]	25% R	10% MP	[32]All splices subjected to stress reversal but not more than one-third of each splice beginning at point of maximum tension; [33]if field welded—100% R.
Ore.	11/26/63	x		F[34]	R	50% R	R[35]		[34]State interprets; [35]17-in. section.
Pa.	12/5/63	x		S	R		R[36]	50% MP	[36]Tension area.
R. I.	12/3/63	x		S	R	50% R	33% R[37]	10% MP	[37]Portion close to tension flange.
S. C.	11/22/63	x		S	R	25% R	R[38]	10% MP	[38]Lower 12 in.
S. D.	11/20/63	x		F	R	R	R	MP[39]	[39]May be required if welds not R.
Tenn.	11/29/63	x		S	R[40]	25% R[41]	25% R	10% MP	[40]All splices subjected to stress reversal but not more than one-third of each splice beginning at point of maximum tension; [41]if field welded—100% R.
Tex.	1/9/64	x		S	R[42]	R[43]	R[44]		[42]A36: 15-30% at random; A441—all shop welded—100% R; [43]A36: 15-30% at random; A441—all shop welded—25% R; [44]25% minimum of field splices of all new welders of all steels.
Utah	11/26/63	x		S	R[45]	R[45]	R[45]	MP	[45]As noted on plans.
Vt.	11/29/63	x[c]		S	R	R	R	MP	
Va.	11/30/63	x		S	R	R	R	MP	
Wash.	11/26/63	x		S	R	MP	MP	MP	
W. Va.	11/19/63	x[c]		S	R	25% R	33% R	MP	
Wis.	11/20/63	x		S	R	R			
Wyo.	11/21/63	x		S	R	R	33% R	10% MP	
P. R.	12/11/63		x						
D. C.	12/24/63	x		S	R	R	50% R[46]	10% MP	[46]25% adjacent to each flange on short spans.

[a]F = fabricator; S = state highway department or independent laboratory.
[b]R = radiography; MP = magnetic particle; DP = dye penetrant; unless otherwise stated, visual inspection also used.
[c]Shop only.

indicated the use of dye penetrant or ultrasonics as a required, or even alternate, inspection procedure.

A summary of the inspection methods and requirements reported for each of a variety of the welds found in welded highway structures is given in Table 2. The principal welds considered are tension butt, compression butt, web splice, and fillet. The table indicates clearly that as the importance of the weld in the overall safety of the structure or component member increases, the inspection requirements become more severe. In the case of tension butt welds, almost 100 percent of the departments now using field or shop welding require thorough examination (radiography) of the interior quality of the weld. In the case of the less critical fillet welds, 66 percent require magnetic particle inspection and only one required any radiographic inspection (1 percent of the length).

In addition to the substantial increase in number of departments using welding as a primary fabrication method, three major changes appear to have occurred in the last few years in the requirements for welding by the highway departments (Table 1):

1. The percentage of departments either making their own nondestructive weld inspection or having it conducted by an independent laboratory under their supervision has doubled, from 44 to 88 percent. Although in the 1960 survey, 28 percent of the replies did not indicate the agency responsible for inspection, there appears to be a change in policy from making the fabricator responsible for the nondestructive testing to placing the responsibility with the state or its agent.

TABLE 2

SUMMARY OF INSPECTION METHODS FOR DIFFERENT WELDED JOINTS

Question	1960 Survey		1963 Survey	
	No. Dept.	Percent[a]	No. Dept.	Percent[a]
Replying	46	88[b]	52	100[b]
Using welding as primary fabrication method	36	78[c]	50	96[c]
Inspection by				
Department	16	44	44	88
Fabricator	10	28	6	12
Not indicated	10	28	—	—
Permit only shop welding of primary stressed welds	—	—	12	24
Requirements[d]				
Tension butt welds				
Radiograph > 50% of length	23	64	49	98
Radiograph ≤ 50%	1	3	1	2
Compression butt welds				
Radiograph > 50% of length	12	33	29	58
Radiograph ≤ 50%	5	14	15	30
Magnetic particle	—	—	1	2
Web splice welds				
Radiograph > 50% of length	5	14	22	44
Radiograph ≤ 50%	8	22	18	36
Magnetic particle	—	—	1	2
Fillet welds				
Radiograph	1	3	1	2
Magnetic particle >50% of length	4	11	16	32
Magnetic particle ≤50%	0	—	17	34
Dye penetrant	0	—	3	6

[a]Based on number of departments using welding as primary fabrication method, unless otherwise indicated.
[b]Based on number of requests sent to highway departments.
[c]Based on number of departments replying to requests.
[d]For shop welding.

2. For the various butt welds, there is considerably more emphasis on radiographic inspection. In all three categories of butt welds, the number of states requiring radiographic inspection has increased significantly.

3. The importance of some type of nondestructive testing for the fillet welds has been realized as a means of checking for surface defects and as an incentive source for the welders. Over 70 percent of the departments now require some type of nondestructive testing, in addition to visual inspection, for fillet welds.

From the analysis of the replies from the 1960 and 1963 surveys, it appears that the policies of the various highway departments are approaching more closely, at least as a minimum, the current requirements of the U.S. Bureau of Public Roads for Interstate bridges. This trend is probably to be expected, since at the present time a large percentage of major bridge construction is connected with the Interstate system.

SUGGESTIONS FOR DEVELOPMENT OF WELDING INSPECTION SPECIFICATIONS

The development of a welding inspection specification requires the consideration of a number of integral factors. These factors include not only the type of inspection procedures to be used with each different connection detail, but also the weld quality standards desired, the inspection capabilities of the inspecting agencies, the integrity of the welders, and the quantity of welding to be used in fabrication.

A number of general inspection standards are currently available to serve as a guide in the development of a standard for a particular agency. Since the AWS specification (1) is used most widely and covers most of the basic areas to be considered when preparing either an entirely new standard or a supplement to an existing specification, it will be used as a basis for this discussion.

Basic Areas of Consideration

In the development of the inspection specification for welded highway structures, the basic areas of inspection and quality control to be considered are inspection of materials, welding procedure qualification, welder and welding operator qualifications, inspection procedures for various weldments, and quality control requirements for weldments.

The requirements for inspection of materials usually need only to specify that the materials used conform to the specifications. The basic specifications generally indicate the tolerances which are to be allowed and the necessity for cleanliness of the material.

The qualification tests for the welding procedure and the welder and welding operator are essential. The purpose of the procedure qualification tests is to indicate, through a series of static tests, whether the electrode, welding position, heat treatment, speed of electrode travel, etc., specified can be used to fabricate a sound weldment. In the case of the welder qualification tests, the purpose is to insure, through a standard series of static tests, that the welder is capable of producing a satisfactory weld using a qualified welding procedure. The limits of acceptability for the procedure and welder qualification tests are generally those outlined in the current AWS bridge specifications (1). It should be pointed out that the limits specified by AWS are for ordinary structural steels and the high-strength low-alloy structural steels; if higher strength steels are to be used, some change in the requirements will be necessary.

The selection of the inspection procedure for a given weldment is, most often, the result of the experience and judgment of the specification writer. In the selection, he must consider, for example, the type of stress to which the weldment will be subjected, the effect of a partial or complete failure of the weldment on the overall behavior of the structure, the accessibility of the weldment, the materials being joined, and the welding procedures and welders to be used. Although some considerations are more important than others, it is necessary to look at all factors simultaneously.

The selection of quality control requirements is also a result of the judgment of the specification writer. Although a number of tests have been conducted to obtain the critical factors in the determination of weld strengths and the acceptable limits of these factors, a number of different opinions still exist in this area. All known specifications do, however, reject welds with visible cracks, undersize welds, undercutting and overlap. These defects may readily be detected by visual, dye penetrant, or magnetic particle inspection. It is the limits for porosity and fusion defects in which variations for size and frequency of occurrence occur. Since porosity and fusion defects are internal defects, they are normally detected only by radiographic or ultrasonic inspection techniques. The limits for radiography are usually those set by the AWS specifications (1, Appendix E), ASTM Specification E 94-62T, and the ASME Boiler Code for Unfired Pressure Vessels. As the use of ultrasonic inspection techniques in welded highway structures increases, a set of standards similar to those now available for radiography will probably be developed.

Inspection Methods for Welded Joints

Requirements insuring adequate visual inspection for all welds must be included in the specification. Many defects in weldments may be detected before the use of a more thorough inspection technique, and, in many cases where the members are of a secondary nature, the requirement of visual inspection is sufficient.

In welded highway structures, the types of weldments which may require additional inspection, beyond visual inspection, may be divided into the following general categories: (a) tension butt welds, (b) compression butt welds, (c) web splice welds, (d) major fillet welds, and (e) secondary fillet and butt welds. In the determination of the type and amount of inspection to be required, consideration should be given to the type of stress to which the weld will be subjected and the seriousness of a failure of the weld. For tension butt welds, it is, therefore, recommended, and almost universally accepted, that complete radiography of these welds be required. However, in the case of the compression butt welds and web splice welds, the requirement of 100 percent radiography is probably not necessary. If experience shows that acceptable welds normally will be obtained, then a requirement of a reduced percentage (generally about 25 percent) of weld radiography is generally sufficient if the option is included to radiograph all of the remaining welds in the event defects are observed.

Major fillet welds would include mainly flange-web welds and cover plate attachment welds. Although internal defects in these welds are undesirable, they are generally not sufficiently detrimental to require internal inspection. A number of states specify magnetic particle inspection for these welds. However, since in nearly all applications this inspection technique can only detect surface defects, a thorough visual inspection will yield almost the same results. Nevertheless, one benefit of magnetic particle inspection is its psychological effect on the welder and the resultant improvement in his performance. For this reason it is believed desirable to include limited requirements for magnetic particle inspection of major fillet welds in any inspection specification for major structures.

For secondary fillet and butt welds, a thorough visual inspection program should be adequate. Such inspection should locate all cracks and external geometrical defects serious enough to be of concern.

A thorough examination of available test data, a review of current practices of highway departments and a study of the available inspection techniques indicate that the following general requirements should be included in the development of any inspection specifications for welded highway structures:

1. For all welds, a thorough visual inspection should be made on completion for cracks, undercut, overlap, and incorrect size. If possible, visual inspection of major groove welds should be conducted after each pass during fabrication. Dye penetrant inspection may be used as a supplement, provided all penetrant and developer is completely removed from unfinished welds before any additional welding on the joint is initiated.

2. For major groove welds, radiographic inspection should be conducted on 100 percent of all primary stressed tension butt welds and 25 percent of all primary stressed compression butt welds and web splice welds in girders. However, for

compression butt welds and web splice welds, a requirement should be included that the remaining weld length be radiographed if more than 10 percent of initial radiographs show welds that should be rejected. The locations of the initial radiographs should be selected at random. Quality control requirements should be selected from one of the recommended standards previously listed.

3. For major fillet welds and secondary welds, no additional inspection beyond visual inspection should be necessary under normal conditions. However, if the fabricator does not have sufficient experience in welding, it may be desirable to require that 10 percent of all major fillet welds be inspected using the magnetic particle method. Although there is some doubt about the capability of magnetic particle inspection to indicate more information than a thorough visual examination on defects of fillet welds, the use of another inspection technique and the possibility of every weld being examined generally insures the integrity of the welder.

ACKNOWLEDGMENTS

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REFERENCES

1. Specifications for Welded Highway and Railway Bridges. Amer. Welding Soc., New York, D2. 0-63, 1963.
2. Inspection Handbook for Manual Metal Arc Welding (Emergency Standard). Amer. Welding Soc., 1945.
3. Welding Handbook—Part 1. 5th ed. Amer. Welding Soc., 1963.
4. McMaster, R. C. Nondestructive Testing Handbook—Vols. I and II. New York, Ronald Press, 1959.
5. McGonnagle, W. J. Nondestructive Testing. New York, McGraw-Hill, 1961.
6. Nondestructive Testing. Prod. Eng., Mid-Oct. 1956.
7. Gilbert, L., and Bunn, W. B. Inspection Techniques for Quality Welding. Welding Jour., pp. 614-619, July 1953.
8. Lincoln, R. B. Sampling of Welds by Trepanning and Allied Methods. Welding Jour., pp. 809-811, Oct. 1948.
9. Kruger, R. J., Wink, S. A., and McMaster, R. C. The Present Status of Nondestructive Test Methods of Welded Joints in Ship Structures. Ship Structures Comm. Rept. 72, Oct. 1953.
10. Dick, C. M. Industrial Radiography by Gamma-Ray Projection From Radioactive Isotopes. Welding Jour., pp. 457-562, May 1957.
11. Betz, C. E. Principles of Penetrants. Chicago, Magnaflux Corp., 1963.
12. Doane, F. B., and Betz, C. E. Principles of Magnaflux. 3rd ed. Chicago, Magnaflux Corp., 1951.
13. Rankin, A. C. Practical Butt Weld Inspection. Welding (London), Vol. 18, p. 243, 1950.
14. Ultrasonic Testing. Welding Eng., p. 40, June 1960.

Fabrication of Orthotropic Deck Sections for Port Mann Bridge

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Though in use in many parts of Europe for some years, orthotropic bridge decking is comparatively new on this continent. High labor cost as compared to European wage scales has inhibited its use in North America. Production line methods and equipment have been able to achieve efficiency and economy in this field, bringing orthotropic deck construction into competition in long-span bridge design and construction.

•THE MAIN span of the Port Mann Bridge was fabricated in the Vancouver branch of the Dominion Bridge Company. The Vancouver plant is situated on a 50-acre site with 7 acres under roof. About 600 people are employed in this branch at present and the services that they supply are fully integrated from engineering to erection. The fabricating plant is divided into three main areas of operation—structural shop, plate shop and machine shop. The shops are well supplied with the machines and handling equipment needed to produce heavy steel structures and vessels. A 92-ton bridge member and a 135-ton vessel are the two largest single pieces produced in the shop to date.

The main span of the Port Mann Bridge was constructed with six basic structural members: pier members, arch ribs, stiffening girders, deck sections, verticals and lateral bracing members. Fabrication problems were encountered with all members. Devices and techniques, new to the Dominion Bridge Company and to this continent, had to be developed to produce the orthotropic deck. This task might appear formidable, but the work was accomplished with standard machines and equipment. The design of the jigs and fixtures required to semimass-produce 83 deck sections was the main contribution to the fabricating techniques. Since all members other than the deck sections went through relatively familiar steel fabricating practices, this paper describes deck fabrication only.

The orthotropic deck was wholly fabricated and spliced in the plate shop. Some vessel work was done in the shop while the deck was being processed; however, most of the shop was devoted exclusively to deck fabrication. The plate shop is 80 ft wide and 620 ft long; 160 ft at the input end has a low roof and the rest of the shop has a high roof. The low roof area is serviced by four 6-ton overhead hoists on a Cleveland interlocking bridge system. The high roof area is serviced by two 60-ton cranes fabricated at the plant. These cranes have 10-ton auxiliaries with a 40-ft clearance under the main hook. Tracks are installed through the shop for rail shipments and road access is available for trucks to enter the shop.

GENERAL DESCRIPTION OF A DECK SECTION

The bridge deck consists of a series of deck sections spliced end to end across the length of the bridge. The width of the deck sections is constant at 65 ft, and the length of each deck section varies according to geometry requirements; however, most of the sections were approximately 25 ft long.

An average deck section (Fig. 1) consisted of four floor beams, 31 stringers and a 65- by 25-ft deck plate. These components were continuously welded into a complete unit to form a deck section. The deck plate forms the top flange for the floor beams

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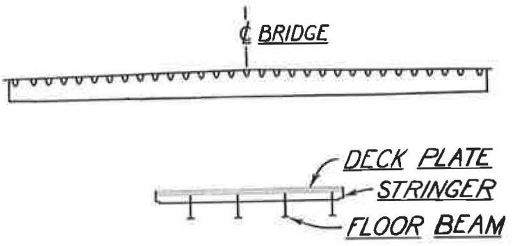

Figure 1. Deck section.

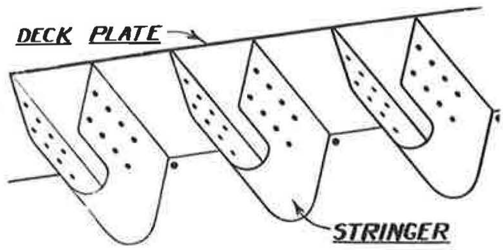

Figure 2. Floor beam web.

and for the bent plate stringers. The top of the floor beam web was cut to a profile to receive the bent plate stringers as shown in Figure 2. The top edges of each stringer were welded to the deck plate with a single outside pass of automatic welding and the top of the floor beam web was hand welded on both sides to the deck plate and the stringers. The bottom flange of the floor beam was welded on both sides to the web. The deck plates were fully butt-welded from both sides.

The deck was $\frac{7}{16}$- or $\frac{1}{2}$-in. plate, the stringers were $\frac{5}{16}$ in. thick, the floor beam web was $\frac{5}{16}$ in. thick and the floor beam flange was $\frac{5}{8}$ in. thick. The heaviest deck section produced weighed 36 tons.

MATERIAL PREPARATION

The orthotropic deck for the Port Mann Bridge was fabricated entirely from plate. The shapes necessary were made by forming in the case of the stringers, and welding and profile burning in the case of the floor beams. All material was steel shotblasted before entering the shop for fabrication. All mill scale, without exception, had to be removed from this structure before painting.

After blasting, all material was ripped to width and trimmed to length on an automatic burning machine locally referred to as the Straightograph. The machine, designed and manufactured in the Vancouver shop several years ago, consists of a bridge mounted on two parallel rails. Burning torches are mounted on the bridge. As the bridge travels down the rails it passes over plates lying between the rails and parallel edges are burned on the plate. By adjusting the angle of the torches, bevel cuts may be burned on the plate edges for weld preparations. The machine is strictly a ripping device; the same results could easily be obtained using any modern shape-cutting machine.

Some buckling was encountered when the floor beam webs were burned to width. This was minimized by mounting two heating torches on the bridge at about the third points across the plate and heating two stripes along the plate as the edges were being cut.

STRINGER PREPARATION AND SUBASSEMBLY

Thirty-one stringer plates were used in each deck section. The ripped plates from the Straightograph were moved to pit drills and were stack drilled with ordinary pit-type bogie drills. During the early stages of fabrication, full-sized splice holes were drilled at one end of the stringer plates and $\frac{1}{4}$-in. undersized splice holes at the other end of the stringer. The intention was to ream the small holes to size during the deck-to-deck splicing operation. However, it soon became apparent that it was a real problem to get "good" holes consistently. Many refinements were made in the bending techniques and welding controls in an attempt to improve the hole alignment. Eventually, it was decided to drill full-sized splice holes in one end of the stringer plate and "blow" and template ream the holes at the other end of the stringer during the deck-to-deck splicing operation. Since one set of holes had to be reamed anyway, little expense was added by the extra operation of "blowing" or "burning" the pilot holes. However, substantial savings were realized by avoiding the welding and grinding of "bad" holes before reaming.

After drilling, the stringer plate was bent into its final shape in a 750-ton hydraulic press brake. This was accomplished with one stroke of the brake. Since the stringers were to be spliced between deck sections, the profile of the stringer had to be uniform, particularly at the ends. It was also most desirable to have the two longitudinal edges of the stringer straight and parallel to facilitate subsequent welding operations. A nearly perfect stringer was consistently produced after minor die adjustments were made. Diaphragms were welded into the stringers which were then ready for the final deck assembly.

The 750-ton Dominion press had to be modified to press the stringers with one stroke. Five-foot extensions were bolted to either end of the bed and the blade, extending the capacity from 16 to 26 ft. This extension proved to be very successful.

The top or male die had two locating pins projecting vertically down on the die centerline. These pins entered two drilled holes in the stringer plate before the plate started to bend. They restrained any tendency toward differential slippage of the plate in the die and also insured that the pre-drilled splice holes would be equally spaced from the centerline on either side of the finished stringer.

The top male die consisted of a solid round bar 25 ft long welded to plates and stiffeners so that it could be mounted on the press blade. The female die was a square trough stiffened to resist the side thrust. Hardened round rods were set in the top inside edges of the trough to form the contact and wearing surfaces. The final profile of the stringer was controlled by shimming the bottom of the trough so that the blade descended to exactly the right depth (Fig. 3).

FLOOR BEAM PREPARATION AND SUBASSEMBLY

The bridge deck was crowned 7½ in. To save material and facilitate fabrication, the floor beams were fabricated from a plate girder one half the length of the finished floor beam and twice the mean height of the floor beam. The web was then split diagonally, as shown in Figure 4, and the two wide ends of the beams were spliced to form the crowned floor beam.

The basic girder was assembled from two flange plates and one web plate in a plate girder fitting and welding jig. The plates were clamped in position by air jacks at intervals along the girder. The flanges were tack-welded and then automatically welded to the web. The assembly was turned over in the jig and the other side of the joints were welded.

The web of the resulting plate girder was then laid out and stress relief holes were template drilled in the web. A burning machine mounted on a bridge that spanned the flanges of the girder was then used to cut the profile of the deck and stringers on the web of the floor beam. It is important to note that this profile was the theoretical profile with regard to stringer spacing. The machine used to burn the profile was mounted on a frame with an "overhead" steel template the same as the profile. Once the equipment was properly located on the plate girder, a following device made the profile burning automatic. After profile burning, the two half girders were then ready for the final assembly.

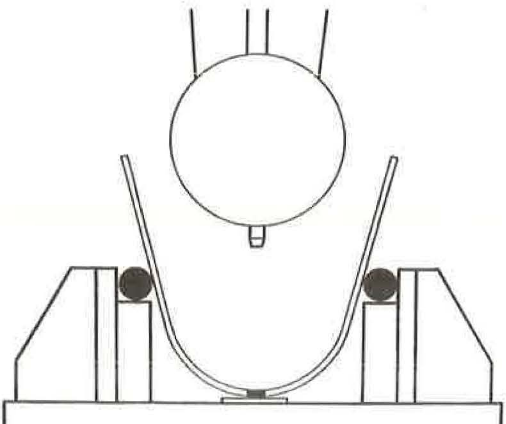

Figure 3. Stringer forming.

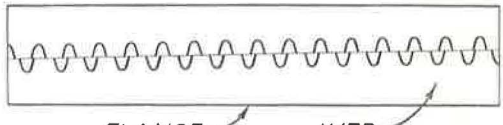

Figure 4. Floor beam preparation.

DECK PLATE AND STRINGER ASSEMBLY

A rather large fixture was constructed to facilitate butt-welding the individual plates into a whole deck plate and welding the stringers to the deck plate. The fixture was stationary and the plate and stringers passed through the fixture. This first stage of assembly was done with the deck plate on the bottom and the stringers on top.

The fixture consisted of a grillage of beams over which the deck plate could be progressively hauled as each new plate was butt-welded to the preceding deck plates. Two very rigid bridges spanned the fixture and the deck plate. On the first bridge was mounted an automatic submerged arc welding machine which welded one side of the butt joint between the individual deck plates. About 15 ft farther down the fixture, another bridge spanned the deck plate. This bridge served a dual purpose. It carried an automatic submerged arc welding machine on either side and located and clamped the stringer to the deck plate during welding.

The deck plate welding machine was mounted directly over a large water-cooled copper backing bar. An air hose ran full length under the bar. When the air was put into it, the bar lifted the deck plates against the bridge and clamped them in position until they were welded.

The same type of air hose clamping device was used on the second bridge to clamp the stringers to the deck plate during welding. In this case, the stringer was forced, as shown in Figure 5, into a series of profile plates in line. These plates were hinge mounted to the underside of the bridge so that they could be flipped up out of the way to permit the assembly to be hauled through the fixture.

In the initial planning it was anticipated that some reverse camber in the stringer would be necessary at this point of fabrication to compensate for the eccentricity of the weld relative to the neutral axis of the stringer. However, in practice, the distortion due to welding was negligible and no attempt was made to camber at this point.

The weld applied between the stringer and the deck plate was a single fillet weld. Farther down the assembly line, the fillet weld was returned inside the end of the stringer to the first diaphragm and across the first diaphragm, making the fillet continuous around the exposed side of the joint.

This stage of fabrication of the deck was by far the most critical. If the stringers were located perfectly on the deck plate after welding, the subsequent operations became much easier.

The stringer-to-deck plate welding reduced the narrow dimension of 25 ft by approximately $\frac{1}{4}$ in. and the spacing between each stringer by about $\frac{3}{64}$ in. It was necessary to experiment for some time to establish the shrinkages and great care had to be taken in locating the stringer on the plate before welding.

As the assembly was passing through the fixture, it was essential to keep the centerline of the assembly tracking the centerline of the fixture. The plate was guided by stops on one side of the fixture. Since the overall shrinkage was $\frac{1}{4}$ in., to keep the centerline tracking properly, it was necessary to move the stops past the welding station $\frac{1}{8}$ in. closer to the fixture centerline than the stops ahead of the welding station.

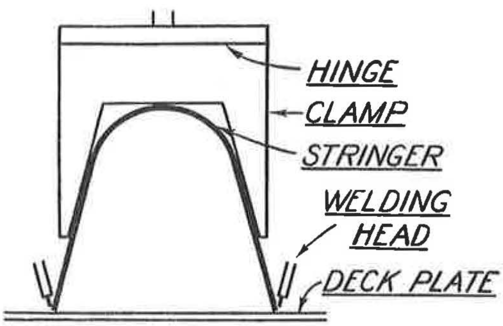

Figure 5. Stringer welding.

The fixture originally was designed with end stops to locate each stringer automatically. It was found to be almost impossible to locate the stringers accurately with the equipment as fabricated. Some time was lost attempting to get better results mechanically. Eventually the location of each stringer was laid out on the deck plate before assembly and welding. This proved quite satisfactory but did not give perfect results. Misalignment of stringers between deck sections continued to occur throughout the job. Filler plates were added where necessary when the splicing operation was being done. However, in considering the size of the weldments and a condition where 31 bent plate stringers on one deck section were being lined up with 31 stringers on another deck section, the results after the initial run-in period were excellent.

FLOOR BEAM ASSEMBLY TO DECK AND STRINGERS

The deck plate was then moved to another large fixture where the floor beams were added to the assembly. The deck of this fixture was shaped to the crown of the bridge. Vertical posts were mounted at each end of the fixture to receive the webs of the floor beams. The posts positively located the floor beams relative to each other. This, of course, was important so that the beams would match the floor beam brackets on the stiffening girders.

As was previously mentioned, the contoured profile of the floor beam web was cut to the theoretical profile. At this point in the assembly, the deck plate and stringers were fitted to the floor beam. Fitting was assisted by jacking the floor beam against the deck plate which was in turn being pushed against the fixture shaped to the crown of the bridge. Very consistent deck splices resulted from forcing the deck plate to assume the theoretical profile of the crown.

When the floor beams were first applied to the stringers and deck plates, the latter always appeared to be short. The floor beams were fitted progressively from the center of the deck to the outside; each stringer was fitted and tacked to the floor beam. This operation had the effect of "tightening" the deck plate. Since the stringers were being fitted to the theoretical profile, this operation was the test of the accuracy with which the stringers were located on the previous operation.

A beam or bridge was set above the floor beams running parallel to the stringers, as shown in Figure 6. By jacking between the beam and the flange of the floor beam the floor beam web was forced tightly against the stringers and deck plate. It was then tacked in that position and the jacking beam was moved on to the next fitting position. If a stringer were so far out of position, over $\frac{1}{8}$ in., that it was binding on the floor beam web profile, some of the web was burned away to permit entry of the stringer. Where the joint was open too much, a small backing plate was added to assist the welding at the next operation. The two halves of the floor beams were spliced in the center at this station and were welded at the next operation.

FLOOR BEAM WELDING TO DECK ASSEMBLY

At this stage the complete deck panel was assembled; however, the floor beams had to be welded to the deck plate and stringers, the floor beam halves had to be welded together and the deck butt welds had to be welded on the top side.

The assembly was then moved to another station where it was clamped, on the deck side of the section, to two beams, as shown in Figure 7. The ends of the beams were curved through 90 deg and projected about 3 ft beyond this curve. A series of holes was drilled in the curved portion of the beams. Two other beams directly below the curved beams had pintles projecting upward from the top flange. The position of the pintles matched the holes in the curved beam. The whole arrangement

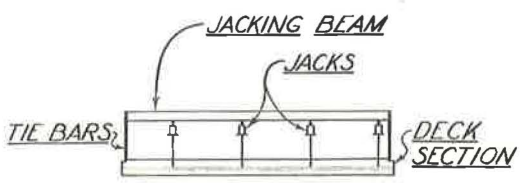

Figure 6. Floor beam fitting.

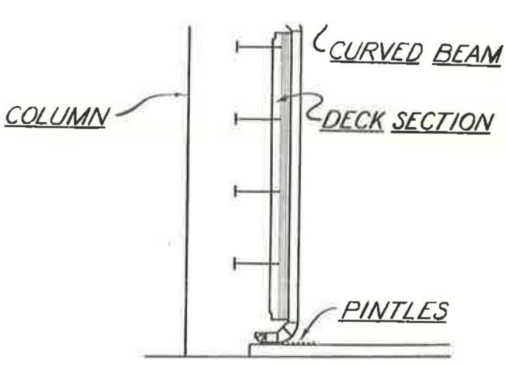

Figure 7. Floor beam welding.

was used as a device to support the deck section on edge for position welding of the floor beams to the deck and stringers.

The crane was connected to one end of both curved beams through an "evener" beam. The crane hook lifted the evener beam which in turn lifted the one end of the curved beams. The pintles below entered the holes in the curved beam and prevented any slippage of the assembly during raising. When the deck section was vertically on edge the upper ends of the curved beams were connected to the side of the building to hold the assembly in position. When the welding was completed on the upper side of the floor beams, the whole assembly was turned over by the same process and the floor beam welding was completed. The deck section was then put into its normal position, with the deck plate on top, and it remained in this position in all subsequent work.

The deck plate welds were completed with automatic submerged arc welding and spot X-rays were taken of the deck seams. The deck section was then ready for drilling and splicing.

POSITIONING AND SPLICING OF DECK SECTIONS

The complete bridge deck was spliced in the shop before field erection. The fabrication of the orthotropic deck was started early so that the shop splicing started at one end of the bridge and progressed to the center section and then from the other end to the center without interruption.

Since the bridge construction started over the piers and worked both ways toward the approaches and the center of the bridge, it was essential that the first few deck sections erected be oriented correctly over the piers. To insure correct fit and alignment on these sections, four stiffening girders and five deck sections immediately over the north pier were completely assembled in the shop. All connections were pinned and partially bolted and the assembly was drilled and reamed in position. The assembly proved to be so satisfactory that two girders and three panels were assembled for the south pier.

All of the deck splicing was carried out at one position in the shop. The deck sections were supported on large beams below each end of the floor beams. The supporting beams were high enough to allow head room for the men making the stringer splices. The supporting beams were long enough to accommodate two deck sections.

In the splicing operation, two deck sections were positioned end to end as they would be on the completed structure. The centerline of the center stringer on each deck section was marked at each end of the stringer, and then the stringer centerline was transferred to the top of the deck. The centerlines of the two deck sections were then carefully aligned. The two sections were squared up until the diagonals of the assembly were less than $\frac{1}{8}$ in. different in length.

Steel drilling templates with hardened steel bushings were then applied to the deck and carefully aligned to the layout. The holes for the deck to stiffening girder connections were drilled undersized for field reaming.

Most of the deck-to-deck holes, both in the stringers and the deck plate, were put in full size. However, control splices on the deck plate were positioned at about every 150 ft, and these holes were shop drilled undersized for field reaming.

After the deck plate holes had been located, the positions of the holes in the ends of the floor beams were established from them. These holes were also template drilled in this position. An attempt had been made at the beginning of the job to drill these holes in the fixture where the floor beam was fitted to the deck plate and stringers. It was found easier to control the relationships of the floor beam holes to the deck holes rather than the reverse process.

All 31 stringer splices were made at this station. Any misalignment of the stringers was adjusted with filler plates. The blank-ended stringers had holes blown into them and were then template reamed. The splice material was, as a consequence, identical.

The drills used on the deck were mounted on wheels. One was a radial arm type and the other was a universal type. The difference was not significant. However, to get good production speed, it was important to use fairly powerful, rugged drills.

CLEANING AND PAINTING

In the early stages of the deck fabrication, all welds were neutralized before priming. This process was not only time consuming and costly, but satisfactory results were difficult to obtain. Eventually sandblasting was substituted for passivation with excellent results. This work required 6 man-hours per deck section instead of the 32 man-hours previously necessary, and the prime coat of paint was excellent on the blasted surfaces.

After blasting the welds, the deck sections were primed with one coat of red lead iron oxide alkyd oil type primer and were stored in the yard. Just before shipping to the field, the deck sections were brought back into the shop. The faying surfaces at the splices were cleaned or blasted and clean splice material was ship bolted to the deck. A final paint inspection and touch-up was also carried out at this time.

CONCLUSIONS

A substantial number of man-hours were lost throughout the deck fabrication, mainly because of inaccuracies in the location of the stringers. Man-hours were also lost attempting to pre-drill the stringers completely before bending.

In retrospect, it is apparent that a large fixture should have been made to locate the stringers accurately at each end and where they passed through a floor beam. This could have been done based on a calculated or experimentally determined shrinkage. Regardless of the amount of final shrinkage, the location of each line of stringers would have been consistent between deck sections and between any two stringers. In such a fixture, the stringers would be tack-welded to the deck plate in a separate operation.

It would also seem to be desirable to drill the stringers at least after bending and preferably after the deck section is completely welded. All holes on any single panel that connected to the supporting steel and all holes that in any way influence the longitudinal bridge alignment should be drilled after all fabrication and welding has been completed.

Since welding makes up a large portion of the work involved in fabricating a deck section, fully automatic welding or at least semi-automatic welding should be used.

Apart from using good shop practice on normal operations, the single key to success in the fabrication of an orthotropic deck is accurate dimensional control at all stages of fabrication.

Survey of Steel-Formed Bridge Decks in Illinois

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ABRIDGMENT

•PERMANENT GALVANIZED metal bridge deck forms have been increasing in use since 1957. Compared to wood forming, metal forms offer advantages of safety and economy, since all installation work is from the top side and no stripping is required. Although individual jobs formed with metal bridge forms have been surveyed, little or no published information exists on the performance of the metal-formed decks in service. This paper gives the results of a survey on 21 metal-formed and 10 wood-formed adjacent bridges on the Interstate and secondary road system in Illinois.

The purpose of the investigation was to determine:

1. Whether there is a significant difference in the strength and quality of concrete decks on permanent metal forms as compared to wood forms;
2. Whether there are any variations in the condition of the bridge deck surfaces attributable to the forming method;
3. Whether the metal forms are bonded to the concrete; and
4. Whether there were any observable effects of salt, traffic, or weather on the overall appearance of the metal forms and the bridge structure.

Bridges surveyed were from 6 mo to 5 yr old, covering approximately 256,000 sq ft of metal-formed bridge deck and 106,000 sq ft of wood-formed deck. Many configurations and types of bridges, including steel and precast concrete stringers, simple and continuous spans, rectangular and skewed plans, parallel and converging stringers, composite and noncomposite beams, and epoxy, linseed oil and silicone coated decks, were surveyed.

Concrete quality studies used the Swiss hammer, and calibrated impact-rebound nondestructive instrument. Multiple readings were taken at uniformly spaced locations along the bridge decks. Pilot tests had established that for bridge slabs of normal thickness, the method was not affected by the presence of the structural framing members.

A statistical analysis of the Swiss hammer readings resulted in the following observations:

1. The concrete quality was independent of the method of forming;
2. The variations in concrete quality within a given bridge were generally larger than the variations between comparison bridges;
3. Comparison metal-formed and adjacent wood-formed bridges in each of three geographical areas showed no significant difference in average Swiss hammer values; and
4. For three bridges, comparison of metal- and wood-formed areas on the same bridge showed no statistical difference in quality of concrete at the 5 percent significance level.

A hammer, swivel mounted on the end of an extendable 24-ft telescopic aluminum pole, was used to inspect the underside of metal-formed decks for bond and honey-

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comb. The bonded sheets give a clear ring, honeycomb areas a dull thud, and unbonded areas a clatter when struck a sharp blow.

The hammer tests showed that the sheets were tightly bonded to the concrete slab on 20 of the 21 slabs. Leakage through floor drains without metal linings caused one bridge to break bond in the edge span. On the 21 bridges, only two 6- by 6-in. areas of honeycomb were found. On two bridges some 2- by 2-in. small unbonded spots, probably due to oil drips from construction equipment, were found. On bridges with a longitudinal edge construction joint, the subsequent placement of the curb caused additional form deflection in the outside formed span since breaking of the sheet bond on the previously poured side of the joint was generally noted. Penetration of moisture through the joint to the metal form and subsequent freezing may have aggravated the breakage of the sheet bond along the joint line.

Scaling and spalling of concrete decks was usually found in the gutters and on the curbs of the more frequently salted bridges near metropolitan areas. Rural structures, not on the Interstate traffic routes, suffered little or no scaling. Popouts seemed in all cases associated with underlying chert aggregate particles.

On 3 of 31 bridges, two metal and one wood formed, rusting of the top transverse reinforcing occurred. This was associated with insufficient cover over the reinforcing bars. Transverse cracks in the deck near the piers were, in many cases, initiated by curb joints.

In general, visual appearance of the metal forms was excellent with the galvanized spangle still showing. One bridge without metal-lined floor drains showed corrosion in the areas of the floor drain. This corrosion was in form of penetrating pits which drained the water and salt away from the slab.

In conclusion, the concrete quality and deck condition were not affected by the use of permanent metal forms. The forms were bonded to the concrete decks and presented pleasing underside appearances.