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METHODS FOR JOINING OF RAILS: SURVEY REPORT

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16. Abstract

The performance of track structures depends greatly on the integrity of the connections between rail sections. Because the majority of service and detected rail failures occur at joints, particularly conventional bolted joints, this survey was conducted to review existing practices, examine potential joining methods, and identify promising new methods and modifications of joining methods that can provide improved rail performance and lower fabrication cost. Methods for joining rails in the field as well as in plants by both metallurgical methods (welding and brazing processes) and nonmetallurgical methods (mechanical fastening and adhesive bonding) are reviewed. Joining procedures, inspection methods, laboratory and in-track performance, failure modes, adaptability to shop and field fabrication, personnel skills required, and costs are discussed. Joining methods that warrant additional development are identified and developmental efforts are outlined.

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PREFACE

This report presents the results of a program to review the processes and procedures used for joining of railroad rails. This report has been prepared by Battelle-Columbus Laboratories under Contract DSA 900-74-C-0616, Mod. P00008 for the Department of Transportation, Transportation Systems Center. The program was conducted under the technical direction of Mr. Roger K. Steele, at the Transportation Systems Center.

Although a major portion of the program consisted of reviewing available reports, significant contributions were made through discussions with railroad personnel, equipment suppliers, and others. The assistance of those listed below is gratefully acknowledged.

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The literature search for relevant information on rail joining covered primarily the period of 1954-1975 and comprised both manual and computer searches. Manual searches were conducted in the Battelle-Columbus Laboratories library system, principal railroad and welding industry journals, the U. S. Patent Office, American Society for Metals Metals Abstracts, Engineering Index, and Applied Science and Technology Index. Computer methods were used for the Department of Defense/National Technical Information Service collections and both manual and computer searches were made in the Metals and Ceramics Information Center, Railroad Research Information Service, and Transportation Research Information Service literature collections.

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
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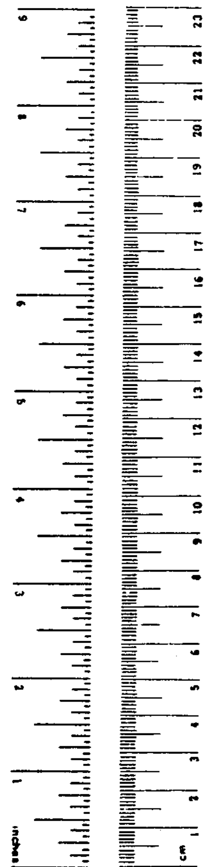
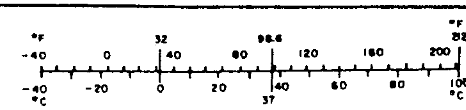
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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1. INTRODUCTION

The performance of railroad track structures depends on many factors, one of the most important of which is the integrity of rail joints. A large proportion of rail failures occurs at the connections between rail sections, particularly at conventional bolted rail joints. The cost of these failures, and the inspection and maintenance performed to reduce their incidence of occurrence, is high. For example, during the decade of 1963-1972, 1522 train accidents resulted from rail failures in the joint area or joint failures.(1)* The cost of these accidents due to damage of equipment, track, and roadbed only was $\$36.5 \times 10^6$. The total cost was much higher. In terms of inspection and repair, 180,074 service and detected rail failures accompanying the inspection of 238,000 miles of track in 1970 were reported, of which over 89,000 occurred in the web-in-joint region.(2) (There are about 206,000 miles of line-haul track in the United States.) Seventy-two percent of these joint failures were detected. Detected failures are those failures that are found by visual or instrument inspection.

Although significant reduction of bolted-joint failures has been achieved by the adoption of continuous welded rail (CWR), improved fasteners, and adhesive bonding of bolted joints, some 80 million conventionally bolted joints remain in United States' track. Much of this track is experiencing increasing yearly traffic tonnages, higher wheel loads, and higher train speeds that have increased the rate of track-structure deterioration and rail-joint failures.

Furthermore, the presently available rail welding and laying facilities limit the conversion of bolted track to CWR to about 6000 miles per year. In addition, methods for joining CWR strings at the track site frequently do not provide reliable connections due to the vagaries of in-field process control and inspection.

Therefore, this study was undertaken to review the practices used for joining rails and to examine potential joining methods. The objective was to identify promising new methods and modifications of joining methods that could provide improved rail performance and lower

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* Numbers in parentheses designate references listed in the Reference Section.

fabrication cost. Methods for connecting rails in the field as well as in plants by both metallurgical joining methods (such as arc, resistance, gas-pressure, thermite, and friction welding and brazing) and nonmetallurgical methods (mechanical fastening and adhesive bonding) are reviewed. Although much of what is presented concerning existing rail-joining methods will be well known to members of the United States railroad community, the information concerning new, potentially useful joining techniques, hopefully, will privide some new insights. In addition, this document, as a whole, should serve to help those firms and individuals who may be able to offer the industry improved appraoches to railjoining by defining more clearly some of the technical problems and economic constraints faced by the railroad industry.

2. SUMMARY

The high cost associated with rail failures and the deterioration and maintenance of track structures, particularly at bolted rail joints, strongly motivates the development of improved rail-joining methods. Furthermore, although the adoption of continuous welded rail greatly reduces these costs, this has been accomplished for only about 15 percent of all track in the United States and there remain some 80 million conventionally bolted joints. Much of this track is experiencing increasing yearly traffic tonnages, higher wheel loads, and higher train speeds that have increased the rate of track-structure deterioration and joint failures. Because it will be many years before the greater part of mainline track will be constructed of continuous welded rail and because of the difficulty in installing reliable rail connections in the field, this study was undertaken to review the practices used for joining rails and to examine potential joining methods. Mechanical fastening, adhesive bonding, welding, and brazing processes for joining rails in the field as well as in plants were considered.

The major deficiencies of the bolted rail joint, which is the oldest and most widely used connection for track structures are (1) even when they are new and tight the joint region has lower stiffness than the rail, and (2) as wear and corrosion occur so that bolt tension decreases, there is an increase in the rate of further wear and damage to the track structure. Rail-head, joint-bar, and track-bolt breakage also can be attributed to loose joints. Low joints accelerate joint deterioration by augmenting dynamic forces. Furthermore, because of the reduced stiffness at the joint, resulting rail deflection and stress development in bolt holes and stress raisers, such as drill gouges, burrs, and the rail brands, can significantly reduce the fatigue life at the joints.

To improve the performance of standard bolted joints, several approaches would need to be considered:

1) Increasing the resistance to deflection of the joint region.
   a) Shortening the time period between such maintenance activities as bolt tightening, joint lubrication,

rail-end straightening, and ballast cleaning and recompaction near joints

b) Reducing the rate of joint-bar loosening by development of improved fasteners, corrosion inhibitors, and wear-resistant surface treatments

c) Decreasing the tie spacing near joints to augment rail support that is not provided by the joint bars.

2) Reducing the stress concentrations in the vicinity of bolt holes, especially the holes nearest the rail ends.

However, the extent to which any of these could be applied in practice will depend strongly on cost considerations. These approaches, which are interrelated, are discussed in more detail in the section of this report on bolted rail joints.

Adhesive-bonded joints, which include mechanical fasteners, are relatively easy to prepare, require low capital investment, and can be fabricated readily in a shop or in the field. Both laboratory and service performance of these joints has been good. These joints are relatively high cost and this precludes their use for joining of 39-ft-long rail sections. They do appear to have economical applications for joining of continuous welded rail strings, insulated joints, frogs, switches, crossings, and turnouts. It is recommended that the performance of adhesive bonded joints continue to be evaluated and that potential applications be sought.

Although thermite welding of rails in track is attractive because of the process portability, low capital investment, and relatively short time required for welding, the service performance of these welds is considerably poorer and more variable than, for example, flash welds. Thermite weld performance can be improved and its variability can be decreased by undertaking the following efforts:

1) Determine the range of weld properties obtained when welds are made in the field by railroad personnel and when made in the laboratory.

2) Obtain an initial determination of the range over which welding parameters can be varied without seriously impairing weld properties. This would include assessment

of procedures for rail preparation, welding, and weld finishing.

3) Identify variables that are most critical to obtaining sound welds.

4) Identify and evaluate processing modifications that will reduce welding parameter variability and consistently provide good welds having properties that will give better performance in track.

5) Develop ways to automatically carry out selected critical procedures or operations that now require manual skills.

Flash welding is an excellent process for rail joining. Relatively rough and dirty joint surfaces can be accommodated in the process and welds do not have an as-cast structure. In several rail-welding-plant operations, the process is highly automated so that weld productivity is higher and the weld property variability, cost, and service failure rate are lower than for any other welding process. Because an obstacle to maximizing the production rate of flash welds and minimizing the cost is lack of adequate rail end straightness, improved methods of rail straightening are needed. This might be accomplished at the rail mill, in the welding machine or after welding. Some rail purchasers inspect rails at the steel mill before shipment to the welding plant. In order to increase productivity and to reduce the occurrence of stress concentrations, it also is recommended that the Soviet automatic shear for the in-track flash welder be evaluated and modified as needed.

In comparison with flash welding, the gas-pressure-welding process is slower, more expensive, and more susceptible to weld defects caused by surface contamination. This process may have application, however, to in-track welding and it is recommended that the development and performance of the existing in-track unit be followed.

Among the several existing arc-welding processes, a fully automated submerged-arc welding process, electroslag welding process, or a combination of these two processes offers the greatest promise of successful development of an in-track, arc-welding method. Additional efforts should be directed toward developing the equipment and welding procedures to optimize productivity, degree of process automation, and weld properties. Although an arc-welding process may not compete economically and in performance with flash welds, it may compete with the thermite welding process.

Brazing is highly susceptible to defect formation due to joint surface contamination and lack of complete surface contact without substantial upsetting. Because the joint preparation requirements are stringent, additional studies to develop brazing methods are not recommended.

Friction welding is an attractive method for joining rails because, like flash welding, the process tolerates relatively rough and dirty surfaces, the welding time is short, a cast fusion zone is not created, the amount of upsetting required is small, and excellent weld properties are obtained. Also, the monitoring of several process parameters provides a means for predicting and controlling weld quality. Limited development of this process has been accomplished and further evaluation is recommended, including the welding and testing of full-size rail sections and determining if postweld heat treating is required.

3. RAIL JOINTS AND THE ADOPTION OF CONTINUOUS WELDED RAIL

Rail joints are connections between continuous sections of rail designed to support vertical and lateral forces imposed under traffic and prevent vertical and lateral movement of the rail ends relative to each other. In conventionally bolted track, joints are designed to permit longitudinal rail movement in order to accommodate thermal expansion and contraction of the rails. Other types of joints must be capable of supporting longitudinal forces. The wear resistance, strength, toughness and resistance to deflection of the joint should approach that of the rails being connected. For train control using signal block construction, some joints also must provide electrical insulation between the rail lengths.

The service performance of rail connections depends on many interacting factors including the following:

a) The type and quality of the specific connection
b) Speeds, wheel loads, gross tonnage, and nature of traffic
c) Track curvature, superelevation, gage, line, and surface
d) Rail section and rail-steel properties
e) Design, quality, and maintenance of the track structure
f) Range and frequency of ambient temperature changes.

The development and adoption of CWR has been motivated almost entirely by the reductions of maintenance costs and rail failures and an increase in rail life in comparison with standard bolted joints. The major improvement accompanying the installation of CWR is increased stiffness at the rail joints. The principal advantages of CWR are the following:

1) Elimination of bolted-joint maintenance that includes bolt tightening, joint oiling, joint-bar and bolt replacement, rail-end hardening, rail-end buildup or

rail grinding to remove end batter, rail-end straightening to remove droop; and rail-end cropping, redrilling, and relaying[3-7]

2) Reduction of damage to ties, fasteners, ballast, subgrade, rolling stock, and freight[3-9]
3) Elimination of signal-bond installation and maintenance and improvement of track-circuit conductivity[5-7]
4) An increase of rail life (158.5 mi. of 131-136 lb. rail) from 19 to 26 years for 6 railroads on track carrying an average of 18.4×10^6 tons annually.[10] Recent estimates of the increase of life of flash-welded rail in comparison with bolted rail have ranged from 15 to 50 percent.

Because CWR costs more to install than bolted rail, it may not be economical for track that carries low tonnage or low wheel loads or for track that has such very sharp curves that rail-head wear determines the need for replacement. On the other hand, CWR has been installed on some lines that experience many joint failures due to high individual car weights of 100 tons or more even though they carry less than 1 million gross tons per year.

Although the savings accrued with the use of CWR in comparison with bolted joints are dependent on many factors, the following figures have been reported:

1) Track maintenance costs are reduced by \$198-\$1,200 per mile per year.[5]
2) Thirty to 40 percent of bolted-track maintenance is at the rail joints and 45 percent of bolted-rail renewal is required because of end batter and rail-end drooping.[5]
3) Surfacing costs were reduced by 40 percent and overall track maintenance costs by 20 percent on the French railway system (SNCF) in 1961.[11]

The reduction of rail-joint failures accompanying the installation of CWR is indicated by the reported failure statistics.[2] During 1970, for all rail sections and ages, there were 75.6 rail failures of all types per 100 track miles inspected of which 37.5 failures per 100 track miles were web-in-joint failures. Of a total of 180,074 service and detected failures covering 238,078 track miles, 89,396 failures (50 percent) were attributed to the web-in-joint region. It is significant that 64,273 joint failures (72 percent) were detected defects. These figures compare closely with the 46 bolt-hole cracks detected per 100 miles of track tested by Sperry Rail Service in 1970.(12) A total of 119,509 defects were detected in 151,741 miles of track of which 70,542 were joint defects that represented removal of over 400 miles of track. During 1973, Sperry Rail Service detected 126,000 rail defects in 185,000 miles of track of which 60 percent (75,000) were joint defects.(13) This is equivalent to 41 joint defects detected per 100 miles of track tested.

In comparison to these figures, the accumulative failure rate for flash-welded joints through 1970 was 5.8 per 100 track miles and for gas-pressure welds was 22.9 failures per 100 miles as shown in Table 1. With the available data, a comparison with thermite welds was made on the basis of failures per 100 weld years, which includes the ages of the welds. Compared with flash welds, gas-pressure welds fail three times as frequently and thermite welds fail 85 times as frequently.

During 1971, 54 percent of main-line rail defects that developed during service in Japan were at bolt holes.(14) The portion of the defects that was detected is not known. During the period of 1961-1963 on British Railways, there were three rail failures (excluding switch and crossing rails) per 100 track miles per year of which 61 percent occurred at rail ends and 30 percent were through bolt holes.(15) Failure statistics for bolted and welded rail on British Railways during the period of 1968-1972 are presented in Table 2.(16) These figures are not comparable to those in Table 1 because the failure rates are based on the number of miles in track and are not accumulated failure rates. The weld failure data include both flash and thermite welds so that the failure rates of these two processes cannot be compared. The authors noted that the increasing failure rates accompanied the introduction of higher wheel loads and train speeds in 1967.

TABLE 1. ACCUMULATED SERVICE AND DETECTED RAIL-WELD FAILURES TO DECEMBER 31, 1970[2]

	Flash-Welded Joints[a]	Gas Pressure-Welded Joints[a]	Thermite-Welded Joints[a]
Track miles	14,100[b]	6,337[b]	(c)
Track-mile-years	69,372[b]	40,050[b]	(c)
Failures	817	1,449	358
Failures/100 track miles	5.8[b]	22.9[b]	--
Failures/100 track-mile-years	1.18[b]	3.62[b]	--
Number of welds (millions)	3.82	1.72	0.035
Average weld age, years	4.92	6.32	2.78
Weld years (millions)	18.8	10.8	0.097
Failures per weld years x 10^6	4.35	13.4	368

(a) Includes new and relay rail

(b) Derived from data assuming all joints were between 39-foot-long rails

(c) Unknown.

TABLE 2. BOLTED AND WELDED RAIL JOINT FAILURES ON BRITISH RAILWAYS[10]

Year	Bolted Track, miles	Rail End Bolt Hole Failures[a]	Bolt Hole Failures per 100 Bolted Track Miles[a]	CWR Track, miles	Weld Failures[b]	Weld Failures per 100 CWR Track Miles[b]
1968	20,500	667	3.25	3900	217	5.6
1969	18,500	741	4.01	4600	195	4.2
1970	17,100	977	5.71	5300	248	4.7
1971	16,900	1030	6.09	5900	435	7.4
1972	16,500	1350	8.18	6400	434	6.8

(a) Cracked and broken

(b) Flash and thermite welded.

The number of track miles of CWR laid by year since 1933 to 1971 is given in Table 3.[17] These figures show that although flash welding has been used to produce most CWR, oxyacetylene gas-pressure welding was used to manufacture increasing amounts of CWR through 1970. Classifications of CWR laid in 1971, Table 4, show that nearly all was for main track and that more than half was flash-welded, new rail for main track The second largest category was flash-welded, second-hand rail for main track.

Although there were approximately 50,000 miles of CWR in track at the end of 1974,[18] which would require 13.5 x 10^6 bolted joints, this accounts for only about 25 percent of the 206,400 miles of line-haul track[6] and 15 percent of the 343,370 total miles of track in the United States.[19] The remaining 293,470 miles of bolted track represent about 79 x 10^6 bolted joints between 39-ft-long rail sections. The 14 railroads that have the greatest amounts of CWR (1144 to 5450 track miles as of December 31, 1975) account for 44,378 miles of the 49,527 total miles.[20] Assuming that all CWR is in line-haul track, there remain some 42.5 x 10^6 bolted joints in this track category. The largest amounts of CWR installed to date were 6179 miles in 1970 and 4434 miles in 1972. During 1975, 2592 miles of CWR were installed and during 1976, 2604 miles of CWR installation are planned.

In summary, CWR has given excellent service performance and continued installation, particularly in heavily traveled track, will improve the economics of railroad operation. Additional details on the fabrication, installation, performance, and costs of CWR are given in subsequent sections of this report.

TABLE 3. TRACK MILES OF CONTINUOUS WELDED RAIL LAID BY YEARS, 1933-1971 (17)

		Total*			Oxy-acetylene	Electric Flash	Total
1933	--	0.16	1955	--	194.50	72	266.50
1934	--	0.95	1956	--	372.33	89.10	461.43
1935	--	4.06	1957	--	390.47	159.65	550.12
1936	--	1.52	1958	--	148.11	312.13	460.24
1937	--	31.23	1959	--	378.65	691.92	1070.57
1939	--	6.04	1960	--	299.42	961.20	1260.62
1942	--	5.48	1961	--	94.13	926.50	1020.63
1943	--	6.29	1962	--	310.59	1183.34	1493.93
1944	--	12.88	1963	--	497.52	1360.48	1858.00
1945	--	4.81	1964	--	586.76	1796.74	2383.50
1946	--	3.91	1965	--	700.59	1655.74	2356.33
1947	--	18.70	1966	--	746.61	1984.71	2731.32
1948	--	29.93	1967	--	784.28	1800.27	2584.55
1949	--	33.05	1968	--	643.10	2543.61	3186.71
1950	--	50.25	1969	--	674.35	2930.01	3604.36
1951	--	37.25	1970	--	800.30	5378.32	6178.62
1952	--	40.00	1971	--	504.28	3604.72	4109.00
1953	--	80.00					
1954	--	87.00			8,579.50	27,450.44	36,029.94

* CWR mileage installed during the period of 1933-1954 included thermite-, gas-pressure-, and electric flash-welded rail.

TABLE 4. CLASSIFICATIONS OF CONTINUOUS WELDED RAIL LAID IN 1971 - TRACK MILES (17)

	Oxyacetylene		Electric Flash		
	New	Second-Hand	New	Second-Hand	Totals
Main track	270.62	213.60	2371.90	1196.68	4052.68
Sidings and yard track	0.06	20.00	--	36.14	56.20
	270.68	233.60	2371.90	1232.82	4108.88

4. CURRENT RAIL JOINING PROCESSES

In the sections that follow, methods currently used or evaluated for joining rails are described and discussed. For each method, procedures, properties, service experience, principal failure modes, process variability, personnel skills, costs, and adaptability to shop and field fabrication are included depending on the available information.

4.1 MECHANICAL FASTENING

The bolted rail joint is the oldest and most widely used connection for rail. As pointed out in the Introduction, there are on the order of 80 million bolted joints in track in the United States. It is important to examine methods for improving the performance and reducing the maintenance required at bolted joints because around 40 million of these are in United States' mainline track. In addition, it is neither feasible nor economically justifiable presently to replace bolted joints with CWR to any significant extent in main, branch, and secondary lines that carry low traffic tonnages and relatively light cars. Ultimately, when much mainline track is CWR, this CWR will follow the usual cascading sequence to more lightly traveled lines. At the present rate of CWR installation and reduction of track mileage, however, the amount of CWR in secondary lines probably will not be significant before the year 2000.[6]

Because the vast majority of rail joints, currently and for many years to come, will be mechanical joints and because the long-term structural adequacy of mechanical joints is particularly difficult to achieve, an extensive discussion of mechanical joining methods and their historical development has been included in this review. This is not to imply that, necessarily, there are any economic incentives to improve the mechanical joint except in special cases such as insulated joints and in turnouts and crossovers. However, an understanding of the need for improved rail joining processes must proceed from an appreciation of the difficulties encountered in the use of mechanical joints.

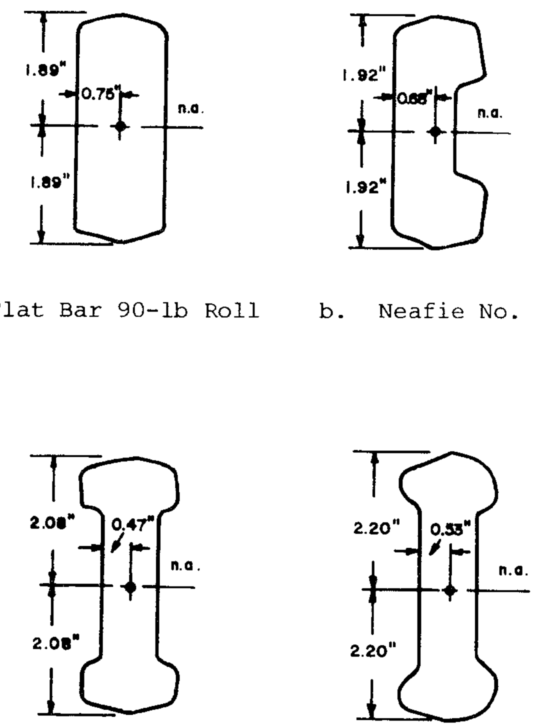
4.1.1 The Development and Action of the Bolted Rail Joint

Numerous studies have been conducted over many years to develop an improved understanding of the performance of bolted rail joints as a function of the design of joint components and service conditions. Significant findings concerning bolted rail joints from several of these investigations are summarized below.

One of the earliest reports on the design and performance of bolted rail joints was based on work done by the Special Committee on Stresses in Railroad Track that was formed in 1914 and was under the direction of the late Professor A. N. Talbot. In the 5th Progress Report,[(21)] it was stated that without the presence of joints in track, there would be uniform distribution of bearing pressure on the ties and ballast and improved uniformity in the stiffness and flexibility of track. The goal in bolted joint development has been to design joints that would approximate the conditions of the continuous rail in terms of stiffness, strength, uniformity, and flexibility, with appropriate consideration of cost and maintenance requirements. Over many years prior to 1929, various types of rail joints had been developed, which were intended to provide full or at least partial continuity. Full continuity implies that the bending moment taken by the joint bars at the rail ends is as great as that taken by a continuous rail. In addition to providing vertical bending strength and stiffness, and resistance to vertical bending moments, rail joints must also withstand lateral pressures and lateral bending.

The first joint bars used in the United States were fishplates which are flat plates connecting the rails. The friction between the plates and the web of the rails was the only source of resisting moment in the bar. Later, joint bars were developed that contacted the underside of the rail head and the top of the rail base. Both of these types of bars were symmetrical with respect to a horizontal axis through the center of gravity. Because symmetrical bars do not deflect laterally under vertical loads, bolt tension is not essential for developing vertical resisting moments in the bars when a vertical bending moment is applied to the rails. These forms of joint bars are shown in Figure 1.

Somewhat later, the angle bar was developed to allow a wider distribution of the metal, a greater depth of section, and apparently greater lateral resistance and a greater value of moment of inertia of the section about both horizontal and vertical axes. An example of this bar is shown in Figure 2. The angle bar is an unsymmetrical bar, however, so that when a bending moment is applied only in a vertical plane and



a. Flat Bar 90-lb Roll   b. Neafie No. 1 105-lb Rail

c. Neafie No. 2 118-lb Rail   d. Neafie No. 3 130-lb Rail

FIGURE 1. SECTIONS OF SYMMETRICAL BARS[21]

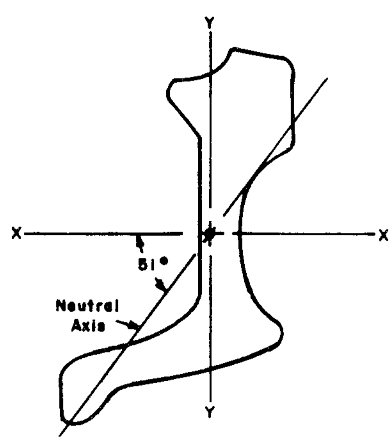


FIGURE 2. EXAMPLE OF UNSYMMETRICAL TYPE OF JOINT BAR[21]

there is no lateral restraint, the neutral axis of the bar is inclined at an angle with the vertical, and the deflection of the unsymmetrical section is at right angles to the axis, both downward and laterally toward or away from the rail.

When the rail ends are bent downward (positive bending moment), the unsymmetrical angle bar deflects at mid-length downward and toward the rail and, at its ends, bends away from the rail, because the neutral axis is inclined to the horizontal axis. The twist introduced into the angle bar is reversed upon application of a negative bending moment. Reversals from positive to negative bending moments tend to work the joint bars away from the position of close fit to the rail unless adequate bolt tension is applied to prevent it. Having an inclined neutral axis, the stresses in the angle bar are greater than those calculated for a horizontal neutral axis. The amount of inclination of the neutral axis and therefore the augmentation of stress was reduced when higher bolt tensions were used.

Talbot stated that the purpose of providing tension in bolts can be given as follows:

(a) To draw the joint bar as far into its place between the two fishing surfaces of the rail as the surface roughness and differences of dimensions of the two rails and the two bars will permit;

(b) To prevent the bar from working out of place when the bending and twisting of the bars and the rail may act to force it out;

(c) To make the bar fit more closely to the rail, and thus to decrease the vertical movement between the bars and rail produced upon application and release of load;

(d) To give lateral restraint to the angle bar type of joint bar and thus reduce as far as possible the deflection of the neutral axis from the horizontal. This lateral restraint would reduce the corresponding lateral bending and twisting of the angle bar or other unsymmetrical bars when a vertical load is applied to the joint;

(a) In the case of lateral bending, to provide a lateral resisting moment at the joint, both in the case of low bolt tension and in the case of bolt tension sufficiently high to bring into play integral action of the bars and rails.

With regard to the first purpose given above, Talbot reported that even with the passage of a single passenger train after angle bars had been installed with the bolt tension of 20,000 pounds applied, the bolt tension was found to be reduced to 13,000 pounds. The amount that the joint bars work out of place was found to be greater with angle bars than with symmetrical-type joint bars.

An important conclusion from these investigations was that a minimum average tension of 5000 pounds in each bolt using symmetrical joint bars will give sufficient integral action of the joint to support vertical loads satisfactorily and adequately resist lateral bending. Also with reasonably tight bolts, the fiber developing the highest vertical bending stress in angle bars or symmetrical joint bars will not usually be the one that develops the highest lateral bending stress, so that the maximum stress in combined vertical and lateral bending will be less than the sum of the maximum bending stresses for the two bending actions. For angle bars, a minimum bolt tension of 10,000 pounds is required to minimize the inclination of the neutral axis from the horizontal and accompanying twisting of the bars.

Talbot also stated that if the vertical stiffness of two symmetrical joint bars is one-third of that of a continuous rail, the deflection of the bars between their reaction contact points will be four times as great as that for a full rail under a uniform moment throughout the same distance. Unsymmetrical bars having an inclined neutral axis will show even greater deflection than symmetrical bars. Play between the bars and rails at the middle and near the ends of the bars may greatly increase joint deflection. Figure 3 shows the bending moment and rail depression for a single load, P, applied at a rail joint.[21,22] The ratio of the resisting moment at a section at the center of a joint to that developed in a continuous rail

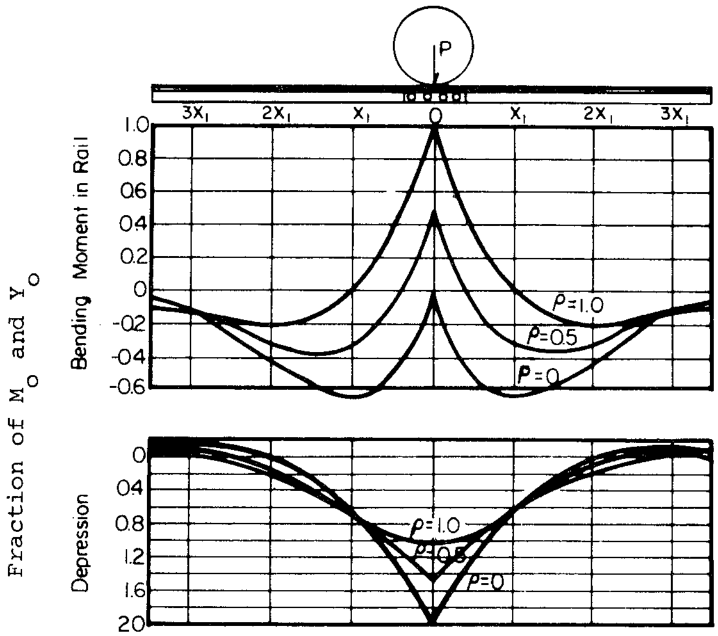


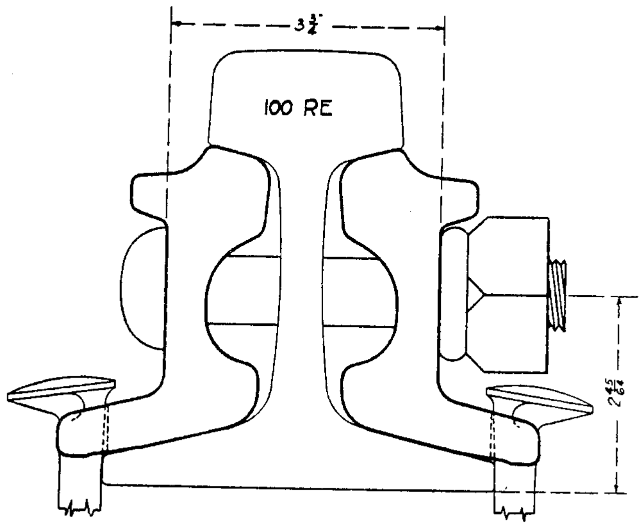
FIGURE 3. BENDING MOMENT AND RAIL DEPRESSION FOR A SINGLE WHEEL LOAD AT A JOINT(21,22)

M_o and Y_o are the maximum bending moment and depression, respectively, for a single wheel load P. X_1 is the distance from the point of load application to the point of zero bending moment in a continuous rail (ρ = 1.0) at some other location under the same conditions of loading and support is denoted ρ and can vary from 0 to 1.0. The actual value of ρ can vary widely from joint to joint and is strongly dependent on bolt tension, fit between the joint bars and rails, and ballast support. For example, in field tests of 90 lb/yd A.R.A-A rail with 24-inch-long, 4-bolt angle bars, the joint moment ratio was only 0.35 when the average bolt tension was 1375 pounds (0 to 2500 lb range). The joint moment ratio was 0.79 when the average bolt tension was 14,000 pounds (10,500 to 17,500 lb range).

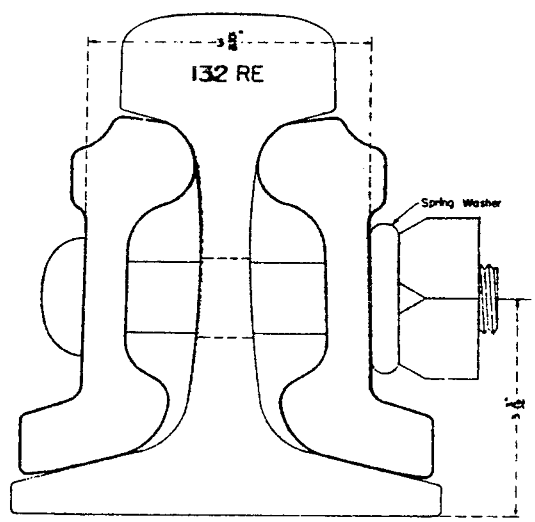
It also was shown that an angle bar joint, in comparison with a rail joint having bars of symmetrical or nearly symmetrical section, developed higher stresses and greater vertical and lateral deflection and twist, gave lower beam stiffness, required higher bolt tension for effectiveness of joint action, and possessed a greater tendency to become loose by twisting and working out of place. It is for these reasons that the symmetrical or nearly symmetrical types of joint bars have gradually replaced the angle bar form of joint bar. Typical joint bars of the head contact, long toe design (unsymmetrical) and head free, short toe type (nearly symmetrical) that are currently recommended are shown in Figure 4.[23]

In these studies, it was found that where there is lack of fit between the rail and bar so that upon application of load, the rail and bar move vertically with respect to each other, even if only on the order of 0.001 to 0.03 inch, there is a very significant effect with respect to the moments developed at the joint. A greater relative vertical movement was found to be accompanied by the development of smaller resisting bending moments in the joint bars and greater deflection of the rail joint.

The action of the rail joint in lateral bending was also found to depend on the degree of fit between the joint bars and the rail and the tightness of the bolts. If the bolts are loose, the two joint bars act individually and lateral stiffness is relatively low. For medium and high bolt tensions, the action of the joint approaches that of the rail joint acting integrally. For lateral bending moments that may be incurred in straight track and light curves, the bolt tension that is desirable to maintain for other reasons also will give adequate integrity in lateral



a. Head Contact, Long Toe Joint Bar and Assembly for 100 RE Rail



b. Headfree Short Toe Joint Bar and Assembly for 132 RE and 136 RE Rail

FIGURE 4. JOINT BARS AND ASSEMBLIES FOR 100 RE, 132 RE, AND 136 RE RAIL(23)

bending. The unit action of the two bars makes the joint stronger and much stiffer in lateral bending.

The fracture of joint bars, which frequently initiates at the top surface at the rail ends, has been associated with the vertical bending action of the bolted joint.

Under a vertical load at a joint, the joint bars and rails do not bend with each other along their length. Near the ends of the joint bars, the rail has a maximum bending (curvature) and the joint bar a minimum, while near the end of the rail the reverse is true. Therefore, under loads that bend the rail ends downward, the underside of the rail head presses on the top surface of the joint bars for some distance along the bar away from the rail end. This contact can introduce a notch in the bar that ultimately results in a fatigue failure. There is also an upward bearing pressure of the upper surface of the rail base along a length of the joint bars further away from the rail ends. Because of differences of vertical stiffness and curvature between joint bars and rails along their length, under vertical loads a space develops between the joint bars and the rail surfaces even when the bars fit tightly against the rails under no load.

In an investigation of the bending fatigue behavior of bolted joints, it was found that the use of a head-free joint bar offered greater resistance to cracking than did any of the head contact designs.[24] In these tests, the joints in 112 and 131 lb/yd rail sections were loaded to 65,000 pounds at the joint, which was centered between supports 36 inches apart. The load was applied 50 times per minute and all joints were assembled with 20,000 pounds bolt tension. These tests also showed that the best solution for the prevention of top center and base center cracks in head contact bars was the introduction of a ground easement to prevent notching of the bars by contact with the rail ends. The use of toeless bars was effective in eliminating cracking in spike slots. For both toeless and toe bars, top center easements resulted in improved resistance to top center cracking. An observation during the test on loosening of bolts was that where one-inch bolts were used with 24-inch-long bars, bolts started to loosen at about 40,000 cycles, and by the end of the test some were very loose. On the other hand, with 1-1/8-inch bolts on 24-inch-long bars, the bolts remained tight throughout the entire test. With 36-inch-long bars and using 1-inch-diameter bolts, the bolts remained tight for the duration of the test.

During the period of 1937 to 1947, service tests of various designs of joint bars for 112 and 131 lb/yd RE rail were conducted on tangent track.[25] The track sections using these rails carried 139 million gross tons and 178 million gross tons of traffic, respectively during this period. It was concluded from the evaluations that, for both rail sections, compared to 24-inch bars, the 36-inch joint bars had substantially longer service life, maintained a better track surface at the joints, and prolonged the life of the rail itself. The performance of head-free joint bars in these tests was as good as the average performance of head contact bars of comparable section and length. Among the several designs of angle bars that were included in the tests, the short toe designs generally gave equal or superior performance in comparison with long toe angle bars.

An investigation was conducted from 1938 to 1945 to determine the bolt tension necessary to support rail joints properly.[26] From the results of these studies, it was concluded that the purposes for providing tension in track bolts were:

(1) To draw the joint bars into place when they are first applied. An initial bolt tension when bars are first applied of from 20,000 to 30,000 pounds per bolt helps overcome the roughness of the fishing surfaces, thereby providing proper seating of the bars.

(2) To hold the bars in place during actual service and to produce an integral action of the two bars of a joint in resisting bending in the vertical or horizontal planes. A minimum bolt tension of 10,000 pounds per bolt for the long toe (unsymmetrical) joint bar or 5,000 pounds per bolt for the short toe (nearly symmetrical) joint bar was found sufficient to accomplish these purposes.

(3) To provide sufficient reserve tension to hold the bars over the period between tightenings. This requires that the applied tension be high enough to withstand the loss in bolt tension under traffic between the tightenings and still be sufficient at the end of the period to insure proper action of the joint bars. Bolt tension loss was relatively rapid immediately following the application of the joint bars until the mill scale had been abraded from the fishing surfaces and averaged from 5,000 to 10,000 pounds per bolt the first month. After the second month, the rate of bolt tension loss averaged from 500 to 1,000 pounds per bolt per month. Loss of tension was not uniform in each joint and some bolts lost twice the above amounts; others lost very little tension. Bolt tension loss was principally due to a decrease in distance between the two bars at the joint as a result of fishing surface wear. This decrease varied from joint to joint and averaged approximately 0.015 inch per year. Traffic density had little effect on joint bar pull-in except that on very heavy traffic density lines, the decrease of the midlength separation of the bars could average 0.025 to 0.030 inch per year. The use of spring washers helped to maintain bolt tension as the inward movement of the joint bars occurred.

(4) To provide necessary joint bar support without unduly restricting longitudinal slippage of the rail ends caused by temperature change. The slippage resistance of a rail end with its joint bars increased as expected with the amount of bolt tension.

On the basis of the results that were obtained, the following practices were recommended.

(1) The applied bolt tension should be in the range of 20,000-30,000 pounds per bolt for the initial tightening and within a range of 15,000-25,000 pounds for subsequent tightenings.

(2) Track bolts should be retightened as required, preferably from one to three months after the joint bars first are applied and at intervals of one year thereafter. More frequent tightening is unnecessary and therefore uneconomical. Less frequent tightening requires too high an applied bolt tension to carry over the longer period of service.

(3) A corrosion-resistant lubricant should be applied to the bolt threads prior to the application of the nuts. This will reduce the variation in thread friction and increase the uniformity of bolt tension.

These recommended practices remain in effect today.[9,19]

During the field tests, conducted from 1938 to 1945, two disadvantages of low-bolt tension were clearly demonstrated. The first was that when a bolt becomes loose enough in a joint to rattle, in only a few days time the threads will be so battered against the web of the joint bar and by the spring washer, if one is used, that the nut cannot be turned to retighten the joint. The second significant disadvantage of low bolt tension was excessive wear of the joint bars if all the bolts in a joint or on one end of a joint became entirely loose. In this case the inward movement of the joint bars reached as high as 0.07 to 0.10 inch. It was believed that the lack of adequate bolt tension caused this excessive fishing surface wear and resultant inward movement of the bars. It also was found in these studies that the nuts did not become loose because they backed off of the bolts. Rather, the wear between the fishing surfaces of the joint bars and the rail permitted the bars to move closer.

Another finding was that, in general, the loss of bolt tension during a one-year period was approximately 60 percent of the initial bolt tension if no spring washer of any kind was used. In tests where high reactive spring washers were used, the loss of bolt tension over a year's period was found to be from 35 to 45 percent. That these reductions in bolt tension with or without spring washers were not significantly different was attributed to the fact that the imposed elastic strains in the initially tightened bolt, the bars, and the rail web provided a degree of reactance that compensated for fishing surface wear. However, in addition to providing even greater reactance for maintaining tension, spring washers prevented bolts from rattling in the bolt holes and being damaged if they became excessively loose.

Near the conclusion of the bolt tension and joint bar service tests conducted in mainline track during the period of 1937-1947, an investigation to evaluate the reactive characteristics of spring washers in assembled rail joints was conducted to determine the capacity of different types of spring washers for sustaining bolt tension for the largest amounts of joint bar pull-in accompanying wear.[27] The 112 and 127 lb/yard rail and joint bar assemblies with the usual mill scale still intact were tested by loading in alternate tension and compression to produce longitudinal slippage. In the first cycle of opening and closing the rail joint gap, the initial 20,000-pound bolt tension dropped 33 and 47 percent in the joints having high and low reactance washers, respectively. This was caused by abrading off of the mill scale on the fishing surfaces of the joint. The rapid loss in bolt tension confirmed the previous recommendation to tighten new joints initially in tension ranging from 20,000 to 30,000 pounds. Generally, the higher the reaction of the spring washer, the greater the tendency was to retard the loss of bolt tension for a given amount of joint bar pull-in. For example, with the low reaction washers, the tension dropped to 5000 pounds at only 400 cycles and 0.036-inch pull-in. The bolt tension in another joint using high reaction washers was in excess of 8000 pounds after 2800 cycles with a joint bar pull-in of 0.036 inch. Furthermore, the loss in bolt tension in joints with head-free bars was smaller for a given number of cycles of opening and closing of the joint than joints with head-contact bars. It was recommended that a spring washer with at least 5000 pounds reactive spring pressure at the release of 0.03 inch be used. From the standpoint of track maintenance, a spring washer with a permanent travel of 0.10 to 0.12 inch when an initial load of 20,000 pounds is completely removed should be satisfactory. Five of the spring washers evaluated in this study either met or exceeded these requirements.

From the comprehensive investigation initiated in 1937 on rail joint wear, elastic action, and fatigue properties both in track and in laboratory rolling load fatigue tests, it was found that the rate of joint bar pull-in was much greater in track than was observed in the rolling load tests.[28] These results indicated that rail joint wear and joint bar pull-in in track was not primarily due to flexing of the joint under passing wheels. With new rail joints in track, the initial rate of joint wear, as obtained from the amount of joint bar pull-in, was rapid and was attributed to abrading of the mill scale on the rails and bars and also to adjustment of the bars to a better fit with the rail. Following an initial period of rapid wear, the rate of joint wear decreased, but in later years the rate of wear increased appreciably. It was concluded from these studies that joint wear in track depended more on the alternate opening and closing of the rail gap as a result of temperature changes in the rail than on the flexing of the joint under passing trains. It was conservatively estimated that the induced joint slippage in the laboratory tests probably accounted for about half of the wear that was found in track. Therefore, it appeared from the results of several investigations that corrosion probably was the major factor in causing joint wear. It was anticipated that lubrication would minimize corrosion, but that it would only be effective for a limited number of cycles.

Because of the development of cracks in the upper web fillet at the rail end and also through the bolt hole nearest the rail end, a program was undertaken to investigate the effect of bolthole spacing on rail web stresses within the rail joint.[29] For these studies, the then new 115-pound RE rail section with 36-inch head-free joint bars and 1-inch-diameter bolts were used. Stress measurements also were made on 131-pound RE rail joints in tangent track during the passage of regular trains. These joints also used 36-inch-long head-free bars and 1-inch-diameter bolts.

In preliminary tests using a brittle lacquer, the following findings were obtained.

1) The areas of greatest stress in the rail web were in the upper web fillet in the region of the rail end, around the end bolt hole, above the end bolt hole, and in the lower web fillet near the end of the rail.

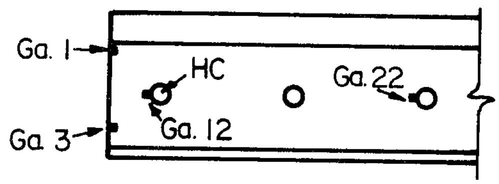
2) The direction of maximum stresses in the rail web generally was vertical although there was some variation from the vertical.
3) Stresses in the rail web due to the joint flexure were less than those due to the application of bolt tension.
4) There was great variability in the magnitude of stresses at corresponding locations on the four web faces of the two rail ends of the joint caused by variations in fit between the fishing surfaces of bars and rail.

Wedging action of the joint bars within the rail fishing surfaces by tightening of the track bolts produced tensile stresses within the rail web in a vertical direction as determined also by strain gage measurements. The vertical forces usually were unequal on the opposite faces of a rail end, and therefore a bending strain was developed that increased the vertical tension on one face and decreased it on the other. In addition, the vertical tension was increased by a factor of about 3 at the edge of a bolt hole due to the stress concentration effect of the hole. Irregularities in bearing contact at the fishing surfaces increased wedging of the bars between the rail head and base and increased the vertical tension in certain areas, particularly near the rail end. As a result vertical tensile stresses in the rail web at bolt holes that would be expected to be in the range of 15,000 to 20,000 psi with 30,000 pounds bolt tension were found to be as high as 50,000 to 70,000 psi. Moving the first bolt hole farther away from the high stress area near the rail end not only lowered the tension stress at the bolt hole, but also reduced the stress in the upper and lower fillets and web area at the rail end.

Provided that a moderate bolt tension was maintained, the stresses in the rail web did not change very much due to the passage of train wheels over the joints (in comparison with static stresses caused by bolt tension) except in the upper web fillet near the rail ends. A high range of repeated stress was measured in the upper web fillets especially if the wheel bearing was eccentric towards either the gage or field side that might produce fatigue failures as had been reported by several railroads. When the bolt tension was low, there was a significant increase in the range of web stress under traffic. The ranges of web stresses with various locations and three levels of bolt tension are shown in Figure 5. The wide rectangles are the averages of the stresses at the four-gage faces at each of two joints that were instrumented. The narrower rectangles are the highest stresses recorded at any one of these web faces. Laboratory rolling-load fatigue tests in which the rail joints were subjected to reversed flexure, simulating traffic conditions, showed that the fatigue strength of the joint was increased somewhat by moving the first bolt hole farther from the rail end. By doing this the stress in the last bolt hole was increased somewhat. Some of the supporting capability of the joint may also have been lost, but this was not conclusively determined. As a result of these tests, it was recommended that the then used spacing of bolt holes at the end rails of 2-1/2 inches - 6-1/2 inches - 6-1/2 inches be revised to 3-1/2 - 6 inches - 6 inches for six-hole joint bars and from 2-1/2 inches - 6-1/2 inches to 3-1/2 inches - 6 inches for four-hole joint bars. There was no significant difference in the amount of rail-end batter or joint deflection in rolling load tests with the various bolt-hole spacings.

In addition to the static performance of bolted rail joints, consideration also must be given to dynamic performance which includes the combined effects of both track characteristics and locomotive and car characteristics.

In an analytical and experimental investigation of the forces and strains accompanying impact at a low ("dipped") rail joint by a train at 100 mph, a peak force, P1, was shown to occur 1/4-1/2 millisecond after a wheel crosses the joint.[16] A second peak force, P2, occurs after 6-8 milliseconds, which is transmitted to the ties and ballast and is responsible for rail deflection. This is shown in Figure 6a. Experimental measurements of strains at the first bolt hole, shown in Figure 6b and c, show that there is a reversal of shear strain as the wheel crosses the joint. The combination of the two peak forces determines the stress range at the bolt hole and,therefore, the fatigue crack initiation. Calculations for several types of locomotives crossing dipped joints at about 100 mph showed that peak forces, rail bending moment, and downward rail deflection



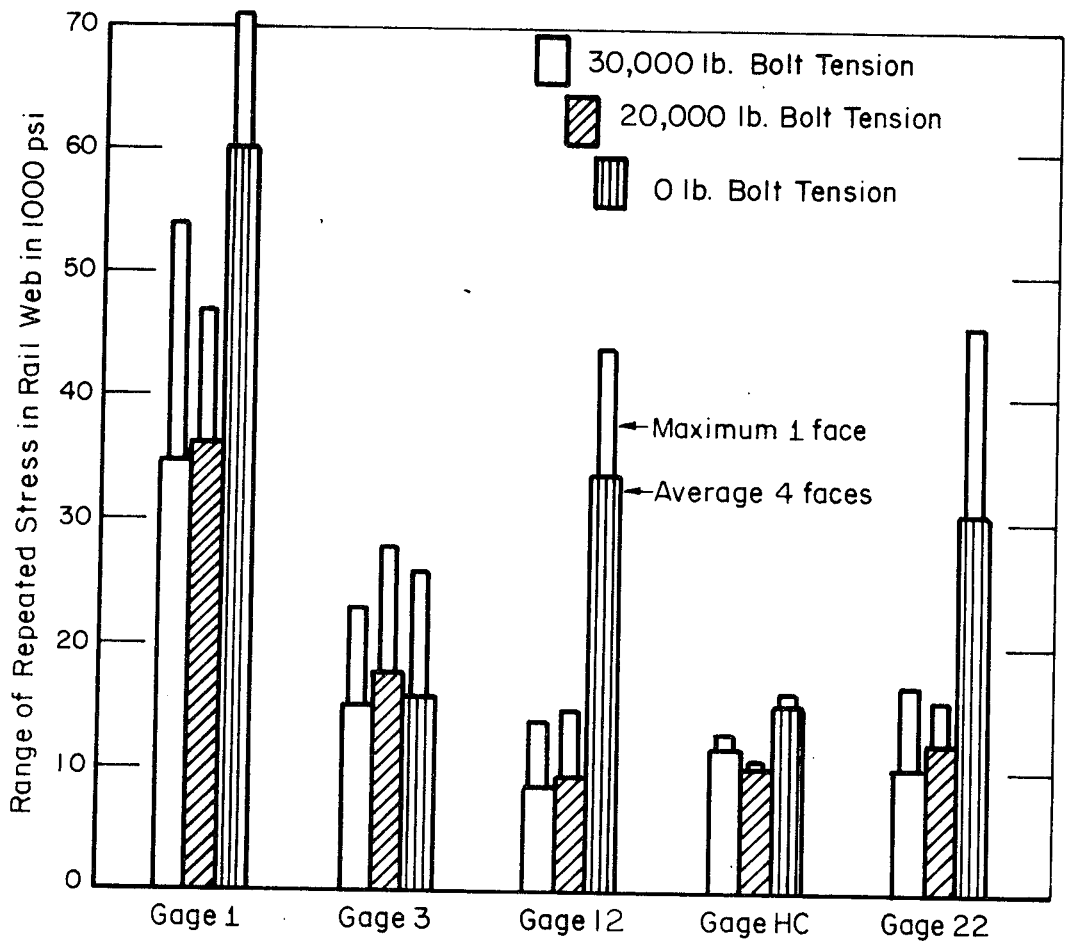
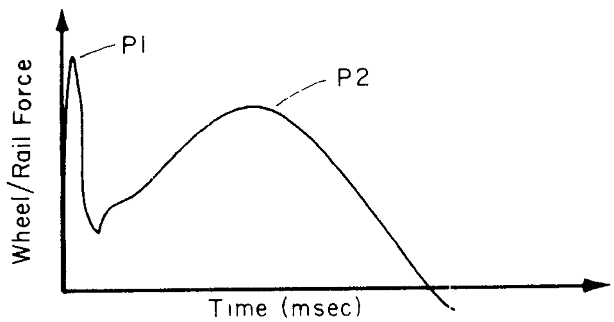
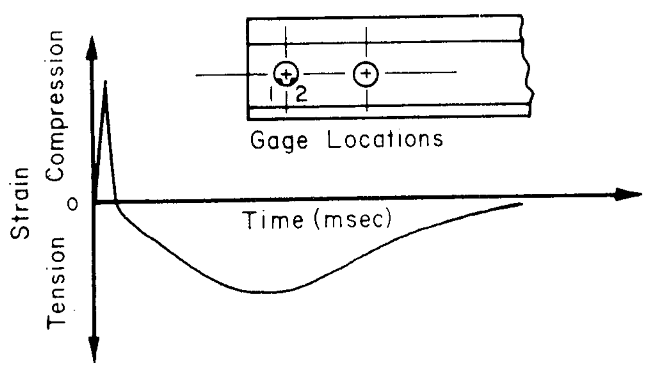


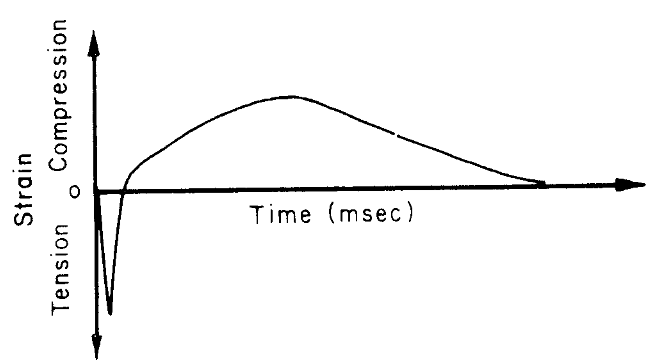
FIGURE 5. MAXIMUM RANGE AND AVERAGE RANGE OF REPEATED STRESS UNDER TRAINS IN THE FOUR WEB FACES OF 131 POUND RE RAIL ENDS WITH 36-INCH HEADFREE BARS DUE TO VARIOUS BOLT TENSIONS AND TO DIFFERENT CONDITIONS OF LOADING[29]



a. Rail Force at Joint Impact



b. Strain Measured at No. 1 Gage



c. Strain Measured at No. 2 Gage

FIGURE 6. TIME HISTORIES OF RAIL FORCE AND BOLT HOLE STRAIN MEASUREMENTS FROM JOINT IMPACT (16)

were roughly 3 to 5 times their values obtained under static wheel loading. Furthermore, each of the peak forces increased nearly linearly with increase of the product of vehicle speed and joint-dip angle (severity of dip). The peak forces also would increase with loss of bolt tension and improper joint-bar fit because the joint dip would be greater.

An additonal demonstration of the importance of dynamic action on rail-joint performance was given by Prause, et al.[22] The stress concentration factor around a bolt hole under static loading conditions was determined both analytically and experimentally using a photoelastic model. The results are plotted in Figure 7 and show that the analytical formulation predicted a maximum stress value of the stress concentration factor, K_θ, of 4.3. The higher experimental value, $K_\theta = 4.9$, was attributed to the concentrated wheel load that was applied at the rail end. Bolt-hole failures, however, almost always initiate at the lower 45-degree position ($\theta = 315$ degrees) where, under static vertical, downward lead the bolt hole surface would be in compression so that fatigue crack initiation would not be expected. However, as shown in Figure 6, the P2 dynamic force peak does produce a tensile stress at this location ($\theta = 315°$). Furthermore, visual inspection of disassembled joints having bolt hole cracks frequently has revealed that the underside of the rail head at the rail end is heavily worn suggesting that loose joint bars can produce an equivalent upward-acting shear stress at the rail end near the bolt hole.

In another study of dynamic loads at a single, opposite pair of dipped rail joints (about 1/2 inch low over a span of 24 inches when unloaded), it was found that there were greater differences in bolt-hole stresses due to passage of locomotives of differing unsprung mass in the speed range of 50 to 80 mph than at 100 mph.[30] On the basis of various studies of factors affecting bolt-hole stress, the authors concluded that the measured stress ranges could not account for failure at bolt holes by fatigue cracking or by brittle fracture initiation. It was suggested that variations of joint-bar fit and bolt tightness would explain the cause of failures that are observed. From other investigations reviewed in this survey, it is likely that loose bolts and joint bars cause bolt-hole failures by substantially increasing strains and stresses at the bolt holes.

Another area of track-train dynamics that is intimately associated with bolted rail joints is the development of "car rocking"

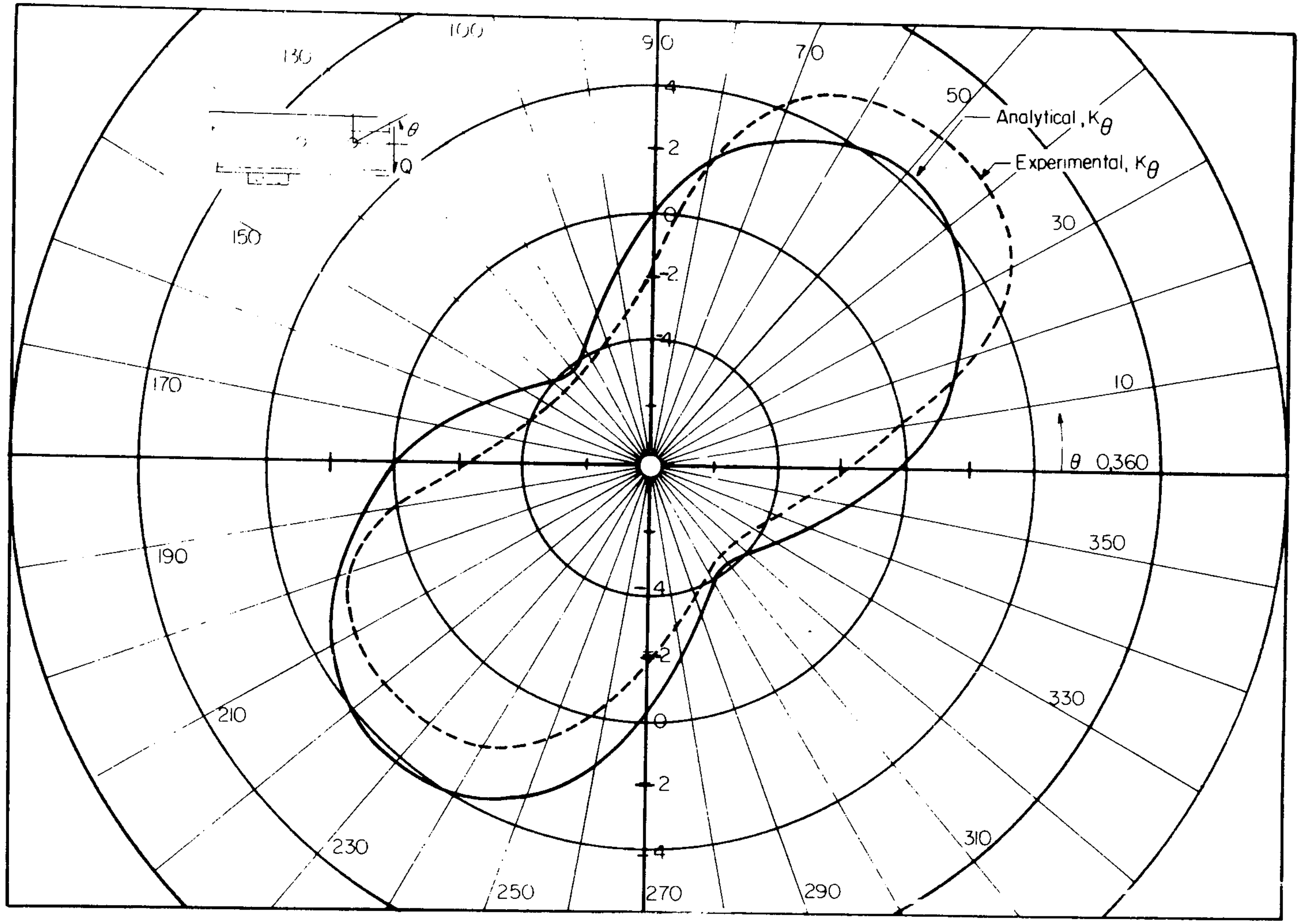


FIGURE 7. STRESS CONCENTRATION FACTOR AROUND A BOLT HOLE [22]

or "rock-and-roll", which can cause train derailment and damage to the vehicles, rails, joints, and track structure. This dynamic effect is unique because it occurs at low speeds. In order for car rocking to develop, three requirements must be met simultaneously:[31,32]

(a) There must be several consecutive low joints in track where the joints are staggered about half a rail length.

(b) The car must have a high center of gravity, a construction that is relatively stiff in twisting, and a truck center separation of 36-45 feet.

(c) The car must be traveling at a speed of 12-25 mph.

As the car passes over the track described, the rocking motion increases until wheel lift and, possibly, derailment occur.

Field and analytical studies have shown that the high wheels on curves lift off of the track near the rail joints and the impact loads, which may be more than twice the static wheel load, occur near the mid-length of the rail.[32] Car rocking develops more readily on curves than on tangent track so that, in addition to eliminating any or all of the requirements given above, reducing superelevation on curves so that it is appropriate for actual train speeds will help to prevent rocking.

Low bolted rail joints are a common occurrence and joints can be straightened during periodic track maintenance operations. It has been suggested that low joints result from compressive plastic deformation of the rail head surface that develops in small increments with the passage of train wheels. Rail-end batter and the initiation of rail-end dipping probably accelerate the development of low joints by increasing the impact forces at the rail ends. Although practically no published information that discusses this condition has been located, it has been reported that 10-20 percent of the joint bars break during the straightening operation. Some bolt-hole and head-web fractures are detected 3-9 months after straightening, which suggests that these fractures might be due to changes in the residual stress pattern from straightening. Web-base fractures also are encountered during straightening when the rail joint is in compression.

Standard, 6-hole, bolted joints can be installed by two men with predrilled rails in about 10 minutes. A 1-inch-diameter by 6-inch-long bolt, nut, and lockwasher costs about $0.75 and a 1-1/8-inch-diameter by

6-1/2-inch-long bolt, nut, and lockwasher costs $1.08. Assuming that a pair of joint bars costs $15 and that direct labor is $6 per hour, the direct cost of an installed joint is about $25.

4.1.2 Bolted Joint Performance

As stated in the Introduction, about half of all service and detected rail failures occur in the joint region. There were about 75,000 joint defects detected by Sperry Rail Service over 185,000 miles of track during 1973.(13)

Bolt hole fatigue cracks propagate at about 45 degrees to the rail axis at one or more of the bolt holes. On the basis of railroad experience, approximately 96 percent of these failures occur at the bolt hole nearest the rail end, about 85 percent occur on the running on rail for single direction traffic, and about 66 percent occur on the lower rail on curves.(33)

The major causes of the high failure rate of bolted joints are the lower stiffness and the presence of stress concentrations in the joint region.

Even when bolted joints are new and tight, the joint region has lower stiffness than the rail, which decreases the distance from the joint over which traffic loads are distributed.(34) For example, the two joint bars for the 140 RE rail section have about one-third the moment of inertia (vertical rigidity) of the rail and the joint deflection will be 4 times as great as that for a full rail under a uniform moment over an equal span.(21) Recent calculations of rail deflection for various combinations of car weight, rail weight (90,115, and 132 lb/yd), and modulus of elasticity of rail support (1000-5000 pounds load per inch of rail required to depress the rail 1 inch) are shown in Figure 8.(35) In field measurements it was found that rail joint deflection was less than 25 percent higher than deflection of the rail at its midspan for average or better track having a modulus of 3000 psi or higher. However, under poor track conditions, track having a modulus of 2000 psi or lower,

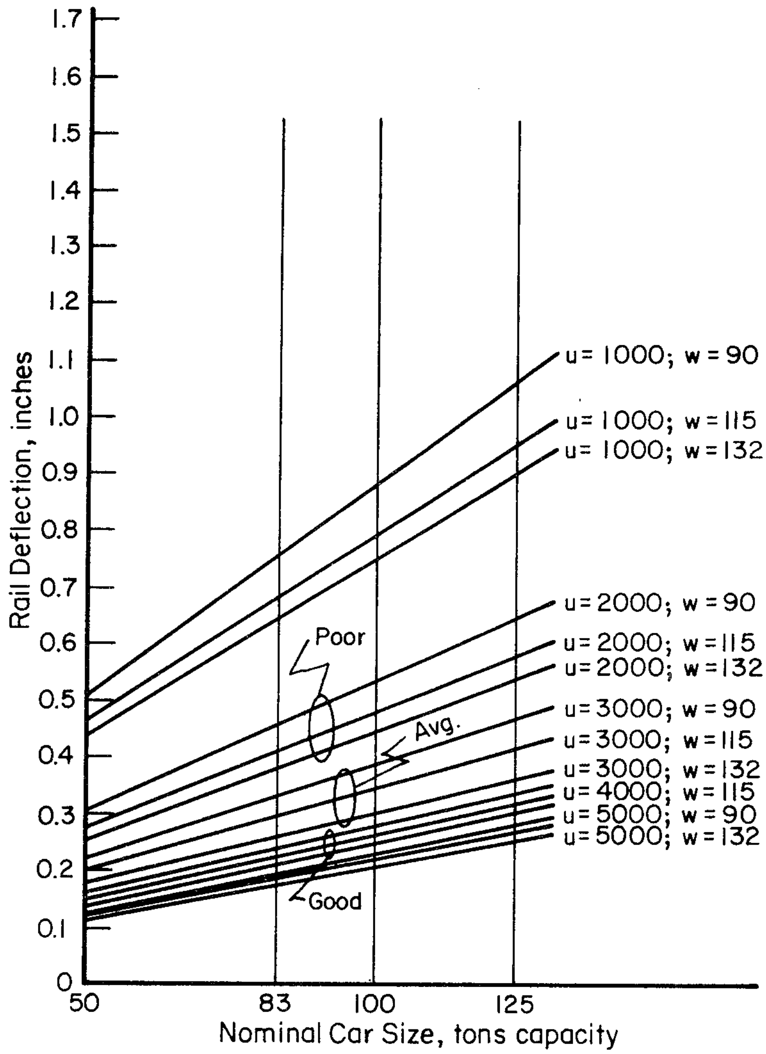


FIGURE 8. RAIL DEFLECTIONS VERSUS CAR SIZE, TRACK MODULUS, AND RAIL WEIGHT(35)

the joint deflection often was 400 percent of the rail midspan deflection and the continuous beam on elastic foundation model was not applicable. The lack of support led to greater joint deflection, and accelerated bolt loosening, fishing surface wear, and bending of the rail ends and joint bars.

Because of the reduced stiffness at the joint and resulting rail deflection and stress development at bolt holes, stress raisers such as drill gouges, burrs, and the rail brands can significantly reduce the fatigue life at joints.

Studies of the effect of stress raisers around a bolt hole on the fatigue life of rail were conducted by the Association of American Railroads (AAR) using 132 RE and 140 PS rail sections.[36] After various types of bolt-hole preparation, the rails were loaded to produce fully reversed bending in the rail web (simulating lateral bending loads in track) and a maximum stress of $\pm$ 50 ksi (R = -1) at the edge of the bolt hole. For each type of hole preparation, a maximum of 3 fatigue tests were conducted at stress levels of about $\pm$ 29, 36, 43, and 50 ksi. If a specimen did not fail in 10^7 cycles, the test was stopped. The following results were obtained.

a) Increasingly reduced fatigue lives were obtained with drill gouges, burrs, and holes drilled through rail brands. As expected, the depth and number of gouges, the size of the burr, and the location of the hole in the rail brand influenced the results. At the lowest stress levels, a drill gouge resulted in failure at 2-4 x 10^6 cycles compared to about 12 x 10^6 cycles without failure in the absence of a defect in 132 RE rail.

b) A further reduction in fatigue life (to 1.5-2 x 10^6 cycles) was measured with a combination of a bolt hole drilled through a brand with a burr.

c) Reaming the holes to eliminate gouges produced a slight improvement over standard holes (containing fine drill gouges) and holes with drill gouges.

d) A flat, 59-degree chamfer greatly increased the fatigue life in the case of burrs and holes through brands. Machining a radiused chamfer further improved fatigue life.

e) Tool peening around the bolt hole was ineffective when the hole was through a brand but improved life when a burr was present.

f) Shot peening around the hole was the most effective method for improving fatigue life.

Chamfering of bolt holes to remove burrs is recommended but the extent of usage of this practice is not known. As expected, dull drills and drilling through rail brands should be avoided.

An improvement of 25 percent in the fatigue strength of drilled rails is accomplished by the British Railways by broaching the carefully drilled holes to enlarge them and work harden the internal surface of the hole.[37-39] The nominally 1-3/16-inch-diameter holes are enlarged by 1/32 to 3/64 inch without damaging the steel around the circumference of the hole. The metal around the hole is work hardened to a depth of about 1/2 inch and surface irregularities are removed. A portable unit has been developed that permitted the work hardening, including the engagement and disengagement of the broach, to be completed in 1 minute.

The practice of broaching new bolt holes in rails (broaching of old holes was found unsatisfactory but no details were given) was instituted after the Hither Green accident in which 49 people were killed in 1967. Train derailment occurred after a piece of the running-on rail broke off. Other measures that were taken to prevent this type of failure included:[38]

a) Thickening the web of the 110 lb/yd rail section and making it parallel sided so that the web would not be thinnest near the bolt holes. This modification in conjunction with hole broaching was expected to increase resistance to fatigue failure by 50 percent.

b) Initiating efforts to develop a rail steel with higher fracture toughness so that brittle fracture would be less likely.

c) Installing bolts 1/4 inch longer with a locking device to prevent the nuts from loosening and using a higher strength bolt steel for points, crossings, and insulated joints.

d) Increasing the usage of ultrasonic pulse-echo, rail-flaw detectors with multiple heads that could detect a fatigue crack such as the 1/8-inch-long one that initiated the rail fracture at Hither Green.

e) Abandoning the practice of installing short bolted rail sections between CWR and a set of points.

Other methods that have been used by British Railways to improve fatigue strength are reducing the hole diameter, producing a smooth finish in the hole, and chamfering or radiusing the hole edges.(37,40,41) Hole-edge radiusing may be less effective in vertical bending of rail ends than in lateral bending which produces maximum fiber stresses at the hole edge. Lateral bending was used in the AAR study summarized earlier.(36,40)

In a recently completed study, sleeve cold working of fastener holes was shown to improve significantly the life of ultrahigh-strength steel components under cyclic loading.(42) In this process, the best fatigue properties were obtained by pushing a high-interference, tapered, carbide mandrel through a lubricated fastener hole to produce a high-level, compressive, hoop prestress that effectively reduces the hole stress concentration. For example, with 300M steel heat treated to 270-300 ksi tensile strength, cold working produced a 4 to 1 improvement in fatigue life in high-load-transfer applications at 110 ksi maximum net tensile stress (11 ksi minimum tensile stress). High-load-transfer conditions would apply to a bolted rail joint supporting longitudinal stresses. A 20 to 1 improvement in fatigue life was obtained under conditions of zero load transfer for the same stressing conditions. Under zero-load-transfer conditions, the fasteners are only required to clamp the parts together. For 3/8-inch-diameter holes in 3/8-inch-thick steel plate, the optimum expansion of the hole diameter was about 0.025 inch. A mandrel taper of 0.045 inch per inch was used. The best performance was obtained by cold working as-drilled (versus prereamed) holes and countersinking with a 100-degree angle after cold working. At a maximum net tensile stress of 110 ksi with net-fit fasteners in the completed holes, this procedure gave a fatigue life of more than 10,000,000 cycles compared to only 70,000 cycles for a reamed but not countersunk hole. Scoring of the cold-worked holes had no effect on the improved fatigue life. Reaming before cold working was more detrimental to fatigue life than either proper or even abusive drilling of holes before cold working. Furthermore, if the bolt holes were cold worked after fatigue cycling or 0.030-inch fatigue cracking, there was no reduction of fatigue life in comparison with unfatigued or uncracked holes. Similar results have been obtained by coining the region around the bolt holes. However, much higher forces are required for coining than for sleeve cold working with a tapered mandrel.

4.1.3 Summary and Recommendations

Although the bolted joint is relatively simple and inexpensive; its low stiffness, tendency to loosen, and the presence of stress concentrations combine to cause a high failure rate and to require high expenditures for maintenance of the joints and the overall track structure.

On the basis of prior studies that have been surveyed in this section, the performance of bolted joints (particularly those presently in track) can be improved by the actions described below; however, it is recognized that economic constraints would severely limit the application of any of these approaches.

1) Increasing the resistance to deflection of the joint region
   a) Shortening the time period between such maintenance activities as bolt tightening, joint lubrication, rail-end straightening, and ballast cleaning and recompaction near joints. The availability of maintenance funds severely limits this approach.
   b) Reducing the rate of joint-bar loosening by development of improved fasteners, corrosion inhibitors, and wear-resistant surface treatments. This approach could be feasible in special cases of joint rehabilitation.

c) Decreasing the tie spacing near joints to augment rail support that is not provided by the joint bars. This could introduce problems during track surfacing operations.

2) Reducing the stress concentrations in the vicinity of bolt holes, especially the holes nearest the rail ends.

These recommendations, which are interrelated, are discussed in the paragraphs that follow.

The relatively low resistance of rail joints to deflection under load is their major deficiency. Joint deflection deforms the rail ends and bars, distorts and reduces the support provided by the ballast and subgrade, and results in accelerated damage as the joint depression and impact loads increase (peak dynamic loads are a function of both train speed and amount of joint dip). Under conditions of increasing traffic tonnage, wheel loads, and train speeds, resistance to joint deflection is more important than in the past. Under circumstances where severe service conditions prevail, increasing the frequency of joint and support maintenance would decrease the rate of joint deterioration. Of course, for this and any other possible alteration of track construction or maintenance practice, the availability of personnel and facilities, the relative costs, and the benefits to railroad operations must be carefully assessed.

Bolt tightness has been shown to be a critical parameter for maintaining the resistance of joints to bending moments. With the accompanying requirement of permitting longitudinal slippage to compensate for rail-temperature variations, simply applying higher bolt preloads with higher strength bolt steels will not satisfy joint design requirements. An improvement might be obtained, however, by using bolts that have a more predictable torque-tension relationship. For example, washer-faced lock nuts are used extensively by the aerospace industry. Typically, these nuts have free turning washers built into the nut face to provide controlled bearing frictional loads during installation. This feature provides constant torque-tension relationships as well as improved bearing surface contact. It is possible to obtain nuts of this type with a set of spherical washers on the base to allow for installation on sloped surfaces. In addition to the washer face, nuts of this type are generally manufactured with a slight deformation of the cylindrical thread section to provide residual locking characteristics.

Another approach worthy of further evaluation is the application of coatings to bolt and nut threads that provide lubrication for constant torque-tension relationships and long-term corrosion resistance and lubrication for retightening at appropriate intervals.

Because joint loosening is caused primarily by joint bar and rail wear and corrosion where they contact, and also to some degree at contact areas between the bolt head, washers, nut surfaces, and joint bars; the use of compounds with greater and longer lived lubricity and corrosion resistance at these locations also might reduce joint loosening. Although this approach has not been studied extensively in this survey, film coatings such as dry graphite, molybdenum disulfide, and others developed for the aerospace industry to provide lubrication under severe temperature and atmospheric conditions should be evaluated further. Such coatings might be easily applied during normal joint reworking operations.

It has been demonstrated adequately that stress concentrations in the joint region frequently are responsible for initiation of fatigue failures. Several concepts may be applied to the problem of fatigue-property improvement. Precise hole size control may be obtained through the use of broaching mandrels. This technique is much more accurate and faster than "on-site" precision hole drilling and may be accomplished by the use of commercially available installation tools and hydraulic or mechanical pullers. Once a precision hole is prepared, several concepts can be employed.

Oversize mandrels or broaches can be pulled through the hole to provide favorable compressive residual stresses that reduce the maximum tensile stress during cyclic loading. An interface sleeve may be used to reduce friction and allow a greater degree of cold working; however, a postreaming or broaching operation may be required for final hole sizing. Shot peening also could be used to increase bolt hole and rail web surface compressive stresses although its use in the field in joint reworking operations has not gained acceptance.

A fastener can be pulled into an interference hole. This procedure preloads the area around the hole and reduces the stress amplitude during cyclic loading. The combination of hole cold working and using an interference-fit fastener will improve fatigue life by reducing both the maximum tensile stress and the stress amplitude experienced at the rail-bolt holes under traffic. This approach to improving bolted joint fatigue behavior can not be considered practical for conventional joints, however, because oversize holes in the joint bars would be required to allow slippage with rail temperature changes.

Chamfering to reduce the stress concentration at the bolt-hole edges has been effective also.

If bolt-hole and rail fractures following rail-end straightening are frequent, the residual stresses produced by straightening should be determined and methods to modify harmful stress patterns should be developed. Preheating the rail ends before straightening or thermally stress relieving after straightening may be effective in reducing the incidence of subsequent failures. However, the costs of the equipment and procedures to reduce stress concentrations, alter residual stress patterns and to impart compressive residual stresses for fatigue property improvement should be defined in detail and compared to the expected increase in joint reliability and life.

4.2 ADHESIVE BONDING

Adhesive bonding of rail joints was developed during the 1950's to improve the service performance of the insulated joints in continuous-welded-rail territory, required for track-signal circuits.(5,43) Thermal contraction of welded rail strings caused large gaps between the rail ends at conventional insulated joints, which, because of greater rail-end batter, reduced the insulated-joint life. Using proper installation procedures, the combination of a strong adhesive and high-strength bolts provides a frozen joint requiring little or no maintenance.

Adhesive bonding presently is used for fabricating insulated joints, standard joints, switches, and frogs. Bonded joints can be prepared in 10- to 39-foot sections in a shop and installed in track by thermite welding or can be fabricated at the track site.

4.2.1 Adhesive-Bonding Procedures(43-55)

The first step in the adhesive bonding of rail joints is rail-end preparation including careful drilling of bolt holes in the web if they are not already present and removal of fins and burrs around the holes by grinding. The rail brand is removed from the joint-bar area by grinding with care so that the web is not gouged. For insulated joints, the rail ends sometimes are beveled to a depth of 1/4 inch and 1/16 to 3/32 inch horizontally so that rail-head metal flow will not cause electrical shorting across the insulator. In some instances, the rail ends also are induction or oxyacetylene-torch hardened to a hardness of about 388 BHN to reduce head metal flow.

The rail and joint-bar contact surfaces are thoroughly cleaned by grinding or sand blasting with dry silica, which is preferred, followed by wiping with a clean cloth and solvent such as trichloroethylene, perchloroethylene, or methyl ethyl ketone. It sometimes is recommended that the rail ends be heated to about 200 F to remove moisture and to accelerate the curing of the adhesive.

For an insulated joint, the end post is installed and the rail ends are brought into modest compression using a rail puller or by heating or cooling the rails as required. After proper mixing, the adhesive is spread evenly over the clean and dry joint bars that are covered with an insulator. The bars are bolted to the rail being careful not to get dirt into the joint. It also is essential that insulating bushings or fiberglass tape around the bolt shanks in insulated joints are not damaged during installation. The bolts or Huck pins then are fully tightened, moving outward from the center of the joint. Application of the adhesive to the bolt and nut threads helps prevent loosening in service.

One manufacturer supplies fiberglass-reinforced-epoxy joint bars while all others use steel joint bars.

Depending on the ambient temperature and the adhesive used, the joints may be heated to temperatures in the range of 100-300 F to accelerate hardening. The joint should be cooled to 160 F or lower before traffic is restored or tensile or compressive loads are applied to the joint. A fine water spray sometimes is used to cool the bonded joint rapidly.

Placement of a tie under the center of an insulated joint requires an insulated tie plate. At least one railroad has recommended 3 ties under insulated joints.

During the field installation of adhesive-bonded joints, the most critical operations to be controlled are:

1) Grinding the joint region to remove bolt-hole burrs and raised rail-identification markings so that the joint bars will contact the rail uniformly
2) Sand blasting the rail ends to remove mill scale, rust, and dirt to insure a strong adhesive bond
3) Alignment of the rail ends
4) Application of the adhesive to insure complete joint coverage with no contamination
5) Placement of the joint bars so that an electrical short is not produced.

Somewhat more detail on the use of epoxys is contained in Appendix B.

It was determined during 1971 that one supervisor and seven men could install as many as 20 adhesive-bonded and bolted joints per day in the field at a cost of $41 per joint.[44] The material cost for the joint was $29. New, head-free, toeless joint bars and 6 Huck bolts were used with 133-lb rail. The cost of these joints is estimated to be close to $60 at the present time. Kits for bonded joints that include full-web-contact, "D" bars presently cost about $150. The relatively high cost of the adhesive bonded joints tends to preclude their use on low tonnage lines.

4.2.2 Mechanical Properties of Adhesive-Bonded Rails

Although there are relatively few reports containing mechanical property data for adhesive-bonded joints, those available indicate that satisfactory joints can be produced by this process.

After surviving 2 million cycles in a 33-inch-stroke rolling load machine under a wheel load of 44,400 lbs, a shop-bonded, insulated joint in 132-lb rail appeared to be in excellent condition.[52,56] The rail-end batter was small, only 0.005-0.006 inch greater than the average wear over 6 inches from each rail end. The difference between deflections at the maximum positive and negative bending moments was nearly constant at 0.036 inch, which indicates that the joint stiffness was nearly equal to that of 132-lb rail. The joint included full-web-contact joint bars, 6 high-strength bolts (1-inch-diameter) and a 5/32-inch-thick end post. The electrical resistance was 100 megohms after the test was completed.

A bonded joint in 132-lb rail from another supplier supported a static, longitudinal, tensile load of 800,000 lbs (61.8 ksi on the rail cross section) before yielding after it had been subjected to 2×10^6 cycles in a rolling load test using a wheel load of 44,400 lbs.[43]

A static, longitudinal, compression test of a bonded joint in 133-lb rail using 6-hole, full-web-contact joint bars with a 1/2-inch gap between the rail ends produced slippage at 660,000 lbs.[44] In a similar test using head-free, toeless, 6-hole angle bars in 133-lb rail, slip first occurred at 540,000 lbs. To prevent rail movement in continuous welded rail areas at -40 F, the joint must be able to support a load of 287,000 lbs which is much less than the loads at which the bonded joints slipped.

4.2.3 Service Performance of Adhesive-Bonded Joints

Although adhesive bonding of rail joints is not as widely used as other joining methods, the experience to date indicates that excellent performance can be expected. For example, an insulated joint in 132-lb rail was epoxy bonded with phenolic insulation on each bar, fiberglass matting between each bar and the rail web to evenly distribute the epoxy, and phenolic thimbles for insulation around each bolt.[(46)] In comparison with only 3-4 months life of this joint, when fabricated without an adhesive, the epoxy-bonded joint had experienced more than 100 million gross tons of traffic over 4 years with only rail-end batter to indicate that it had been in service. The joint had not moved and the adhesive was still intact.

Because properly made adhesive-bonded joints do not move with the varying rail forces, rail-end batter, the limiting factor in joint life, can be reduced by using narrower end posts in insulated joints.[(46)] By using a 5/32-inch-thick end post instead of the normal 3/8-inch-thick piece, rail-end batter has been reduced significantly. This appears to be a significant improvement in the construction of insulated joints because, once end batter has occurred, the adhesive in the joint cannot tolerate the heat introduced during rebuilding of the rail end by a welding process.

In another field test, begun in November, 1969, 3 shop-fabricated joints in 132-lb rail and 2 field-fabricated joints in 136-lb rail survived 5 years with temperatures as low as -23 F without opening.[(43)] As of March, 1972, this track had carried 35 million gross tons at a rate of over 1 million gross tons per month. A group of 75 bonded joints has survived for 4 years where the minimum temperature has been below -40 F.

During 1970 and 1971, one railroad installed over 1000 adhesive bonded joints in continuous welded rail carrying heavy traffic. These were obtained from several suppliers.[44] The joints prevented rail movement between the rail strings. Because of the satisfactory results obtained, the railroad began adhesive bonding standard, head-free, toeless angle bars for joining CWR strings and has discontinued using field thermite welding. In June of 1973, this railroad had about 500 miles of track in which all conventional and insulated joints between CWR strings were adhesive bonded.[45] This represents some 1800 glued joints. More than 1000 miles of CWR had been installed with glued joints by June, 1974.

Because a number of joint-bar fatigue failures originated at the base of the full-contact joint bar where it contacts the rail-base fishing surface, the installation practice now used is to grind the rail base with a taper to a maximum depth of 0.050 inch for 2 inches back from the rail ends. After combining this easement grinding with adhesive application over only the 12-inch end segments of each standard, head-free, toeless joint bar, for 825 joints installed during 1973, no broken joint bars were encountered as of October, 1974.[57]

To evaluate further the installation procedures for glued joints in CWR, 27 different test joints in 133RE rail were assembled using structural adhesive and Huck fasteners.[57] Four types of joint bars were evaluated, either adhesive bonded over the entire bar length or over only the end 12 inches of each bar. In addition, on some specimens, the rail-base ends were ground with a taper to a maximum depth of 0.05 inch and 2 inches back from the rail end as described above for in-track evaluations.

These joint specimens were tested head up in 3-point bending fatigue using a span of about 36 inches. A load was applied repetitively to produce a positive, downward moment of about 400,000 inch-pounds and then reduced to about 40,000 inch-pounds. At the minimum load, springs built into the testing fixture under the joint produced a negative moment of about 200,000 inch-pounds.

Based on the preliminary results, given in Table 5, the following conclusions were made:

(1) A fully glued joint with a rail base easement has the greatest resistance to fatigue failure.

(2) A fully glued joint is superior to a half glued joint. The cause of the improvement may be that the adhesive acts as a cushion to prevent impingement of the rail on the joint bar or that the full-length gluing may stiffen the joint sufficiently to reduce either the bending stress or the relative motion between the bar and the rail.

Of the more than 600 adhesive-bonded joints in service on another railroad in the Fall of 1973, only 2 have required removal from track. One failure was caused by a prior bolt-hole crack in the rail and the other resulted from improper assembly that caused an electrical short.[46]

On a railroad that thermite welds shop-built, insulated-joint sections into track, one joint that formerly was replaced about once a month has required no maintenance in 7 years. Here the insulated joints are prepared in a shop because this permits closer control of the bonding process. It was concluded from another investigation that the service life of bonded rail joints, which has not been fully established, may be limited by head wear and end batter rather than by deterioration of the adhesive and insulation materials.[46] For insulated joints, although the initial cost is higher, the lifetime cost including maintenance is expected to be significantly lower.

TABLE 5. 133 LB RE TEST JOINTS ASSEMBLED WITH STRUCTURAL ADHESIVE AND HUCK FASTENERS (57)

	Cycles to Failure			
Description of Application	133 lb RE 38 Inch Headfree Joint Bar	133 lb RE 38 Inch D Bar	133 lb RE 36 Inch Insulated Joint Bar	133 lb RE 38 Inch Bonded Insulated Joint Bar
Glue applied entire length of bar without rail base easement	3,900,000	870,000	Ran Out (8,000,000)	Ran Out (6,000,000)
Glue applied entire length of bar with rail base easement	Ran Out (6,000,000)	In Test	Ran Out (8,000,000)	Ran Out (6,000,000)
Glue applied end 12 inches of bar without rail base easement	490,000	In Test	4,900,000	--
Glue applied end 12 inches of bar with rail base easement	720,000*	To Be Tested	Ran Out (6,000,000)	To Be Tested

* Failure initiated at grinding burr at beginning of rail base easement.

Note: Rail base easement ground and tapered maximum of 0.050 inch deep and two inches back from rail end.

4.2.4 Summary and Recommendations

Adhesive bonding of rail joints has the attributes of simplicity, low capital investment, and portability. This method also can be used readily in a shop or in the field. The joint quality is less sensitive to variations in procedures and human judgment than that of thermite welding. Although service experience has been limited, the performance reported has been significantly better than that of conventional bolted joints. The most profitable applications have been for insulated joints and joints between CWR strings. Other promising applications may be in turnouts and crossovers. Because of the high cost of adhesive bonded joints, the general use of the technique in place of conventional mechanical joining is generally is not justified.

Continued evaluation of glued joints by the industry is recommended. Emphasis should be placed on determining, quantitatively, the load carrying capabilities and life of these joints along with their installation and maintenance costs so that accurate life-cycle costs can be established. Efforts to reduce the costs of fabrication and installation are encouraged with the ultimate goal of broadening the applicability of this type of joint.

4.3 THERMITE WELDING

Thermite welding is a process that produces coalescence "by heating with a superheated liquid metal and slag resulting from a chemical reaction between a metal oxide and aluminum, with or without the application of pressure. Filler metal, when used, is obtained from the liquid metal."[58] The most common exothermic chemical reactions used for thermite welding are:

(1) $Fe_2O_3 + 2\ Al \rightleftharpoons 2\ Fe + Al_2O_3 + 181.5$ kcal

(2) $3\ Fe_3O_4 + 8\ Al \rightleftharpoons 9\ Fe + 4\ Al_2O_3 + 719.3$ kcal.

The first reaction is used in the Goldschmidt process (Orgotherm)[59] and the Calorite and Boutet (Delachaux)[60] processes. The second reaction is the basis of the Thermit welding process that was developed in the U. S.[58]

The temperature of the molten metal is about 3500 F, which is less than the theoretical temperature of about 5000 F. The reduction of temperature results from various heat losses and the addition of other materials to the mixture. These other materials include carbon, manganese, pieces of high-carbon steel, and other alloying elements to increase abrasion resistance and provide grain refinement. The alloying elements are added so that the solid weld metal will have mechanical properties similar to those of the rail steel being joined.

Thermite welding is used extensively in the United States. This welding process is used almost exclusively for joining rails at the track site. Because the mechanical properties of thermite welds usually are inferior to those of welds made by other processes, usage in Japan is largely restricted at present to emergency repairs of continuous-welded rail, although an effort is being made to improve the weld properties, particularly fatigue resistance.

Efforts are being directed by Japan National Railways to improving the fatigue strength of thermite welded joints by eliminating or removing the bottom reinforcement and by improving the soundness and metallurgical structure of the weld. Improvement of 10-15 percent in fatigue strength is sought. Details of these studies have not been reported. At present, thermite welding has been replaced in Japan by enclosed arc welding which is described in a subsequent section. Photographs of the thermite welding operation are shown in Figures 9 and 10.

4.3.1 Thermite Welding Procedures

Although certain procedures used in thermite welding are recommended for specific processes, general procedures for successfully making rail welds can be given. The procedures outlined below are considered to be the minimal practices needed to provide acceptable weld quality:[58-71]

(1) Cut the rail ends perpendicular to the rail axis using a torch, saw, or abrasive-disc. Torch cuts should be relatively smooth and, to prevent rail cracks from initiating at the torch-cut, heat-affected zone, thermite welding should be performed within one hour of cutting. The heat-affected zone is brittle martensite that cracks spontaneously. To avoid this time limitation, the brittle metal at the torch cut can be completely removed by grinding shortly after torch cutting.

(2) Clean the rails within about 5 inches of the joint by filing, wire brushing, and solvent wiping to remove dirt, grease, moisture, loose oxide, and slag. Remove burrs and deformed head metal.

(3) Separate the rail ends by a gap of 1/2 - 1-1/4 inch depending on the rail section and welding process.

(4) Align the rails using a straight edge along the gage side of the rail head.

(5) Raise the rails at the joint to compensate for the greater thermal contraction that occurs in the rail head during cooling relative to the web and base regions. The amount of joint elevation is measured with a 24 to 40-inch-long straight edge centered on the joint. The correct elevation is obtained when about 1/16 inch separates the top of the rail head and bottom surface of the straight edge at both ends.

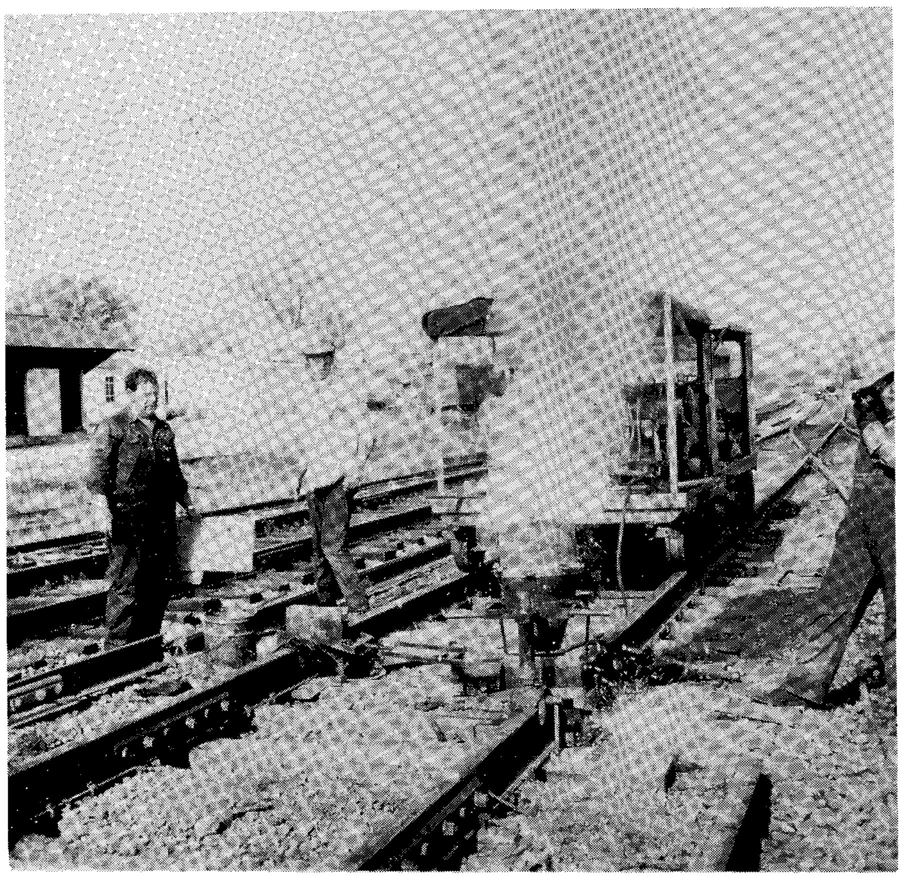


FIGURE 9. REACTION OF THERMITE MIXTURE IN CRUCIBLE BEFORE POURING INTO MOLD

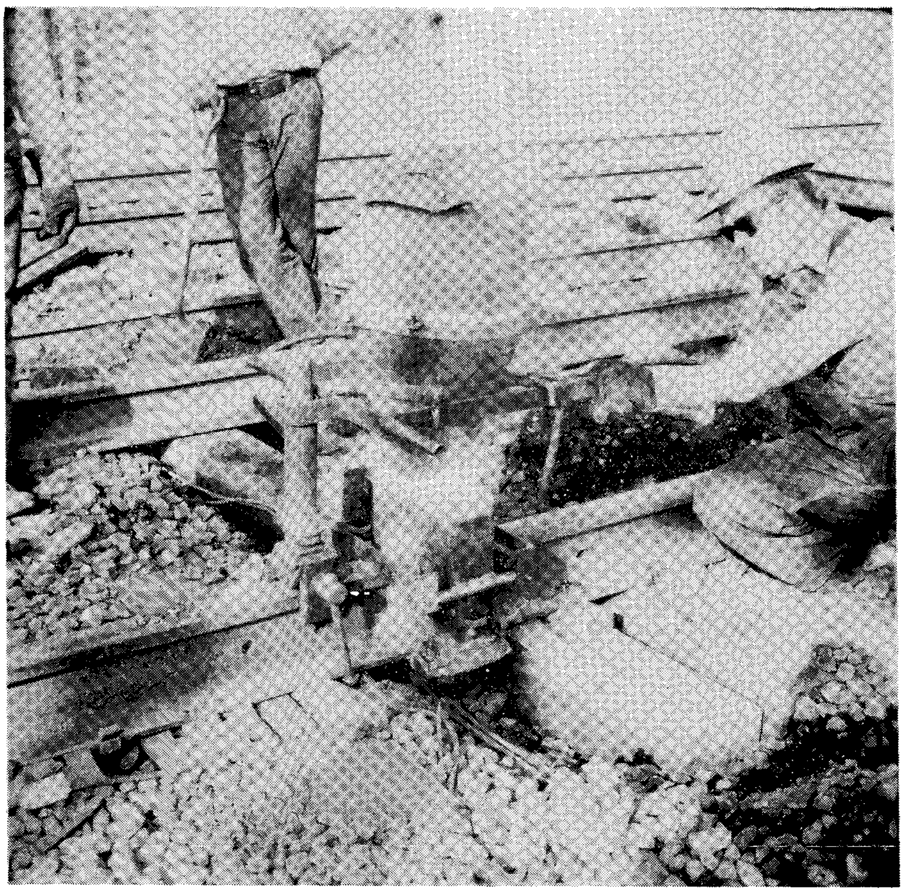


FIGURE 10. MOLTEN THERMITE POURED INTO MOLD ON RAIL

(6) Attach the molds to the rail, centered over the joint, and seal the molds to the rail with luting material, according to the equipment supplier's instructions. The luting material, a mixture of clay and sand, must be handled carefully so that none is introduced into the weld chamber where it might become included in the weld.

(7) Insure that the crucible or reaction chamber is clean and dry. Install the tapping plug in the crucible and pour in the prescribed amount of thermite mixture.

(8) Preheat the rail ends sufficiently to provide good fusion with the weld metal. Preheating is accomplished by a gas flame, a higher temperature filler metal, or by the initial filler metal flowing over the rail ends and into a sump. When preheating is done only with a gas flame, it is carried out for 5-12 minutes in order to raise the rail temperature to at least 1800 F. Melting at the rail ends, which would introduce large oxide particles into the weld metal, must be avoided. Uniform heating of the rail ends is important and can be judged by visual observation of the rails.

(9) Ignite the reaction mixture in the crucible. If the crucible is tapped manually, it is essential that sufficient time be allowed for completion of the reaction and to permit the slag to float to surface of the molten metal. A stopwatch and observation of slag formation in the crucible are two methods used to time the manual tapping operation.

(10) Remove the molds after allowing the weld metal to completely solidify, which requires about 5 minutes.

(11) Chisel and grind off excess weld metal to the rail profile, at least on the top and sides of the head. Final grinding should be performed when the weld and rail have cooled nearly to ambient temperature.

(12) Inspect the weld by visual, dye-penetrant, and ultrasonic nondestructive methods. Some urban track fabricators have radiographically inspected thermite welds using the Co^{60} isotope for a gamma-ray source.

Although detailed procedural descriptions accompany each thermite welding process, railroads have had widely varying degrees of success with the process. This variability is considered to be due to the inherently large amount of human judgement that is required during thermite welding, especially for the rail preparation, torch preheating, and manual tapping steps.

Three approaches have been used to overcome the problem of variable weld quality. The first method is to define the welding procedures in great detail and to supervise closely the welding operation using adequately trained supervisory personnel.[70,72] Typical procedure details that have been used to reduce weld variability include the following:

(1) Keep molds and thermite mixture dry. Discard any thermite mixture that has been exposed to moisture.

(2) To prevent hot cracking of welds as they solidify, which results from rail contraction before solidification is complete, do not weld within 2 hours after the start of rain if the rail has been heated above the ambient temperature by direct sunlight. For the same reason, thermite welds should not be made if the rail temperature might decrease more than 5 degrees F before the weld has completely solidified. The temperature should be measured on the web on the shady side of the rail using the clean surface of a plant weld if possible. Also, remove rail laying equipment from the rail being welded and turn off all power equipment on the track near the joint being welded to prevent disturbing of the weld before it solidifies.

(3) If welding is performed in the rain, when the rail temperature is stable, the rail joint, thermite mixture, molds, crucible, etc. must be protected from the rain. The completed weld should be protected as long as possible or until it has cooled to 100 F.

(4) Cut the rail to adjust the gap immediately prior to welding in order to minimize the likelihood that the gap will change during welding. If the gap is excessive, do not reduce it by localized heating of the rails; however, if the gap is reduced by a rising ambient temperature, the weld can be made while the temperature is increasing.

(5) Depending on the size of the rail, prescribe a minimum time for external torch preheating. The rails should be watched with tinted glasses to be sure that they are heated uniformly. Remove the torch near the end of the preheating for a final check of temperature and uniformity. If these are adequate, preheat about an additional minute to replace the heat lost during inspection.

(6) For the manual tapping of the molten thermite charge, either use a stopwatch or base the tapping time on the size of the slag ring that forms in the crucible after completion of the reaction. If the crucible is tapped too soon, the weld will pick up alumina inclusions and also may have a higher aluminum content that will lower weld ductility. If tapped too late, the molten steel will be cooler and may not produce adequate fusion with the rail ends.

(7) If excess metal above the running surface is removed by torch cutting after the weld has been allowed to cool for about 5 minutes after pouring, protect the rail base with sand or mold material from contact with any liquid steel.

(8) Do not allow the chisels used to remove excess weld metal to gouge the rail web or base. Carefully grind the weld contour so that it blends smoothly into the rail and so that no stress raisers are introduced. Also, avoid excessive grinding pressure, which could overheat the rail steel and cause it to crack upon cooling.

The second approach to decreasing the variability of thermite welds has been to modify the process so as to reduce the amount of human judgment needed. One such judgment associated with external preheating can be eliminated by using a larger and possibly hotter thermite mixture, which preheats the rail ends and melts off a small amount of rail steel as the molten steel washes over the rail ends. This process modification is incorporated into the Thermex Metallurgical "Thermit" process, the Boutet process, and the Orgotherm "SoV" Quick Welding method. The latter process is used for about 98 percent of field welds for the West German Federal Railroad. The Thermex, Orgotherm, and Boutet processes are used successfully by several U.S. railroads. A modification in which preheating was accomplished with thermite attached to the inner surface of the mold reportedly did not raise the rail end temperature sufficiently to provide adequate fusion between the casting and the rail ends.

A second process modification that partly automates thermite welding permits self tapping of the molten charge. To accomplish this, the Thermex Metallurgical "Thermit" process includes 5 metal discs that are placed at the bottom of the crucible and the Boutet process includes a solid plug that is placed in a tapping thimble below the thermite mixture. Both of these tapping devices are designed to prevent the molten steel from entering the rail joint until the reaction is complete and the alumina slag has separated from the melt by flotation in the reaction crucible.

Both of these process modifications appear to improve the service performance of thermite welds significantly. Additional service time in track and accumulation of failure statistics are needed to make adequately supported conclusions.

The third approach to overcoming the problem of uncertain thermite-weld quality, which is used successfully by several railroads, has been to reinforce the welds with bolted joint bars. If excess weld metal is not removed from the rail web and base, specially shaped joint bars can be used. If the rail ends are misaligned, grinding may be needed to obtain proper joint bar fit to the rails. Although this method has higher installation and maintenance costs than a joint that is welded only, the uncertainties of weld reliability can be overcome.

4.3.2 The Cost of Thermite Welds

The direct cost of making thermite welds in the field is estimated to be $65 based on observations of an 8-man welding gang working on a closed track section. During the 8-hour shift, 12 welds were completed and no inspections were performed. The estimated cost is calculated as follows:

Direct labor; 8 men x 8 hours x $6/hr	$384
Welding kits; $30 ea. x 12 welds	360
Other consumables and equipment; $3 x 12 welds	36
Total cost	$780
Direct cost per weld	$ 65

Including indirect costs, the total cost of welding is expected to be much greater than this. The direct cost is in reasonable agreement with a value of $55 for thermite welding on the West German Federal Railroad in 1972.(73)

4.3.3 Mechanical Properties of Thermite-Welded Rails

Some results of slow-bend tests of thermite-welded rails are given in Table 6. The majority of these test results, from References 74 and 75 were obtained at the AAR Research Center with the rail resting on supports 4 feet apart and loaded at 2 points, 6 inches on each side of the weld. The tests were made with the rail base down so that the

TABLE 6. RESULTS OF SLOW-BEND TESTS OF THERMITE WELDED RAILS

Rail Section, lbs/yd	Type of Weld	Max. Load, lbs x 10^{-3}	Max. Defl., in.	Energy for Fracture, ft-lbs	Modulus of Rupture, psi x 10^{-3}	Remarks	Ref.
100RE	Orgotherm	200	0.7	7,900	101.1	Failed in weld	74
100RE	Ditto	216	1.0	12,800	109.2	Base and web failed in weld, head failed 2 in. from weld	74
100RE	"	214	1.1	14,800	108.2	Failed outside of weld	74
100RE	"	223	1.2	16,200	112.8	Failed outside of weld	74
115RE	"	290	1.4	25,000	118.6	Failed outside of weld, columnar grains in head of weld	74
115RE	"	311	1.8	35,100	127.2	Failed outside of weld, columnar grains in head of weld	74
132RE	"	258	0.4	5,000	84.[illegible]	Base and web failed in weld, head failed 3 in. from weld. A large pearl was in the base.	75
132RE	"	396	1.6	[illegible]	129.[illegible]	Base and web failed in weld, head failed 4 in. from weld. Uniformly coarse grained weld metal	75
132RE	Thermex	332	1.0	20,100	108.3	Base and lower half of web failed in weld. Head failed 2 in. from weld. Uniformly coarse grained weld metal	75
132RE	Ditto	395	1.4	34,100	128.8	Failed in weld. Uniformly coarse grained weld metal	75
115RE	Exomet	133	0.2	1,300	54.4	Failed in weld. Large unfused area in base	75
115RE	Ditto	190	0.4	4,100	77.7	Failed in weld. Large unfused area in base	75
112	Orgotherm	129-156	0.6-1.0			SmW quick welding with collar. One meter span	59,76
136	Ditto	190	0.9		66.1	SoW quick welding without collar. One meter span	59,76
99(S49)	"	103	0.6			SoW quick welding without collar. One meter span	59,76
119	"	138-152	0.6-0.7		59-65	SoW quick welding without collar. One meter span	59
136	"	188-198	0.9-1.0		65-69	SoW quick welding without collar. One meter span	59

TABLE 6. (Continued)

Rail Section, lbs/yd	Type of Weld	Max. Load, lbs x 10^{-3}	Max. Defl., in.	Energy for Fracture, ft-lbs	Modulus of Rupture, psi x 10^{-3}	Remarks	Ref.
112	Orgotherm	130-138	0.5-0.8			SoV quick welding without preheat. One meter span	59
119	Ditto	142-153	0.6-0.7		61-66	SoV quick welding without preheat. One meter span	59
99(S49)	"	91-100	0.5			SoV quick welding without preheat. One meter span	59
101	--	150-220	0.3-0.9			Japanese 50 rail	77
112	Thermex	250	1.0	13,750	103	Failed at weld line. No defect on fracture surface.	78
112	Ditto	242	1.2	17,100	100	Failed at weld line. Gas pores on fracture surface	78
112	"	249	1.0	13,800	103	Failed at weld line. Gas pores on fracture surface. Head down.	78

rail base was subjected to tensile loading. Although none of these welded rails met the tentative criterion for oxyacetylene, gas-pressure and flash-butt welds of 140,000 psi modulus of rupture (bending moment divided by rail-base section modulus), two exceeded the minimum requirement of 1.5 inches deflection and several failed outside of the weld.

The bend-test results from References 76 and 59 were obtained by single point loading at the weld with the rail supported at points 1 meter (39.4 in) apart. They indicate that no large bend property differences exist among the three modifications of the Orgotherm thermite welding process.

Bending fatigue tests of thermite welded rails (West German type S49) having a tensile strength of about 100 ksi and a carbon content of 0.5 percent indicate that the welding process without a collar, which is the excess weld metal on the sides of the rail ("SoW"), had slightly higher fatigue strength than welds with a collar ("SmW"). Also, welds with or without a collar ("SmW" or "SoW") that were fully ground after welding had higher fatigue strengths than unground welds.[59,76]

In another study of the bending-fatigue behavior of thermite-welded rail, Loubser[79] showed that the maximum tensile stress of unground welds under a vertical bending load occurred at the edge of the weld under the rail base. At this position, where most welds had some lack of fusion and 70 percent of the bending-fatigue failures occured, a stress-concentration factor of about 1.4 was determined. Superimposed longitudinal tension ($R>0$), which could result from the actual track temperature dropping below the rail installation temperature, significantly lowered the fatigue limit of the rail welds in comparison with tests run at $R=0$ conditions. Conversely, a compressive minimum stress ($R<0$) greatly raised the fatigue limit of the welded rails. Fatigue failures that occurred through the center of the weld metal invariably initiated at a centerline shrinkage cavity.

The results of rolling load tests of thermite-welded rails, conducted at the AAR Research Center, are given in Table 7. The wheel load that was applied usually was based on the rail section being tested.

TABLE 7. RESULTS OF ROLLING LOAD TESTS OF THERMITE WELDED RAILS

Rail Section, lbs/yd	Type of Weld	Rolling Load Machine Stroke, inch	Wheel Load, lbs	Number of Cycles x 10^{-6}	Remarks	Ref.
100RE	Orgotherm	12	40,000	2.0	No failure (4 specimens)	74
100RE	Ditto	33	40,000	2.0	No failure	74
100RE	"	33	60,000	0.27	Rail broke	74
132RE	Exomet	12	57,500	1.35	Rail broke, failure originated at weld metal, nonmetallic inclusion in rail head	74
132RE	Ditto	12	57,500	1.30	Rail broke	74
115RE	Orgotherm	12	48,000	2.0	No failure (2 specimens)	74
132RE	Thermex	12	57,500	2.0	No failure (3 specimens)	75
132RE	Orgotherm	12	57,500	2.0	No failure	75
132RE	Ditto	12	57,500	0.79	Failure originated at weld-metal inclusion,and propagated through web at 45 degrees to head and base fillets.	75
115RE	Exomet	12	48,000	0.62	Failure attributed to high alumina in weld or foreign material in rail steel.	75
115RE	Ditto	12	48,000	1.11	Failure originated at metal fin in fillet beneath rail head.	75
127NYC	Orgotherm	12	54,500	1.53	Failed in rail about 2 in. from weld at horizontal split head.	75
127NYC	Ditto	12	54,500	0.82	Failed in rail about 2 in. from weld at horizontal split head.	75
136RE	Boutet	12	59,500	2.0	No failure (3 specimens), welds containted voids and cracks.	80
136RE	Orgotherm w/ defects	12	59,500	1.43	Failure originated at oxide inclusion in head.	80
136RE	Ditto	12	59,500	0.47	Failure originated at head-web fillet from oxide on weld collar.	80
136RE	"	12	59,500	1.85	Failure originated at oxide inclusion on weld collar surface.	80
136RE	"	12	59,500	1.09	Failure originated at oxide inclusion on side of head in weld metal.	80
136RE	"	12	59,500	1.41	Failure originated at oxide inclusion in weld metal in center of rail head.	80

TABLE 7. (Continued)

Rail Section, lbs/yd	Type of Weld	Rolling Load Machine Stroke, inch	Wheel Load, lbs	Number of Cycles x 10^{-6}	Remarks	Ref.
136RE	Orgotherm w/ defects	12	59,500	2.0	No failure (5 specimens)	80
100RE	Thermex	12	40,000	0.24	Fatigue failure originated at inclusion in center of head at fusion boundary	80
136RE	Boutet	12		2.0	Three specimens showing voids and porosity radiographically, one weld contained a shrinkage crack, void, and porosity in rail head.	81
136NYC	Orgotherm	12		1.43	Voids in base and web, failure started at area of burnt steel in head	81
136NYC	Ditto	12		2.0	No failure, voids in base and web (2 specimens)	81
136NYC	"	12		2.0	No failure, voids in web	81
136NYC	"	12		2.0	No failure, voids in base	81
136NYC	"	12		2.0	No failure, small voids in base	81
136NYC	"	12		0.471	Voids in base and web, failure started at iron oxide deposit in upper fillet	81
136NYC	"	12		1.85	Voids in base, failure started near center of web	81
136NYC	"	12		1.09	Transverse separation and voids in base, failure started at iron oxide deposit in head.	81
136NYC	"	12		1.41	Voids in base, failure started near center of head.	81
100ARA-A	Delachaux	12		2.0	Three specimens, defects found in base radiographically. No failure. Also sustained 2 million cycles of pulsating with maximum tensile fiber stress of 26,600 psi at rail base.	81

In these tests, 2×10^6 cycles of loading without failure were considered a runout. The 12-inch-stroke machine imposes only tensile loads on the rail head from zero to the maximum load. The 33-inch-stroke machine produces alternate tension and compressive stresses in which the maximum compressive stress is one-half the maximum tension stress ($R = -0.5$). The significant features of these tests results are that (1) most failures originated at oxide inclusions and (2) some welds that contained cracks, porosity, and voids survived 2×10^6 loading cycles without failure. This latter result can be attributed to the shape and location of the defects, which would minimize their tensile-stress concentrating effect.

The lower strength and ductility of thermite welds in comparison with flash and gas-pressure welds is attributed mainly to the cast, dendritic structure that is typical of thermite welds. Weld defects, such as inclusions and pores, also reduce the mechanical properties of thermite welds. Unfortunately, no data have been located that show the effect of variations in thermite-weld-metal composition on mechanical properties or service performance.

In a study of residual stresses in thermite welded rails, it was shown that the residual stress pattern developed in welded rails was advantageous under traffic load conditions.[82] In unwelded rails, the longitudinal residual stresses are tensile in the rail head and base and compressive in the rail web. The pattern was reversed in welded rails, with the web in residual tension and the head and base in residual compression. The same residual stress patterns were developed in welds made with or without a reinforcement (collar) on the weld.

4.3.4 Service Performance of Thermite-Welded Rail

Because of their lower mechanical properties and greater variability in quality, thermite welds generally do not perform in track as well as flash-butt and oxyacetylene gas-pressure welds, where performance is considered to be time in track without failure. One measure of this is the accumulation of weld-failure statistics published by the AREA and presented for 2 years, 1965 and 1970, in Table 8. These

TABLE 8. ACCUMULATED BUTT WELD FAILURES TO DECEMBER 31, 1965, AND DECEMBER 31, 1970[2,84]

Rail	Number of Welds		Weld Years		Failures: Service		Failures: Detected		Failures: Total		Failures Per 100 Weld Years		Avg. Weld Age, years	
	1965	1970	1965	1970	1965	1970	1965	1970	1965	1970	1965	1970	1965	1970
Flash-Butt Welds														
New	1,324,693	2,920,254	5,097,471	14,982,021	263	440	45	134	308	574	0.0060	0.0038	3.85	5.13
Relay	387,808	896,631	871,022	3,792,852	111	191	20	52	131	243	0.0150	0.0064	2.25	4.23
Oxyacetylene-Gas-Pressure Welds														
New	571,182	1,087,484	3,704,094	8,088,324	122	414	69	300	191	714	0.0052	0.0088	6.48	7.44
Relay	148,661	628,357	371,900	2,762,020	33	556	14	179	47	735	0.0126	0.0266	2.50	4.39
Thermite Welds														
New	3,581	15,238	24,268	43,400	134	104	12	54	146	158	0.6016	0.3640	6.78	2.85
Relay	1,593	19,722	6,839	53,802	30	78	1	122	31	200	0.4532	0.3717	4.29	2.73

figures, which were collected from the railroads on a voluntary basis beginning in 1962, are incomplete since not all railroads have submitted reports. Although "absolute comparisons and total failure rates cannot be derived from the data", "trends can be examined".(83) These trends indicate that thermite welds have a much higher failure rate than flash welds and gas-pressure welds. During the period of 1965-1970, the accumulated number of thermite welds that were reported increased substantially, which reduced the average weld age along with the failure rate. This reduced failure rate also may be attributed to improved processes and procedures.

The most common causes of in-service, thermite-weld failures are:(85-88)

(a) Porosity, voids, and inclusions (mold material or alumina from the thermite mixture) in the weld metal

(b) Gouges and local regions transformed to brittle martensite produced during grinding of the weld and adjacent rail.

In spite of the poorer service performance experienced by many railroads, thermite welding is widely used and, on at least one railroad, the failure rate is reported to be as low as that for shop-fabricated, flash-butt welds. This has been accomplished by developing detailed procedure specifications, closely supervising the welding operation, and carefully inspecting the welded joints. About 3 percent of the welds are rejected, cut out, and the rails are rewelded.

4.3.5 Summary and Recommendations

Thermite welding of rails in track is an attractive joining process because of its portability, low capital investment requirement, and relatively short time needed for weld completion. On the other hand, thermite welds generally have poorer static and fatigue properties and higher failure rates in service than flash and gas-pressure welds. The poorer performance of thermite welds is attributed to the coarse weld microstructure and to weld defects, which frequently are introduced due to the manual operations and human judgement content of the procedures. The in-track performance of thermite welds is highly variable, however. Because of high failure rates, some railroads do not allow any thermite welding. On the other hand, one railroad reports that thermite welds perform as well as flash-butt welds and several other railroads report that thermite-weld performance is satisfactory. In some cases, reinforcing bars are applied to thermite welds to provide an additional safety factor.

Several important questions raised by this experience are:

(a) Quantitatively what is the likelihood of weld failure if the thermite suppliers' procedural recommendations are rigorously followed?

(b) Which process variables are most critical and must be most carefully controlled?

(c) What effects do departures from suppliers' recommendations have on weld service performance?

(d) What modifications can be made to the process to reduce property variability?

In order to answer the above questions and to improve the performance of thermite welds, the following efforts are recommended for each of the commercially available processes:

(a) Determine the range of weld properties obtained when welds are made in the field by railroad personnel and when made in the laboratory.

(b) Obtain an initial determination of the range over which welding parameters can be varied without seriously impairing weld properties. This would include assessment of procedures for rail preparation, welding, and weld finishing.

(c) Identify variables that are most critical to obtaining sound welds.

(d) Identify and evaluate processing modifications that will reduce welding parameter variability and consistently provide good welds having properties that will give better performance in track.

(e) Develop ways to automatically carry out selected critical procedures or operations that now require manual skills.

Finally, there are a number of German language articles describing investigations of and experience with rail thermite welding.(82,89-96) Making translations of significant foreign language documents more widely available is recommended.

4.4 FLASH WELDING

Flash welding produces coalescence, "simultaneously over the entire area of abutting surfaces, by the heat obtained from resistance to electric current between the two surfaces and by the application of pressure after heating is substantially completed. Flashing and upsetting are accompanied by expulsion of metal from the joint".[97,98]

Although flash butt welding was first used in 1937, the gas-pressure welding process was favored and it was not until 1955 that any significant number of flash-welded rails were placed in service as shown in Table 3.[8,17] The flash-welding process is used predominantly for plant welding of rails although an in-track welding unit has been developed and evaluated.

4.4.1 Flash Welding Procedures[5,8,9,14,77,97-107]

In rail-plant flash welding, the rail ends to be welded are cleaned and then polished at the locations of contact with the current-conducting electrodes on the rail head and base. The rails are positioned in the welding machine and held by both vertically and horizontally acting hydraulic clamps in each platen. The rail ends are inclined so that about 1/16 inch separates the top of the rail head and each end of the bottom surface of a 36-inch-long straight edge that is centered on the joint. This camber is provided to compensate for the greater thermal contraction that occurs in the rail head relative to the web and base during cooling. Electrode clamps in each platen complete the electrical circuit and are separate from the positioning clamps. The rails are aligned by horizontal or vertical movement of the stationary platen that does not provide longitudinal motion. Some newer machines automatically correct for rail twist to provide accurate alignment of the rail head, web, and base.

During the first step of the welding cycle, which sequences automatically, the movable platen brings the rail ends into contact to permit high current flow; e.g. 20,000-100,000 amperes at about 5-10 volts, which preheats the rail ends. The rails are brought together and separated up to 20 times during the preheating stage, which raises the rail ends to a temperature of 1750-2000 F and flashes off rough points on the rail ends. After the rail reaches the proper temperature, flashing is initiated again by bringing the movable platen and rail forward at a controlled and increasing speed. During the flashing period, which removes from 1/4 to about 1 inch from the end of each rail, high spots on the rail ends contact, are rapidly melted, and the molten globules are expelled from the joint. Atmospheric oxygen is virtually excluded from the joint by molten-metal expulsion.

Upon completion of flashing, the movable platen is accelerated so that the rail ends are upset to refusal either with constant platen speed or under impact loading of 60-65 tons. A minimum upset of 0.5 inch is recommended. The welding current is turned off and the electrodes are released from the rail but the rail clamping and upsetting forces are maintained at least 10 seconds while the weld cools sufficiently for safe handling. Monitoring of the welding current, rail movement, and upsetting provide basic quality control.

Depending on the specific flash-welding process used, the hot, upset metal can be removed by a shear either at the welding station or at a separate station, 39 feet or one rail length beyond the welding station. In some new plants, the shear that is beyond the welding station is positioned by sensing the heat in the weld region. The weld then is finish ground either manually or semiautomatically to produce a smooth profile on the top and sides of the rail head and on the underside of the rail base. In some installations, the rail web also is finish ground. Additional stations at rail-length spacings can have facilities for rail straightening and magnetic particle inspection of the completed welds. Flash welds normally are not heat treated. A plant flash weld being made is shown in Figure 11.

Rail-end straightness is a significant problem in plant flash welding because, although the central portion of a rail can be straightened by gagging or rolling, these methods are less effective at the rail end.[6,105] A procedure used by some railroads to avoid the problem of bent rails at the welding plant is to inspect rails for straightness at the steel

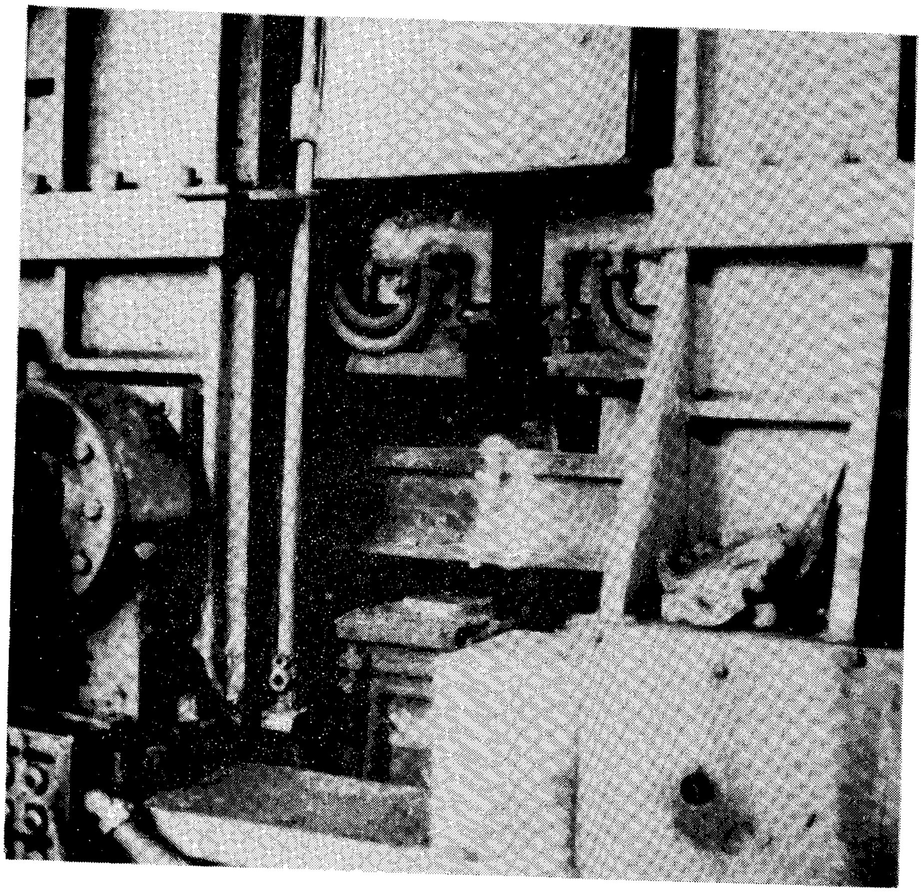


FIGURE 11. PLANT FLASH WELD SHOWING SHEARED UPSET METAL PUSHED OFF OF THE HOT WELD

mill prior to shipment to the welding plant. Even if the rail ends meet the straightness specification of 0.030 inch vertical and horizontal misalignment in the end 4 feet, kinked or out of line welds may be made that have to be straightened in a separate operation or cut out and rewelded. Variations of rail height and head width also are a problem if final rail grinding is not to consume excessive time. Also, if grinding if not done properly, a geometrical stress concentration will be left at the weld.

Relay rail also is welded at flash-welding plants as indicated earlier in Table 8. The welding procedures are essentially the same as for new rail with the addition of dismantling, inspection, and rail-end cropping operations. If the rails have been inspected for internal defects in track, this is not required at the welding plant. Cropping of the rail sections, usually 18 inches from each end, can be done automatically.

In some instances, it is not economical to weld relay rail. For secondary lines having low loadings, up to about 8 million gross tons per year, bolted relay rail strings can be transferred directly from main lines. It has been estimated that this method can save $1500 per mile in comparison with sending the bolted strings through a plant for flash welding.[45]

The reported time required for flash welding is in the range of 1 to 2-1/2 minutes and the welding rate is from about 100 to 160 welds per 8-hour shift.[101,105-107] One railroad, using a contractor at a fixed plant and operating two flash welders each 130 hours per week, produces 2000 acceptable welds per week, 7.7 welds per hour, at a total welding cost of about $16 per weld. This cost does not include the costs of transporting rail from the steel mill to the welding plant and the costs of transporting welded rail strings to the track site. The cost of the fixed rail welding plant recently built by the Santa Fe Railroad at Amarillo, Texas, was reported to cost $7.9 million including $2.7 million for rail loading and unloading units and 4 new rail trains.[101] A contractor's charge for welding rails on a rail train portable plant is $10-$30 depending on the size of the contractor's crew.

4.4.2 In-Track Flash Welding

Flash welding in track has been used in recent years in the U. S., Europe, Japan, Italy, Hungary, and the Soviet Union, where the equipment was developed. This unit, which has been used by several railroads in the United States to make about 33,000 welds since 1972, clamps the rails at the rail webs to achieve electrode contact and to transmit the upsetting force to the rails.(5,77,108-115) In some of the work, where bolted rail was converted to CWR, the rail ends were cropped to eliminate batter, bent down ends, and wear from the joint bars. The rails then were aligned both vertically and laterally. The rail ends were elevated to clear the tie plates and to provide camber so that the rail surface would be flat when the weld cooled. The in-track machine flashes continuously during a 3-minute automatic welding cycle that is completed by upsetting 1/2 inch, but not to refusal, under a 50-ton force. The weld is held in the upset position while it partially cools. If the rail ends do not butt together closely, they can be manually flashed until arcing reaches the full height of the rail ends before the automatic cycle is begun. Upset metal currently is removed from the top and sides of the rail head and from the sides of the rail base. In comparison with plant flash welders, higher operator skill is required, better surface preparation is desirable in order to obtain flashing over the entire joint surface, and the welding time is longer. The in-track welder and a completed weld are shown in Figures 12-14.

This unit costs approximately $500,000 and reportedly is capable of making 8-10 welds per hour and 50-60 welds in an 8-hour shift. In mainline track, with about 6-1/2 hours actual welding time, the rate has averaged 53 welds; a maximum of 86 welds was recently made in 7 hours working time in an 8-hour shift. The total cost per weld would be about $30 for a job requiring 4000 welds. This cost does not include the cost of unspiking, shifting, cropping, and respiking of the rails.

The Soviet in-track flash welder has been used to join long rail strings and to convert conventional bolted rail to continuous welded rail in track. In the latter application, additional

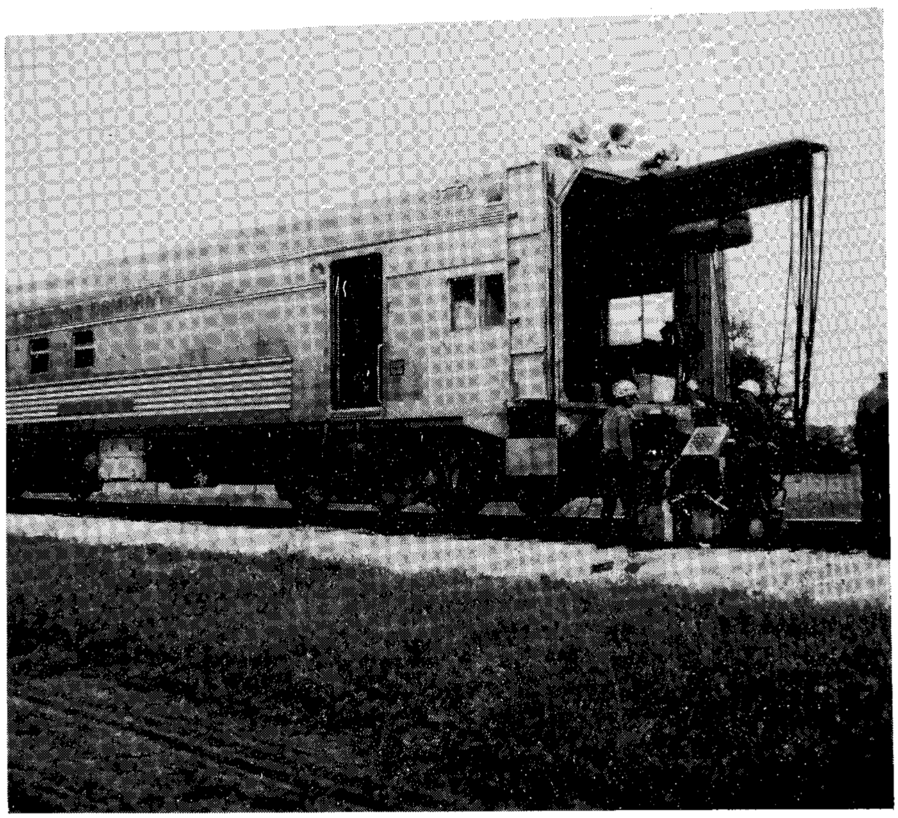


FIGURE 12. RAIL WELDING CAR WITH IN-TRACK FLASH WELDER SET ON RAIL JOINT

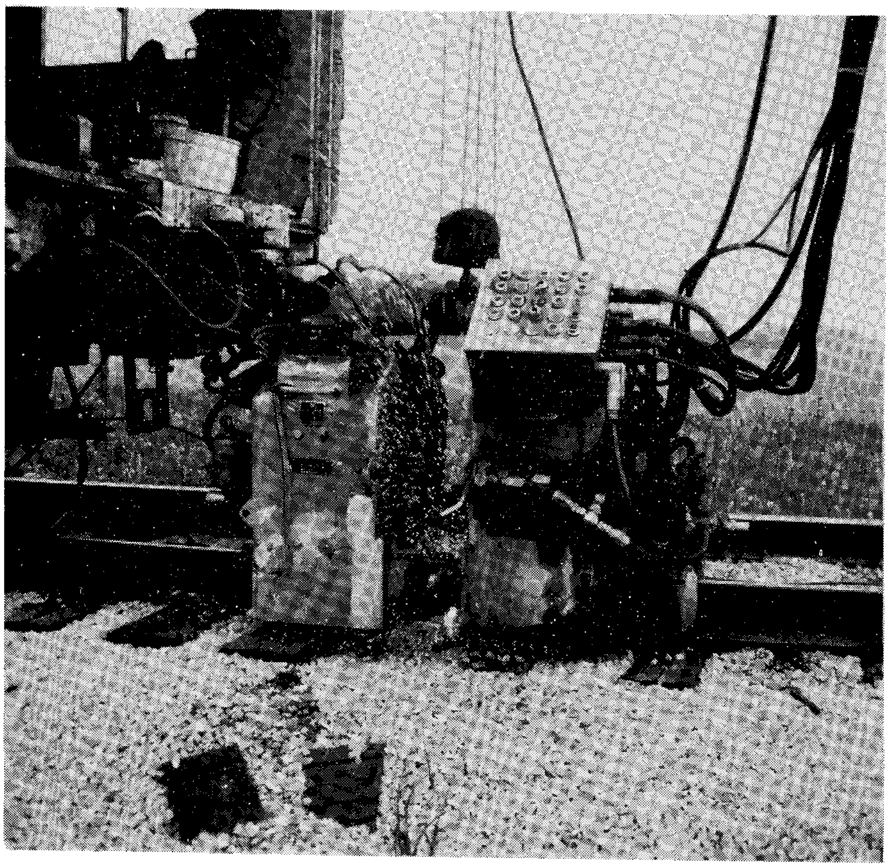


FIGURE 13. IN-TRACK FLASH WELDER DURING WELDING CYCLE

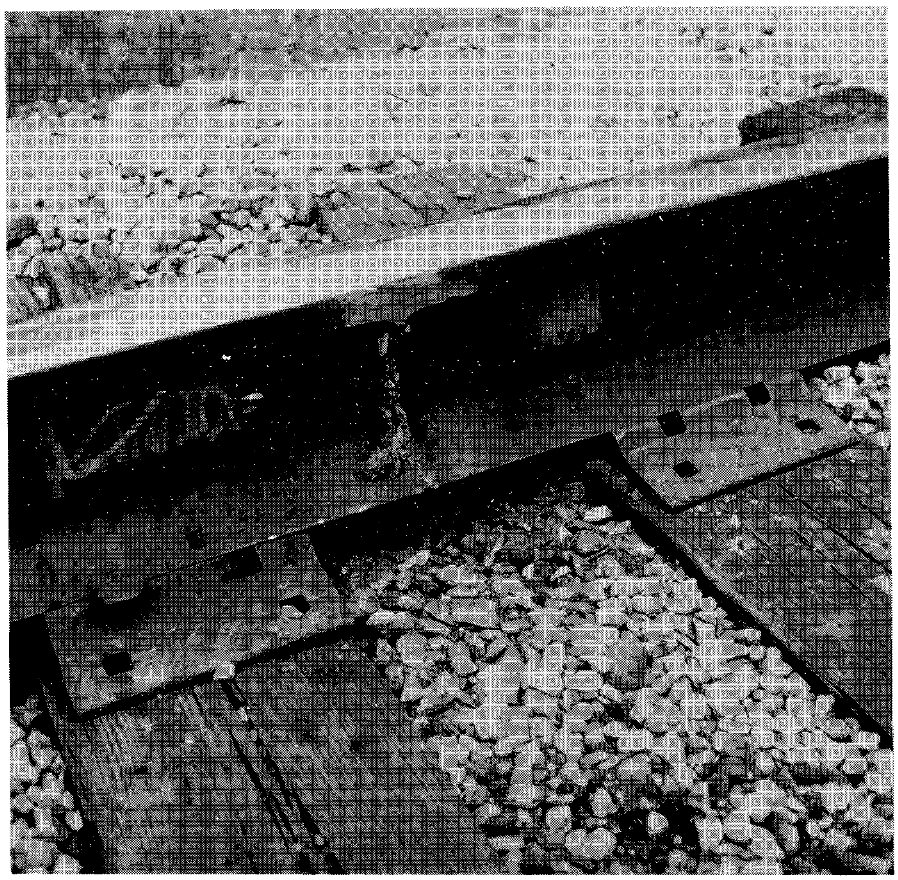


FIGURE 14. COMPLETED IN-TRACK FLASH WELD GROUND ON THE TOP AND SIDES OF THE RAIL HEAD AND EDGE OF RAIL BASE

rail sections are added as the gap between the rails, due to cropping and upsetting losses, reaches a convenient length. This unit also has been used to repair sections of continuous welded rail by removing a short piece and welding in a replacement segment.

In converting bolted, adhesive-bonded, or thermite-welded rail to continuous, flash-welded rail in track, the 2-inch upset required is considered to be a problem. The technique used for this in the Soviet Union first involves cutting out a 10-foot-long piece of rail. A 12-foot-long piece then is welded to the CWR on one end. The CWR on the other side of the short rail is cut so that it overlaps the short rail by 1 inch, unspiked for 200 feet, and bowed laterally so that it contacts the gage side of the opposite rail. During flash welding, 10 men lean against the bowed CWR, tending to straighten it, and force the rail to its final position during the forging stage at the end of the welding process. The rail must be respiked and reanchored after welding. In the time required for these operations, quite possible a thermite weld could have been made.

Assuming that 18 inches are cropped from each end of a 39-foot bolted rail segment and that each weld is made with 2-inch upset, conversion of each track mile from bolted to welded rail requires nearly 300 welds and 24 additional pieces of cropped rail. Welding would require 3-6 eight-hour shifts depending on the amount of on-track time available.

An important consideration in the development and application of any in-track rail joining process is that it must be possible to clear the track for traffic in order to maximize on-track time. Working periods can be extended by scheduling trains through the working area in groups. If the track section to be welded is heavily traveled, however, only a limited increase in on-track time can be obtained and the cost of in-track welding would become relatively high.

4.4.3 Mechanical Properties of Flash-Welded Rails

The results of slow-bend tests of flash-welded rails are given in Table 9. Except for the tests of Japanese and Soviet rail welds, which probably used single-point loading at the center of a 1-meter span, the supports were 4 feet apart and the load was applied at 2 points, 6 inches on each side of the weld that was centered between the supports. Tentative criteria established by the AAR for satisfactory service performance for flash and oxyacetylene gas-pressure welds are a minimum deflection at fracture of 1.5 inches and minimum modulus of rupture of 140,000 psi.[75]

The tests of 131-lb rail show that the flash-welded joint reached a higher load and deflection than an unwelded rail and a higher load but lower deflection than a bolted joint with joint bars. No significant effect of stress relieving on bend properties was found for the 132-lb rail specimens. The lowest values of deflection and modulus of rupture were obtained with a weld having incomplete fusion and a specimen damaged by an electrode burn on the rail base. A number of welded rails sustained 3-5 inches deflection without weld failure.

Rolling-load-test results are given in Table 10. The first test of 131-lb rail was terminated due to crack formation and propagation from a nearby bolt hole. There were no indications of failure at the weld. The second test of 131-lb rail was stopped when failure occurred at another weld, 7 inches away from the weld being tested. At the weld where fracture initiated, the weld flash had been removed from the rail head and down the web to about 1 inch below the rail head. The crack initiated at the top of the remaining flash on the web. Both of these results emphasize the possible harmful effect of stress concentrations in the rail web. In contrast, two 132-lb rail specimens having the flash removed only from the top and sides of the head survived 2,000,000 cycles without failure.

TABLE 9. RESULTS OF SLOW-BEND TESTS OF FLASH-WELDED RAILS

Rail Section, lbs/yd	Max. Load, lbs x 10^{-3}	Max. Defl., in.	Energy for Fracture, ft-lbs x 10^{-3}	Modulus of Rupture, psi x 10^{-3}	Remarks	Ref.
131	300	2.1	100		Rail without weld (a,b)	116
131	201	3.9	111		Joint with joint bars (a,b)	116
131	315	2.5	122		(a,b)	116
132	315	5.0	110	103	Not stress relieved, did not fracture (a,b)	116
132	321	5.3	112	105	Stress relieved, fractured away from weld (a,b)	116
101	232-278	1.2-3.8			Japanese 50 rail	77
133	485	3.4	103.3	158	Fractured on weld line, good grain structure (a,c)	117
133	346	1.1	23.3	112	Fractured on weld line, good grain structure (a,c)	117
136	449	2.0	55.4	143	Fractured on weld line, good grain structure (a,c)	117
140	373	1.1	24.6	117	Fractured on weld line, good grain structure (a,c)	117
140	288	0.5	7.5	90	Fracture initiated at electrode burn-on base (a)	117
100	183	1.0	10.4	92	Incomplete fusion, fractured on weld line (a)	117
100	188	1.1	13.3	95	Fractured away from weld (a)	117
113	322	1.6	40.8	127	Used rail welded without cropping rail ends, fractured at weld line (a)	118
113	383	3.3	80.4	150	Used rail welded without cropping rail ends, fractured at weld line (a)	118
102	244-300	1.4-2.8			Soviet R50 rails, 0.70 or 0.78 percent C, various welding conditions. Ranges for 160 rail welds	119
100	326	5.0	106.6	165	Upset not removed. Soviet in-track welder, no failure (a)	81
100	332	5.0	108.3	168	Upset not removed. Soviet in-track welder, no failure (a)	81
100	299	4.5	88.1	151	Upset removed. Soviet in-track welder. Head failed 7 in. from weld. (a)	81
100	272	3.0	52.0	138	Upset removed. Soviet in-track welder. Head failed 7 in. from weld. (a)	81
133	485	3.4	103.3	156	Fractured on weld line, good grain structure (a,c)	117
133	346	1.1	23.3	111	Fractured on weld line, good grain structure (a,c)	117
136	449	2.0	55.4	143	Fractured on weld line, good grain structure (a,c)	117

TABLE 9. (Continued)

Rail Section, lbs/yd	Max. Load, lbs x 10^{-3}	Max. Defl., in.	Energy for Fracture, ft-lbs x 10^{-3}	Modulus of Rupture, psi x 10^{-3}	Remarks	Ref.
140	373	1.1	24.6	117	Fractured on weld line, good grain structure (a,c)	117
140	288	0.5	7.5	90	Fracture started at electrode burn-on base (a,c)	117
100	183	1.0	10.4	93	Fractured on weld line. Incomplete fusion (a,c,d)	117
100	188	1.1	13.3	95	Fractured away from weld. Good grain structure (a,c,d)	117
140	438	1.6	41.6		Fractured on weld line. Good grain structure. Standard rail (a,c)	78
140	532	4.4	155.4		Did not fracture. Standard rail (a,c)	78
136	467	2.4	67.9		Fractured on weld line. Good grain structure. Flame-hardened rail (a,c)	78
136	305	0.6	8.7		Fractured on weld line. Fracture okay. Flame-hardened high Si rail	78
136	529	4.0	140.0		Flame-hardened rail. Did not fracture (3 specimens) (a,c)	78

(a) 48-inch span and loaded at 2 places 6 inches on each side of weld line

(b) Head down

(c) Base down

(d) French continuous cast rail.

TABLE 10. RESULTS OF ROLLING LOAD TESTS OF FLASH-WELDED RAILS

Rail Section, lbs/yd	Rolling Load Machine Stroke, in.	Wheel Load, lbs	Number of Cycles x 10^{-6}	Remarks	Ref.
131	33	60,000	0.390	Crack initiated at bolt hole near weld. No weld failure at 431,000 cycles.	116
131	33	60,000	0.361	Upset removed except under rail base. Crack initiated at weld stress concentration in web 7 in. away.	116
132	33	60,000	2.0	Upset removed only from top and sides of head. No failure (2 specimens)	116
119	12	51,700	2.0	No failure (2 specimens)	74
132	12	57,500	2.0	No failure (7 specimens), hairline cracks in web	120
132	12	57,500	1.44	Hairline cracks in web. Failed in head-web fillet due to shear drag	120
132	12	57,500	0.957	Hairline cracks in web. Failed in head-web fillet due to shear drag	120
132	12	57,500	1.62	Hairline cracks in web. Failed in head-web fillet due to shear drag	120
132	12	57,500	1.68	Upset removed except on web. Failure intiated in the fillet between the upset metal and web just below head-web fillet.	120
132	12	57,500	2.0	Upset removed except on web. No failure.	120
100	12	40,000	2.0	4 specimens, 2 with upset removed. No failure. Made with Soviet in-track welder	81

Of 10 specimens of 132-lb rail containing hairline cracks in the rail web, 7 completed 2,000,000 cycles without failure and 3 failed by fatigue-crack initiation at the fillet between the head and web due to shear drag introduced during upset removal. One 132-lb sample showed fracture initiation at the fillet between the web and upset metal that was not removed from the web. The location and orientation of the hairline cracks in the rail webs were not reported.

4.4.4 Service Performance of Flash-Welded Rail

On the basis of discussions with railroad personnel as well as weld failure statistics given earlier in Tables 1 and 8, the service performance of flash welds is excellent. The failure rate of 0.0038 service and detected failures per 100 weld years is equivalent to about 1 failure per 100 miles of welded track per year.[2] This failure rate compares favorably with 75.6 rail failures of all types per 100 track miles inspected during 1970, of which 37.5 failures per 100 track miles were web-in-joint failures.

The most common causes cited for flash-weld failures that are associated with the welding process itself, and not the rail-steel quality, are electrode burns on the rail base that form brittle martensite on the rail surface and entrapment of oxidized flash particles in the joint. Weld failures occur less frequently due to insufficient grinding that leaves a stress concentration at the upset, hot tearing by straining the weld before it has cooled sufficiently, and formation of surface martensite by excessive grinding. The most common cause for weld rejection at the welding plant is the formation of surface cracks during weld upsetting in rails containing pipe defects, numerous or large inclusions, and segregation.[78,85-88,117,121-123] A second common cause for flash-weld rejection is misalignment, which was discussed previously. Both of these problems can be eliminated by inspecting the rails at the steel mill prior to shipment to the welding plant. Ultrasonic inspection for pipe defects and large inclusions has been found to be very effective in identifying rejectable "A" rails. Defects have been found to extend into "B", "C", and even "D" rails.

Although failure statistics have not been obtained, the service performance of the in-track flash welds reportedly has been excellent also. The failures that have occurred have initiated most frequently at the upset, stress concentration on the underside of the rail base. To improve the reliability and economy of in-track flash welds, it has been stated by several persons that an automatic shear is needed to remove the hot, upset metal immediately after the weld is completed. This would reduce or eliminate the stress concentration and allow the rail to be pulled over ties. The Soviets have developed and patented a shear[124] which is currently undergoing evaluation in the United States.

It is understood that the Soviet-designed shear, which is installed in the movable platen of the welder, is in the form of a split die that fully surrounds the rail so that all excess metal is cut off including along the underside of the rail base. After welding is complete, the welding electrodes are retracted and the split shear is clamped onto the rail. The platen and shear are pushed longitudinally by hydraulic cylinders and the upset metal is removed about 20 seconds after the weld is completed. A shear has been ordered for installation on the Soviet in-track welder that is in the United States for evaluation and modification as needed for satisfactory performance.

4.4.5 Summary and Recommendations

The inherent advantages of flash welding for rail joining are:

a) Preweld surface preparation is less critical than in other welding processes; the preheating and flashing operations smooth the joint surfaces and expel contaminants in a protective atmosphere.

b) Molten metal is expelled and upsetting eliminates an as-cast weld microstructure.

c) The process can be highly automated to reduce weld-quality variability that accompanies manual operations requiring human judgement.

d) Productivity is higher and the cost and failure rate are lower than for any other welding process.

The problems of rail-end straightness and defects appear to be alleviated effectively by inspecting rails at the steel mills. The evaluation of the shear for the in-track flash welder should be followed because an effective shear will increase productivity and reduce the severity of stress concentrations at the welds.

4.5 GAS PRESSURE WELDING

Gas-pressure welding is defined[125] as a welding process in which "coalescence is produced simultaneously over the entire area of abutting surfaces, by heating with gas flames obtained from the combustion of a fuel gas with oxygen and by the application of pressure, without the use of filler metal". Welding occurs in the solid state by grain growth, grain coalescence, and diffusion across the joint interface.

This process for rail welding was first used in the United States in 1939 and was developed to the extent that welds could be made at a lower cost than that of bolted joints in new rail.[3,8] Because of the longer time required for welding and greater incidence of weld defects, gas-pressure welding largely has been replaced by flash-butt welding in recent years although gas-pressure-welding units are still in service.[17]

In Japan, where 150 and 200 meter-long (492- and 656-foot-long) rails are produced from 25- and 50-meter-long (82- and 164-foot-long) individual rails, both plant gas-pressure and flash welding are used. An on-rail, gas-pressure welding car was built in Japan but was not considered useful due to its low rate of welding. To overcome a shortage of in-track welding capacity, however, this process was re-evaluated and a welding machine weighing 1000 lbs was successfully developed for welding 200-meter-long rails in track.[14,77] Details on the operation of this equipment and the performance of the welds were not available.

An in-track oxyacetylene gas-pressure welder also has been developed in the United States[126,127] but detailed information on its operation and the performance of welded rails has not been obtained. The patents point out that alignment devices permit adjustment to compensate for the tendency of rails to crown or cup during welding and that the system can be used for various rail sizes. An upsetting force of about 20,000 lbs can be applied, which provides a lower compressive stress (2000 psi for 100 RE rail and 1500 psi for 136 RE rail) than normally is used in welding plants (3000 psi).

4.5.1 Gas-Pressure-Welding Procedures[3,4,9,14,77,125-134]

Although the welding procedures used by railroads differ in some details, the following general procedures have been reported.

Rail ends are prepared by butting the rails together, with the joint slightly elevated, clamping them, and sawing the two ends simultaneously to produce smooth and flat joint surfaces. If welding is not performed shortly after rail-end preparation, the surfaces are coated with oil to prevent oxidation. Before welding, the rail ends are cleaned with a suitable solvent, such as carbon tetrachloride. The rails are clamped in the welding machine and an upsetting pressure of 3000 psi over the rail cross-sectional area is applied. Oxyacetylene torches are ignited and oscillated over a 2-inch length to produce uniform heating. Upsetting begins when the rail ends reach about 2000 F. When they reach 2250 F, the rail steel has softened enough for each rail to move 3/8 inch and produce an upset region. The upset on the rail head is removed by a hydraulic shear on some welders and upset on the web and base are partially removed by cutting torches. At another station, when the rail weld temperature is about 900 F, it is reheated over about 6 inches to about 1550 F with oscillating oxyacetylene torches to normalize the weld. The torches above the rail are directed vertically downward and at 45 degrees to the rail-length direction measured in a horizontal plane so that the wheel gradually comes to bear upon the softest zones over a short distance along the rail. When cool, the rail is ground manually and magnetic-particle inspected.

The welding portion of the operation requires 5-10 minutes depending on the rail size[3,4,134] and normalizing, which is not performed at all facilities, takes an additional 5 minutes.[131] The welding and normalizing operations can be performed simultaneously at separate stations. In a large welding program, an experienced welding crew can make 40-50 welds in an 8-hour shift with a welding plant set up at the track site.[131] At a fixed plant using dual welding and normalizing machines and associated equipment, a rate of 15 finished welds per hour or 120 welds in an 8-hour shift, has been achieved.[3]

For a group of 4,078 gas-pressure welds made at the track site in 1952,(135) 4.47 man-hours were required per weld including

a) Equipping flat cars with rollers, etc. (0.04 man-hours)
b) Setting up and dismantling welding equipment (0.08 man-hours)
c) Unloading rail (0.24 man-hours)
d) Sawing and welding operations (3.53 man-hours)
e) Loading welded rail (0.30 man-hours)
f) Employing a watchman (0.28 man-hours).

Materials for the sawing and welding operations ($3.38 per weld) and equipment rental ($1.01 per weld) would cost about three times as much at the present time as in 1952 or $13.20.(10) Assuming an average direct labor rate of $6 per man-hour, the total cost per weld would be about $40. This does not include the cost of shipping the rail to the welding site.

4.5.2 Mechanical Properties of Gas-Pressure-Welded Rails

The results of slow-bend and rolling-load tests conducted at the AAR Research Center are presented in Tables 11 and 12. In comparison with flash welds, the bend-test results are somewhat poorer but the rolling-load-test results are about the same, meeting the 2×10^6 cycle requirement in most cases. These gas-pressure-weld properties generally are better than thermite-weld properties.

4.5.3 Service Performance of Gas-Pressure-Welded Rails

The service performance of gas-pressure welds in rails has been good as indicated in Tables 1 and 8 earlier. The reported failure rate of welds in new rails during 1970 was close to that of flash welds and about one-third of that of gas-pressure welds in relay rails. These gas-pressure welds in new rails were older than the flash welds and the gas-pressure welds in relay rails.

The most common causes of weld failure appear to be lack of fusion due to insufficient cleaning of the rail ends, lack of parallelism

TABLE 11. RESULTS OF SLOW-BEND TESTS OF GAS-PRESSURE-WELDED RAILS

Rail Section, lbs/yd	Max. Load, lbs x 10^{-3}	Max. Defl., in.	Energy for Fracture, ft-lbs x 10^{-3}	Modulus of Rupture, psi x 10^{-3}	Remarks[a,b]	Ref.
132RE	365	1.0	19.6	119	Heat treated rail, partial fusion in web and base, broke in weld	136
132RE	436	1.6	44.2	142	Heat treated rail, broke 5 in. from weld at edge of heat-affected zone	136
136	247	0.4	4.6	79	Broke at weld line, oxidized area at edge of rail base	117
136	395	1.4	33.3	126	Broke away from weld	117
100	158	0.6	5.4	80	French cont. cast rail, broke away from weld line	117
100	164	0.7	7.1	83	French cont. cast rail, broke at weld line	117
112RE	196	0.4	4.3	80.9	Broke away from weld, slag inclusion in upset metal	74
112RE	176	0.3	2.7	72.7	Broke away from weld, slag inclusion in upset metal	74
112RE	292	1.5	27.1	120.5	Rail broke, horizontal break through web in weld	74
112RE	198	0.4	3.9	80.1	Rail broke, horizontal break through web in weld	74
132RE	529	4.7	168.1	172.5	Rail broke, horizontal break through web in weld	74
132RE	487	3.3	102.7	158.8	Rail broke, small portion of base unfused	74
112RE	172	0.3	2.3	71.0	Used rail, used borax flux, no fusion	74
112RE	209	0.5	5.7	86.3	Used rail, used borax flux, no fusion	74
112RE	296	1.5	27.7	122.2	Used rail, no flux, broke in weld, poor fusion	74

(a) 48-inch span and loaded at 2 places 6 inches on each side of weld line

(b) Tests made with rail head up.

TABLE 12. RESULTS OF ROLLING-LOAD TESTS OF GAS-PRESSURE-WELDED RAILS

Rail Section, lbs/yd	Rolling Load Machine Stroke, in.	Wheel Load, lbs	Number of Cycles x 10^{-6}	Remarks	Ref.
132RE	12	60,000	2.0	Heat treated rail, no failure	136
132RE	12	60,000	2.0	Heat treated rail, no failure	136
112RE	12	46,500	2.0	No failure (2 specimens)	74
132RE	12	57,500	2.0	No failure (2 specimens)	74
112RE	12	46,500	2.0	No failure, used borax flux (2 specimens)	75
112RE	12	46,500	1.42	Broke in rail 3 in. from weld	75
100RE	12	40,000	2.0	Hairline cracks in web before welding, no failure (5 specimens)	75,120
100RE	12	40,000	0.9	Hairline cracks in web before welding, fishtail in head, failure originated at fishtail in head and propagaged through weld.	75,120
132RE	12	57,500	2.0	Hairline cracks in web before welding, no failure (6 specimens)	120
112RE	12	47,500	0.033	Used rail, new type welding head, lack of fusion in head	120
112RE	12	47,500	2.0	Used rail, new type welding head, no failure	120
112RE	12	47,500	0.011	Used rail, new type welding head, lack of fusion in head and base, no fusion in web.	120

of the joint surfaces, inclusion of mill scale from the rail surface into the interface, insufficient upsetting, and popout (extinguishing) of the oxyacetylene flame which carburizes the hot joint surfaces by exposure to excess acetylene.[8,78,85,88,122,123] Using an oxidizing flame rather than a neutral oxyacetylene flame, in order to increase the heating rate and reduce the welding time, also can cause lack of fusion by oxidizing the joint surfaces. As with other welding processes, rail defects, hot tearing, and excessive grinding (which overheats the rails and causes brittle martensite formation) also have been the causes of failures. In one instance, a ball of oxidized metal from torch cutting the upset metal adhered to the rail base and provided a stress concentration that was the origin of a failure.[88]

4.5.4 Summary and Recommendations

In comparison with flash welding, gas-pressure welding is slower, more expensive, and more susceptible to weld defects caused by surface contamination. Primarily because of higher welding costs, no new gas-pressure welding units for plant welding are being built and existing units are being replaced by flash welders. However, the development and evaluation of the in-track welder may meet a specific need and should be followed.

4.6 ARC WELDING

In addition to those processes described previously that are widely used for welding rails, there are several arc-welding processes that have been used to a much lesser extent. They include electroslag welding, submerged-arc welding, and "enclosed" welding (a shielded metal-arc welding technique).

The arc welding processes used for rail welding are reviewed in the following sections.

4.6.1 Electroslag Welding

Electroslag welding is a process in which the electrical resistance of a molten slag held in the joint area furnishes the heat necessary for welding.[97,98] The heat generated melts both the filler metal and the adjacent joint walls. When welding rails, a ceramic or water-cooled copper mold, having essentially the same configuration as the rail, is required to hold the molten slag and filler metal in the joint. The weld is accomplished as the filler metal fills the joint volume from the bottom. The process is used mainly for joining heavy sections. Electroslag welding in its standard form has been used most widely in the Soviet Union for the welding of continuous crane rails.[137-139]

In practice, electroslag welding of rails is complicated by the irregular shape of the rail, which makes it difficult to obtain uniform heating without overheating some part of the joint. Also, assembly and dismantling of the mold can be difficult. In spite of these deterrents, successful welds have been made in several sizes of rail. The joints are made without preheating or postheating and require about one man-hour per joint including setup, welding, and finishing.

Electroslag welded crane rails made in the Soviet Union have been shown to have tensile and impact properties equal to or better than those of the unwelded rail. Low-strength, ductile welds with a hardened head area are made by using low-carbon electrode wire for the base and web sections and then adding ferromanganese to the molten metal in the head section. Slightly alloyed welds are made by using different electrode wire compositions and consumable wire guides. There has been no indication that electroslag-welded rails are used anywhere except on crane rails in the Soviet Union.

4.6.2 Combined Submerged-Arc and Electroslag Welding

A hybrid process involving the use of both electroslag and submerged-arc welding to join rails has evolved from studies of arc-welding processes during recent years in Japan. The dual process technique is now referred to as submerged-slag welding.[14,77,140-142] This process, which is still experimental, was developed with the goals of reducing the time required for rail welding and automating the welding process to reduce weld-property variability.

In submerged-arc welding, the heat for welding is provided by an arc between a bare, consumable metal electrode and the workpiece.[97,98] A layer of granular, fusible flux shields the arc and protects the molten weld metal from atmospheric contamination. The flux also can contain alloying and deoxidizing elements.

This process is applied first to weld the rail base and is followed by the electroslag process to weld the rail web and head. The welding current for submerged arc welding is in the range from 800 to 1000 amperes depending on the root gap, which may vary from about 0.6 to 1.0 inch. A copper backing, which has a shallow groove containing solid flux to prevent copper pickup in the weld, shapes the weld-bead reinforcement under the rail.

Welding of the entire rail base up to the start of the web requires three passes. Between welding passes, unfused flux and solidified slag are removed from the joint by chipping and wire brushing. When the submerged-arc welding is finished, the welding nozzle is provided with a consumable tip and a split mold made of water-cooled copper is placed on the rail to hold the molten metal and slag. Then, the remainder of the joint is welded using the electroslag-welding process. It takes about 30 minutes to complete a weld by the submerged-slag process. No preheat or postweld heat treatments are used.

There are many details that must be carefully controlled for the successful production of submerged-slag-welded rails. For example, the electroslag-welding head must be oscillated transverse to the rail in the upper and lower web fillets and rail head as shown in Figure 15 to obtain good penetration and fusion in the transition zones between the web and the head and base. The compositions of the filler metals used were developed on the basis of desired mechanical properties. Wire cuttings, which were placed in the joint between the rail bases to aid arc initiation, were of a different composition than the wire used for the remainder of the weld. These compositions are given in Table 13.

The properties of submerged-slag, rail welds are given in Table 14 along with properties of other types of welds made in Japan with type 50 rails (102-107 lb/yd).[14,77] The composition of the rail steel is 0.60-0.75 C, 0.70-1.10 Mn, 0.10-0.30 Si, 0.035 max P, and 0.040 max S. Because these results were collected from several sources representing tests performed at different times and places, they give a rough comparison of the welding processes.

The bending-fatigue and static-bend properties of submerged-slag welds generally are less than those of flash and gas-pressure welds but greater than those of thermite welds.

Japan National Railways has constructed a car to make field welds by the submerged-slag procedure. This car was used to produce welded rail for two test sites in 1971. Up to the present only favorable results, based on rail-head hardness and profile, have been reported. Refinements of the welding car are being continued by Japan National Railways.

4.6.3 Enclosed-Arc Welding

The enclosed-arc welding procedure was developed to adapt shielded metal-arc welding to butt joints having large cross-sectional

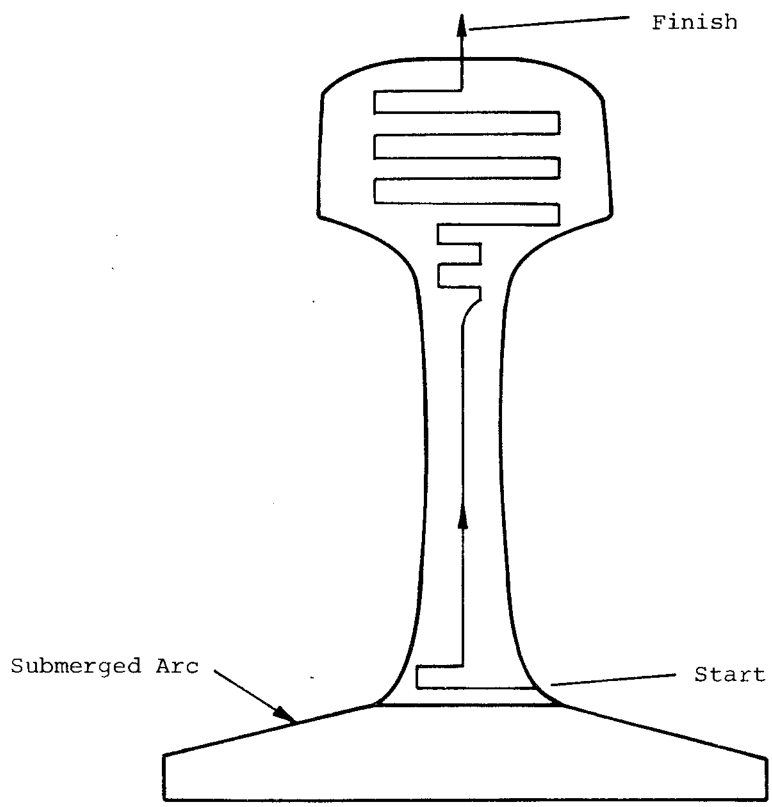


FIGURE 15. WIRE MOVEMENT WHEN ELECTROSLAG WELDING OF RAIL [142]

TABLE 13. COMPOSITIONS OF FILLER MATERIALS FOR SUBMERGED-SLAG WELDING (141)

	C	Si	Mn	P	S	Cu	Cr	Mo
Filler Wire	0.06	0.14	0.86	0.013	0.018	0.10	2.01	0.17
Cut Wire	0.03	0.08	1.76	0.008	0.013	0.09	--	0.48

TABLE 14. SOME RESULTS OF MECHANICAL TESTS OF WELDED RAILS[14]

Property			Flash Weld	Gas-Pressure Weld	Enclosed-Arc Weld	Thermit Weld	Submerged-Slag Weld
			Welding Method				
Fatigue Strength, ksi	Rotating Bending		33-40	41-44		38-44	
	Bending		42-48	48	40	26-31	33
Static Bending Strength[a]*	Bending Strength, lb x 10^{-3}	HU[b]	256-306	267-302	265-300	163-201	243-256
				249-287		218-221	
		HD	218-260	260-289	207-234	187-209	247-262
				220-267		194-218	
	Deflection, in.	HU	1.2-3.8	1.0-3.3	1.1-1.9	0.3-0.4	0.9-1.3
						0.7-0.9	
		HD	0.5-2.5	0.9-3.5	0.6-0.9	0.4	1.1-1.6
						0.4-0.7	
Drop-Weight Strength	Height, ft.	JIS[c]	4.9-16	6.6-11.5			
				3.3-13			
		French[d] Type	9-26 x 5		9-26 x 7	6.6-11.5	
						8.2-13	
	Deflection, in.	JIS	0.3-2.7	0.6-2.1			
		French Type	0.3-2.2	0.2-2.9	0.2-2.0	0.02-0.3	
						0.16-0.4	

* See Footnotes on page 100.

Footnotes to Table 14:

(a) 1.5-m-long rail is supported with a 1-m (39.4 inch) span with the weld in the center. The load is applied with a 5-inch-radius die. The bending strength is the load at fracture and the deflection is that at the center of the span just before fracture.

(b) HU-head up; HD-head down

(c) The span is 36 inches and the tup weighs 2000 lbs and has a tip radius of 5 inches. The specimen is supported by 2 springs.

(d) The span is 19.7 inches, the tup weighs 772 lbs, and no springs are used to support the rail. The height of the tup is stepped up at 0.5-m intervals to 4 m (13.1 feet). If the rail does not fracture, the height is increased to 8 m (26.2 feet) in 1-m intervals. If the rail still does not fracture, the test is repeated at 8 m; 26 x 5 means 5 repetitions at a height of 26 feet (8 m).

areas. As applied to rail joining, it is best described with reference to Figures 16 and 17.[14,77,143,144] The rails are prepared by cleaning and squaring and set with about a 0.7-inch gap. After blocks A, B and B' are placed, the base of the rail is welded by the shielded-metal-arc process after suitable preheating. Japanese practice involves preheating to about 930 F, and it is presumed that others also preheat if the particular application and material require it. After the base has been welded, blocks C and C' are quickly positioned and welding is continued. This is followed by placement of other blocks, first D and D' then E and E' while continuing welding as rapidly as possible. All blocks are placed with about 0.06-inch clearance from the rail. Actual welding time varies with rail size but usually takes less than 1 hour. In Japan, postweld tempering at about 1300 F for 10 minutes follows welding.

Enclosed-arc rail welding has been practiced widely in Europe, England, and in Japan. There are many variations of the same general practice. The success of the procedure depends greatly upon the skill of the welder. The properties of the welds produced are comparable with welds produced in rails by other processes. The Japanese make the comparisons shown in Table 14 for fatigue and bend properties.

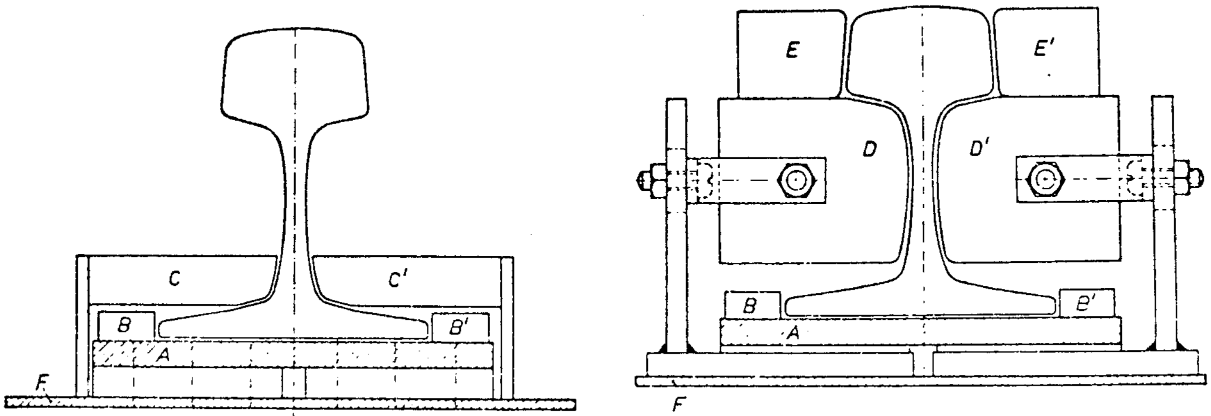
During the past 8 years, the Swedish Railways has developed shielded-metal-arc electrodes and enclosed-arc-welding procedures that result in welds meeting the requirements given below for 50 kg/m (101 lb/yd) rails having a nominal ultimate strength of either 114 or 128 ksi:[145,146]

(1) A fatigue life of 2×10^6 cycles with a load range of 3,300 to 50,000 pounds with the load applied at the weld, centered in a 1-meter span

(2) A minimum bend deflection of 0.79 inch for rails with 128 ksi tensile strength and 0.94 inch for rails with 114 ksi tensile strength with the weld centered in a 1-meter span

(3) The same hardness in the upper part of the weld as in the parent rail

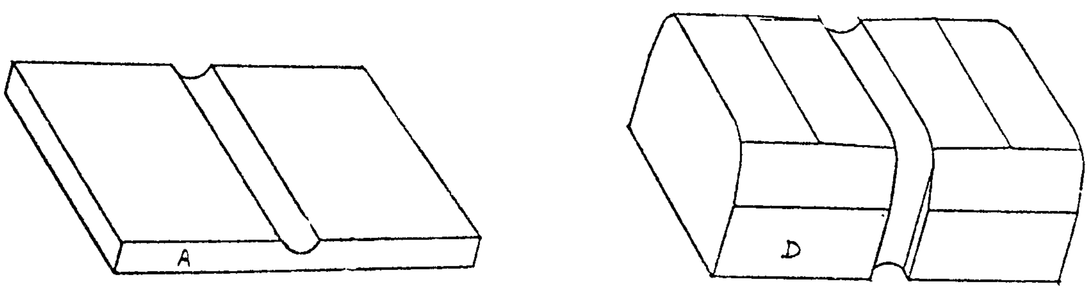
(4) High notch toughness.



Base Welding Setup

Web and Head Welding Setup

FIGURE 16. COPPER ENCLOSING BLOCK ARRANGEMENT FOR ENCLOSED-ARC WELDING OF RAILS (143)



Base Block

Web Block

FIGURE 17. TYPICAL COPPER BLOCK CONFIGURATIONS FOR ENCLOSED-ARC WELDING OF RAILS (144)

The welding electrode developed corresponds to the ASTM-AWS Class E-9018 D1 (American Welding Society Specification A5.5-69) and gives a weld-metal-deposit composition of 0.07C-0.4Si-1.3Mn-0.4Mo with very low S and P. The composition and properties of the higher strength rail and rail weld are given in Table 15.

In the development of welding procedures, a fiberglass-covered, baked-sand-briquette backing was developed to provide a notch-free transition between the weld and the underside of the rail base. The welding procedure, which also is approved by the Danish and Norwegian Railways, is as follows:

a) Cut the rail ends at right angles and bevel the head to facilitate welding the head-web transition and reduce the risk of incomplete penetration in the head.
b) Crown the beveled rails at least 0.08 inch over 40 inches and provide a gap of 0.6-0.7 inch.
c) Electrically connect the rail ends with a split return cable.
d) Preheat the rails to 570-660 F over 5 inches on either side of the joint and maintain an interpass temperature of 660 F for the higher strength steel.
e) Install the backing under the rail base.
f) Using 240-250A with a 0.2-inch-diameter electrode, place a bead across the base of each rail (transverse to the rail length) using the same welding direction. Do not allow the beads to contact each other.
g) Chip and brush the weld beads thoroughly, install a new backing and place a third bead to connect the first two.
h) Complete welding in the rail base. The top pass should blend smoothly onto the top of the rail base and should extend to the rail web.
i) Remove the backup, clean and inspect the rail-base weld, install the 2-piece copper molds on the web and head, and begin welding the web as soon as possible.

TABLE 15. COMPOSITION AND PROPERTIES OF ENCLOSED-ARC RAIL WELDS(146)

Rail Composition: 0.58C-0.33Si-1.30Mn-0.050 max P-0.050 max S

Rail Tensile Strength: 125 ksi

Rail Weld Composition: 0.15C-0.18Si-1.14Mn-0.3Mo

Tensile Properties of Round Bars From the Rail Head:

Yield strength	- 81 ksi
Tensile strength	- 98 ksi
Elongation in 5 x dia. gage length	- 15 percent
Reduction of area	- 60 percent
Notch Toughness (68 F)	- 38-44 ft.-lbs.

j) Complete the web and lower head region with a weaving technique. Maintain a short arc length to prevent porosity formation and change electrodes quickly when needed to prevent the slag from solidifying.

k) Weld the upper 0.25-0.32 inch of the head with an electrode that produces a harder weld metal (0.1C-0.5Si-0.7Mn-3.2Cr all-weld-metal composition). In comparison to a rail hardness of 275-295 HV at a tensile strength of 128 ksi, this electrode gives a weld hardness of about 350 HV which is less than the hardness of the transformed heat-affected zone in the rail (about 370 HV).

l) Remove the molds and coarse grind the rail head.

In order to improve fatigue properties by reducing residual stresses, the weld is heat treated for about 10 minutes or at least until the head, web, and base are at a uniform temperature of 1100-1200 F over 4 inches on each side of the joint. After this, 16-inch-long pads of 2.4- to 3.2-inch-thick mineral wool are pressed against the rail web to produce uniform cooling rates. The rail head and sides of the base then are ground to remove any stress concentrations.

Although the total time required to complete a rail weld was not given, it is estimated to be about 1.5 hours.

A particular variation of enclosed-arc welding for rails is called Secheron welding and has been evaluated extensively in Europe.[147] The technique varies from other procedures mainly in the preparation of the joint. Two configurations of the joint before welding begins are shown in Figure 18. The base is slotted and bent aside as shown. The head may be cut out after the base of the weld is finished (A) or it may be removed before welding starts (B). The notch ductility of a Secheron welded rail is compared to that of a thermite welded rail and an unwelded rail in Figure 19. The higher notch toughness is attributed to the lower weld-metal strength, which is compensated for by deposition of a hard layer at the running surface of the rail. This is analogous

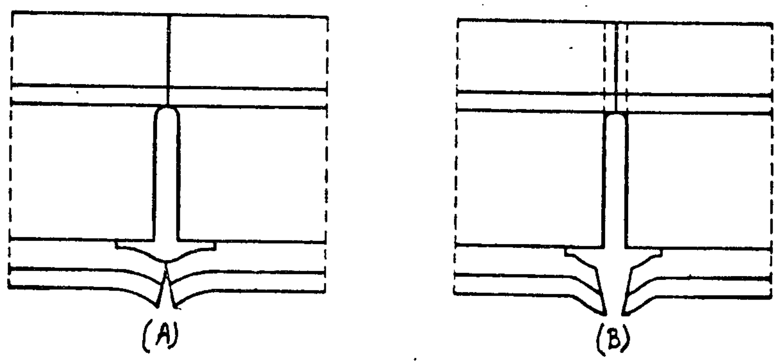


FIGURE 18. SIDE VIEW OF JOINTS AS PREPARED FOR SECHERON WELDING (147)

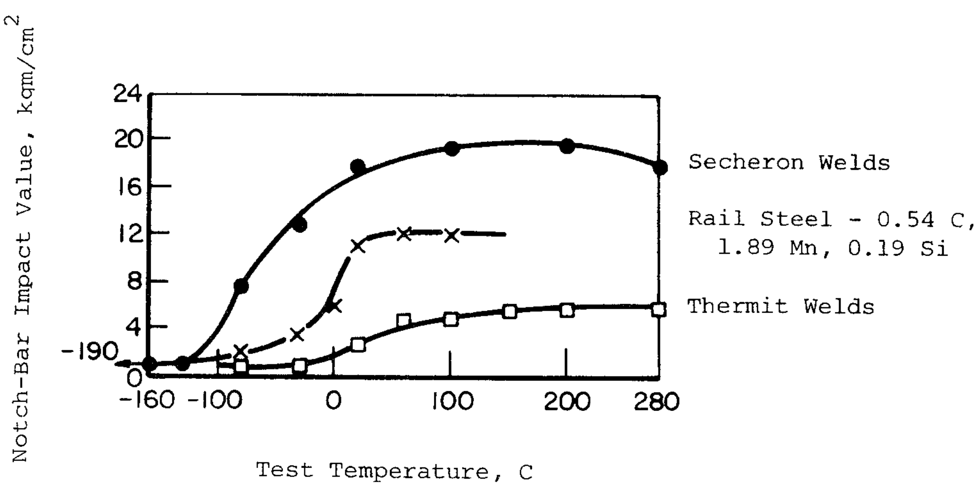


FIGURE 19. NOTCHED-BAR IMPACT DUCTILITY VERSUS TEMPERATURE CURVES FOR SECHERON AND THERMITEWELDS AND ONE RAIL STEEL (147)

to the Soviet technique for electroslag welding described at the beginning of this section and the Swedish technique described above.

4.6.4 Other Arc Welding Processes and Procedures Applied to Rail Welding

The literature is not replete with documents on the use of most other arc-welding processes for butt welding railroad rails. Neither is that which is available very definitive. The few references available are reviewed in the following paragraphs. Most of them refer to rail in applications other than railroads.

An automatic submerged-arc-welding system which was designed to weld in an enclosure of a ceramic mold was developed for welding 175-lb rail for heavy shipyard crane applications.[148] The system seems to very nearly approach electroslag welding. No preweld joint preparation was used but postweld heat treating using flame heaters was incorporated in the system. The equipment was designed for fast welding. Actual welding time for a 175-lb rail was less than 5 minutes. It was reported that the commercial railroads were investigating the usefulness of this system but no reference material indicating this was found.

A short note in the AREA Proceedings comments on a rolling load test of a submerged-arc-welded 110-lb rail.[75] The joint failed after 785,400 cycles in a 12-inch-stroke, rolling-load machine under a 45,500-lb wheel load. Fracture-surface indications, termed "beach marks", indicated that failure initiated in the fillets under the rail head. Under similar test conditions several gas pressure and thermite welded rails did not fail under 2.0×10^6 cycles.

Some European railways produce butt welds in rails for use on secondary railways and tramways by inserting a steel plate in the joint between the rails to be joined.[149] The joint as produced in a tramrail using the shielded-metal-arc welding process is shown in Figure 20. The plate is mild steel but could be an alloy steel chosen for better properties or as a transition material when making a joint

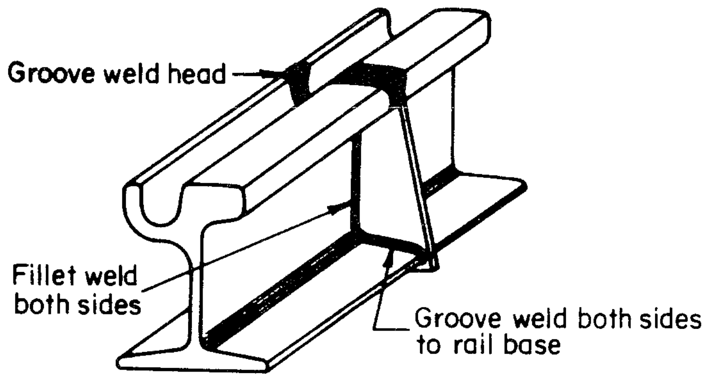


FIGURE 20. RAIL JOINT WITH STEEL PLATE INSERT[149]

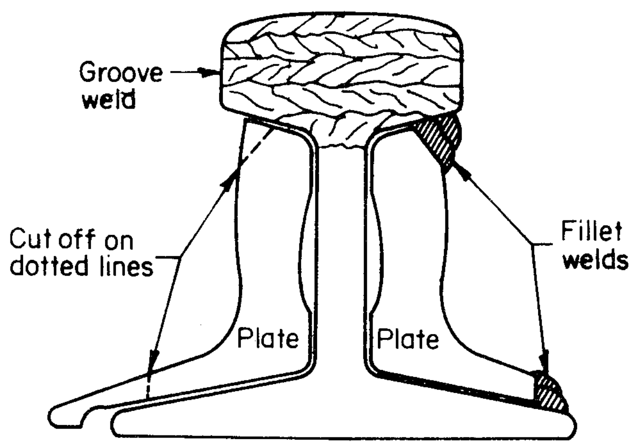


FIGURE 21. CROSS SECTION OF RAIL JOINT WITH WELDED JOINT BARS[150]

with a manganese steel crossover, for example. These joints have shown excellent service life in their intended application.

The patent literature has indicated that considerable effort has gone into producing welded joints in rails by welding the joint bars over the butted rail ends. An example of this procedure was published by Kutuchief.[150] The joint bars were prepared and then shielded-metal-arc welded to the rail as shown in Figure 21. The head of the rail is cut out and the groove filled to join the rail ends. The joint was preheated before welding and the cooling rate was controlled after finishing. A different filler metal is used on the head than on the web and base. This was done in order to get a suitable hardness in the wear surface. This is not a low-cost joint. The welding time was about 1.5 hours and 4.5 hours were required to complete a joint in 120-lb rail.

4.6.5 Summary and Recommendations

Arc-welding processes have been shown to have considerable promise for rail joining. However, there are certain constraints that must be dealt with in any application of arc welding. Briefly these are:

Material constraints

Skill constraints.

Rail steels are high-carbon steels and, therefore, are susceptible to a phenomenon known as underbead cracking. The causes of cracking are discussed in Appendix A. Basically, hydrogen introduced to the weld area during welding is the primary cause. This means that any arc-welding process used on rails must be a low-hydrogen process and that the consumables have to be low in water and other hydrogen-containing materials. It also means that water has to be kept out of the joint region while welding.

Welding of high-carbon steels also can produce hard heat-affected zones having low toughness. Acceptable heat-affected-zone hardnesses can be obtained by using proper welding procedures. Proper procedures usually mean preheat, postweld heat treatment, or high enough heat inputs to reduce the cooling rate and prevent martensite formation. High heat inputs are inherent in some processes (electroslag, for example). Other processes require preheating of the joint or postweld heat treating. Preheating also will aid in preventing underbead cracking.

Arc-welding processes require special operator skills to produce good welds. Mechanization or automation of the welding process is desirable to reduce the degree of skill required. Processes such as submerged-arc and electroslag welding are mechanized processes. Covered-electrode welding (SMAW) and gas-shielded consumable-electrode welding (GMAW) usually are manual processes. Automation would have to include the fixtures and molds required by the rail shape.

Fully automated arc welding does offer opportunity for development of a cost-effective method of producing high-quality rail welds in track. It is recommended that studies be conducted to select an optimum process or combination of processes and to select or develop appropriate equipment, consumables, and welding procedures.

4.7 BRAZING

There has been no experience reported in recent years on the field use of brazing for joining of railroad rails although a laboratory feasibility study was conducted. The feasibility of brazing to join 132RE rails was studied using a conventional gas-pressure welder to apply clamping pressure and to heat the rail ends.[151] The principal goal of the program was to develop a metallurgical joining method that minimized the amount of excess metal that had to be removed such as upset in gas-pressure and flash welding and the casting reinforcement or collar of thermite welds. The rail ends were prepared by sectioning with an abrasive cutoff wheel and cleaning with carbon tetrachloride. Various combinations of electroplated Cu; foils of Cu, brass, Ag-3Li, and a commercial Ni-base brazing alloy; and mild steel were used for filler metal. Torch-tip sizes, gas pressures, heating-zone length, and heating time were established to produce a temperature distribution over the rail section in the range of 1950-2185 F. Because the rail ends did not mate perfectly due to flexing of the cutoff wheel, an upsetting pressure of 1840 psi was used in most experiments to produce 1/4- to 1/2-inch upset and to close gaps at the faying surfaces.

The strongest joint, a solid-state weld, was produced with an 0.030-inch-thick fiber-metal shim only, fracturing in 2-point bending at 220,000 lbs load and maximum tensile fiber stress of 71.8 ksi with the rail head up. The strongest braze was made by Cu plating the rail ends, inserting a fiber-metal shim, and placing 2 pieces of 0.00075-inch-thick Cu foil on each side of the shim. The braze fractured at 117,700 lbs load and 63.9 ksi outer fiber stress in single-point benging. To attain higher bend strengths, it had been recommended that additional studies be conducted using rail-steel fiber shims preimpregnated with Cu. It is not known if the recommended studies were conducted.

Recommendations

The major deterrent to the successful application of brazing to rail joining is the requirement for clean, flat, and reasonably smooth joint surfaces that are in full contact when the rails are properly aligned. The difficulty of adequately and economically doing this, especially in the field, substantially reduces the likelihood of developing a satisfactory brazing process. This problem is similar to that encountered in gas-pressure welding, which includes considerable weld upsetting. For this reason additional effort to develop a brazing method is not recommended.

4.8 FRICTION WELDING

Friction welding is a solid-state-welding process that conventionally relies on pressing a spinning part against a stationary part and producing a solid-state weld by the frictional heating and deformation that are generated.[97,98,152,153]

The process was first developed for joining metals in the USSR in the middle 1950's. With the technique developed there, and used to some extent in the U.S., the rotating part is continuously driven by the power supply. The frictional heat for welding is generated by regulating the angular speed of the rotating part and the axial force. As the metal interface heats to the desired temperature and softens, the workpieces are forged together. A sharp braking action is then applied while the forging pressure is maintained to complete the welding cycle.

The same basic principles apply in an adaptation of the process known as inertia welding, which is commonly used in the U.S. today. In contrast to friction welding, inertia welding uses the kinetic energy of a rotating flywheel that is disconnected from its drive motor. One part is held in a stationary fixture that is designed to resist the high rotational torque and axial thrust forces generated by the process. The other part is held rigidly in a spindle that has a flywheel of predetermined size mounted to it. The spindle is accelerated to impart the desired kinetic energy to the flywheel. The rotating assembly, including the flywheel, is then disconnected from its drive and the two parts to be welded are brought together under a heavy, constant, axial thrust. Frictional forces heat and aid in forging the abutting ends of the parts to complete a weld as the flywheel expends its kinetic energy and comes to rest. During this final stage, the weld actually is completed before the flywheel stops. The remaining flywheel energy plastically deforms the weld and refines the grain structure. No braking action is applied to the rotating part in inertia welding.

The welding time for inertia welding generally ranges from 3 seconds to less than 1 second, compared to 10 to 60 seconds for friction welding. This is because the energy of the flywheel is converted rapidly into heat and the heat is concentrated into a narrow region and is not dissipated into the base metal.

The friction-welded joint is a solid-state weld resulting from mechanical mixing of thin layers of metal on each side of the interface while they are in a plastic state. Little or no melting is produced at the interface even when dissimilar metals with widely different melting points are joined. Any cast structure that is produced is mechanically worked near the end of the welding cycle. The depth of mechanical mixing across the interface has been shown to be about 0.002 inch during inertia welding of some superalloys.[152]

Most of the upsetting and flash production during inertia welding occurs near the end of the welding cycle. As illustrated in Figure 22, high torque is developed just before the flywheel comes to a complete stop. The rotational force developed during this final stage of welding is essential to effective forging in many situations. In cases of large section sizes or difficult-to-forge materials, the forces required would be impractical or impossible to obtain with axial loading alone. The combination of forces produced by rotation and axial thrust results in an adequate force, as shown in Figure 23, and with high energy input (usually 20 to 100 hp/sq. in. of weld area), sound welds are obtained that are free from oxides and voids.

The extrusion of flash during upsetting insures removal of surface contaminants. Without the final upsetting with rotation, defective welds frequently result. The flow lines in friction welds are both radial and circumferential, rather than entirely radial as in forge welds produced by axial upset alone. This is illustrated in Figure 24. The amount of upsetting when inertia welding bars is about 0.1 times the bar diameter. If the amount or shape of external flash is objectionable, it can be removed in a postweld machining operation or on the welder with little increase of cycle time.

Friction welds will not contain gas pockets or slag inclusions that radiographic and ultrasonic inspections can detect. However, magnetic-particle or dye-penetrant procedures will detect defects that extend to an external surface. Deviations from predetermined welding parameters or a desired amount of upset also can be used to determine weld quality.

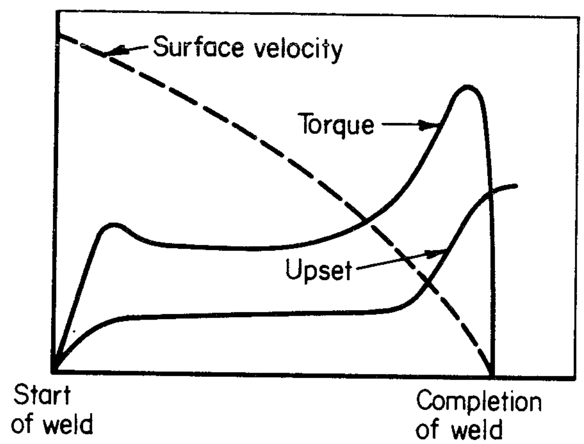


FIGURE 22. TORQUE AND UPSET DISPLACEMENT DURING WELD CYCLE[154]

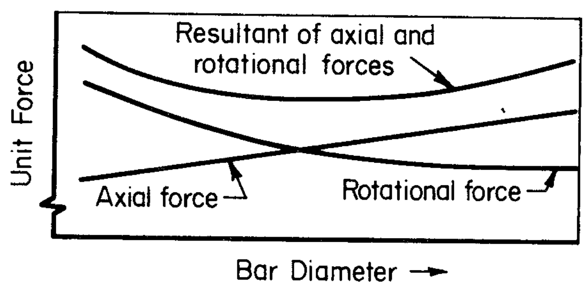


FIGURE 23. FORGING FORCES VS. BAR DIAMETER[154]

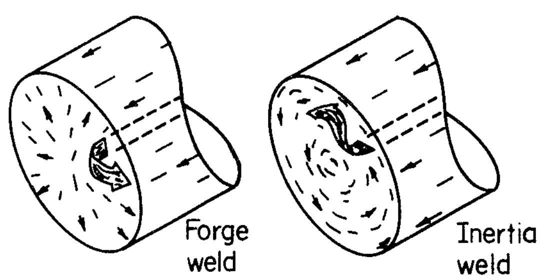


FIGURE 24. FLOW LINE PATTERNS IN FORGE AND INERTIA WELDS[154]

4.8.1 Component Shape

In principle, there are no physical or geometric restrictions on the parts that can be friction welded. However, the process is used most often where one of the parts has a circular weld surface and can be rotated around an axis of symmetry. It is ideally suited for joining solid bars, tubes, studs to plates, caps on containers, and objects of a similar geometry. However, the Arthur D. Little Co. has demonstrated the utility of the technique in joining such shapes as I beams to flat plates as shown in Figure 25.

A wide variety of part sizes have been friction welded. These range in diameter from 0.1-inch drills to 24-inch wheels and in length from less than 1 inch to more than 18 feet. Production machines can accommodate from 1/8- to 4-inch-diameter solid mild steel, or over 30-inch-diameter, thin-walled tubes.

In designing joints, adequate allowance should be made in the length of the parts for upset during welding. This is particularly important when welding dissimilar metals, such as aluminum and steel, where more of the softer material is consumed by upsetting. Sheared, flame-cut, abrasive-cut or sawed joint surfaces can be used. However, with such preparation, it is necessary to have sufficient energy stored in the flywheel to produce full interfacial contact as the surfaces are brought together and to still complete the weld cycle.[124]

4.8.2 Materials

Many similar and dissimilar metal combinations have been successfully joined by friction welding. Practically any material that can be hot worked can be joined, and many dissimilar-metal combinations, including aluminum to steel, have been friction welded. Cast iron, certain brass and bronze combinations, and carbides that have low friction are materials that are difficult to join by friction welding because they do not develop enough frictional heating at the rubbing surfaces. Although sound welds can be made between dissimilar metals with widely varying melting temperature, there are practical limitations of excessive upset and resultant waste of material.

Courtesy of Arthur D. Little, Inc.

FIGURE 25. IRREGULAR SHAPES INERTIA WELDED TO FLAT SURFACES

The rapid quench that occurs after friction welding can produce metal that is too hard to machine. Such welds should be tempered, a process which will also reduce quench-cracking tendencies. Steels with more than 0.35 percent carbon may require tempering. With precipitation-hardenable alloys, postweld aging is required to develop high strength.

4.8.3 Joining Parameters and Results Obtained

There are three principal parameters that determine the characteristics of inertia welds. These are surface velocity, flywheel moment of inertia, and weld-thrust pressure. For any material, component design, and joint geometry, reasonably broad ranges of velocity, moment of inertia, and pressure may be used to produce acceptable welds. Furthermore, once these parameter ranges are established for a given part, the reproducibility of weld characteristics is good. The primary parameter influencing the heating pattern is surface velocity.[152] There is a range of velocities at the start of the welding cycle that will produce the best weld properties and, for example, the preferred range for steel bars extends from 350 ft/min to 1000 ft/min.[153]

The sizes of flywheels, which determine the moment of inertia, are selected to provide the desired kinetic energy and also to produce the desired amount of plastic deformation. The amount of deformation is a function of the flywheel moment of inertia.[153] As shown in Figure 22, torque rises quite rapidly as the surface velocity decreases at the end of the cycle. The increased torque combines with the axial load to cause plastic deformation and upsetting. The amount of deformation is dependent on the remaining kinetic energy of the flywheel, which is a linear function of the flywheel moment of inertia, at a critical velocity of about 200 ft/min when forging begins. Therefore, a large flywheel produces more deformation than a small flywheel because the large flywheel has more remaining kinetic energy when forging begins. The greater deformation produces welds with more flash, for a given axial pressure, and also higher strengths. These welds generally have better quality with fewer oxides and voids than welds made with less deformation.

The high quality of inertia welds in all steels, including tool steels, maraging steels, and stainless steels, has been proven repeatedly by both static and fatigue tests.(153)

One area to be considered in friction welding of steel is heat treatment. When steel bars are heat treated after friction welding, there is no loss in strength relative to unwelded bars.(153) Bars welded after heat treatment maintain most of their strength; any strength losses result from tempering within the heat-affected zone. However, where maximum strengths are desired, heat treatment should follow welding.

Both SAE 1020 (cold rolled) and AISI 4130 (mill normalized) steel have been successfully friction welded with 100 percent joint efficiencies.(154) In the 1020 steel, hardness did not vary appreciably across the weld zone. However, in the 4130 steel, there was a marked transformation hardening in the heat-affected zone with Knoop hardness numbers ranging from about 250 to 600 across the weld. The 4130 steel also required welding parameters that produced a much wider heat-affected zone for successful welds than did the 1020 steel.

Another indication of high friction-weld quality is the attainment of 105-ksi fatigue limit (polished rotating beam specimen) with 8630 steel heat treated to a hardness of R_c 50 after welding. When hardened before welding, the fatigue limit was 80 ksi. The fatigue limit of an unwelded specimen hardened to R_c 50 was 100 ksi.(155)

4.8.4 Friction Welding of Rails

Joining of railroad rails by a modification of the inertia welding process has been evaluated on a laboratory scale at A. D. Little, Inc. in Cambridge, Massachusetts, and Production Technology, Inc. (now Manufacturing Technology, Inc., a subsidiary of Adams Engineering, Inc.)

which manufactures inertia welding equipment. Because it would be impractical to rotate rail sections and to obtain correct rotational alignment of the welded rails, the weld is made by applying upsetting pressure at the joint while rotating a steel disc between the rail ends. Unfortunately, the process details are considered proprietary and the results of evaluations to date with small rail-steel specimens have not been reported.

4.8.5 Summary and Recommendations

Inertia welding appears to be an attractive method for joining rails. The major advantages of inertia welding are:

a) The welding time is short.
b) A narrow heat-affected zone is produced.
c) Energy requirements are low (about one-tenth of that of flash butt welding).
d) A cast fusion zone is not created.
e) Special joint preparation is not necessary and interface surfaces need not be cleaned. Consistent surfaces must be maintained from joint to joint to insure reproducible results, however.
f) The amount of upsetting required is small.
g) Weld properties approaching those of the base metal are obtained.
h) The grain structure in the heat-affected zone is refined, not coarsened.
i) Monitoring of rotational speed, pressure, and amount of upset provide basic quality control.
j) The equipment needed to make weld between sections having a cross-sectional area equivalent to that of 132 lb./yd. rail is relatively light, i.e., approximately 15 tons.

k) If sound welds can be made by rotating a disc between rail ends, it may be possible in some instances to join rails without moving one rail toward the other more than a small fraction of an inch. This would lessen disruption to the track structure accompanying unspiking and moving the rails.

Because of its low energy and low upsetting requirements, the potential for application of inertia welding may be greatest for in-track welding. Further evaluation of this process is recommended. It is particularly important to determine if postweld heat treating is required after welding rail steels.

5. UNCONVENTIONAL WELDING PROCESSES

There are several welding processes that potentially could be used for rail joining. These processes, electron-beam and laser-beam welding, are unconventional in that they are relatively new and have not achieved usage as widespread as most other welding processes.

5.1 ELECTRON-BEAM WELDING

Electron-beam welding takes place by the impingement of a focused beam of high-velocity electrons with the workpiece.[58,98] Although the beam always is generated in high vacuum (10^{-4} torr pressure or lower), the workpiece can be at atmospheric pressure, low vacuum, or high vacuum. The unique characteristic of this process is the high energy density of the beam that permits deep penetration and a small weld width. For example, single-pass, full-penetration welds can readily be made with 25 kw beam power at 4 in/min travel speed (375 kJ/in energy input) in 5-inch-thick steel plate under high-vacuum conditions (10^{-4} torr pressure at the workpiece). In contrast, with the workpiece at atmospheric pressure, the maximum penetration capability in carbon steels is about 1.5 inches at 36 kw beam power and 25 in/min travel speed (86 kJ/in energy input).[156,157] The depth-to-width ratio of electron-beam welds, typically in the range of 5:1 to 25:1, is much greater than the ratio of 1:3 that is typical of many arc-welding processes. Because the fusion zone is narrow, good fitup between the parts to be welded is required; the joint gap should not exceed 0.005 inch unless filler-metal additions are made. In addition, the electron beam must be accurately located on the joint and the parts must not be magnetized, which could cause deflection of the beam off of the joint.

It is believed that rails could be successfully electron-beam welded in track using either the nonvacuum mode with gas shielding of the joint to prevent oxidation, or the low-vacuum mode, at about 0.1

torr, with a small vacuum chamber that would be clamped onto the rails at the joint. Low-vacuum welding is attractive because it permits deeper penetration than nonvacuum welding and requires only a few seconds for pumpdown. Following rail-end preparation and positioning of the electron gun, the actual welding operation would be fully automatic, provided adequate seam tracking and programming equipment could be developed. Welding could be begun at the edge and top surface of the rail base and proceed up the web, around the rail head, and down to the opposite edge of the rail base. It is estimated that welding would take about 3 minutes.

For plant welding, rails could be automatically welded using two or even three electron guns simultaneously in a high-vacuum, low-vacuum, or nonvacuum environment. Low-vacuum welding appears attractive because gas shielding is not required, deep weld penetration is possible, and the production rates can be high. The power requirement for electron-beam welding, 30-50 kW, is much less than that required for flash welding, 100-800 kW. It is not known if nonvacuum electron-beam welding has been applied to high-carbon steels.

The primary welding equipment would cost approximately $200,000 to $250,000 and the welding rate should be equivalent to the rate obtained by flash welding, 12-20 welds per hour.

In the development of electron-beam-welding equipment and procedures, particular consideration must be given to several factors that could impair weld properties. First, it is unlikely that rails could be welded without some regions where partial-penetration segments of the weld intersect. It would be necessary then to determine if welding procedures for rail steels could be developed that would not result in defects, such as porosity and cold shuts, at these locations. It also is important that optimum welding procedures and, in the absence of preheating, postweld heat-treating techniques be developed to prevent excessive hardness and low toughness in the weld fusion zone and heat-affected zone.

5.2 LASER-BEAM WELDING

Laser welds are produced by focusing a high intensity light beam onto the parts to be joined. Like electron beams, laser beams provide a high-energy-density heat source and can make narrow welds.[58,158,159] In contrast to electron beams, however, laser beams are not attenuated during transmission through air or other gases and are not deflected by magnetic fields. They can be reflected with mirrors and can weld parts through a glass window. The major limitations of lasers at the present time are that the maximum penetration is about 0.8 inch at a power level of 20 kW and 50 in/min travel speed (24 kJ/in energy input) and the units that can achieve this penetration (called "continuous wave, gas dynamic" lasers) are bulky and expensive in comparison with electron-beam welders. The developmental efforts for laser welding of rails would be similar to those for electron-beam welding.

5.3 RECOMMENDATIONS

Of these two high-energy-density processes, electron-beam welding has greater potential at the present time for successful development of a rail-welding method. For in-plant rail welding, flash welding is at a more advanced stage of development than electron-beam welding and does not appear to have any serious deficiencies that would be overcome by electron-beam welding. The principal advantage of fusion-welding processes over flash or gas-pressure welding in track is the elimination of the need for upsetting. Automatic arc welding and thermite welding are recommended for further development before electron-beam welding.

6. DISCUSSION

It is evident that, for economical joint fabrication and satisfactory service performance, which implies physical and mechanical properties approaching those of the rail sections being joined, there are several characteristics of the joint and joining method that are important. These characteristics include the following;

a) Production of a composition and structure to provide the required physical and mechanical properties
b) Tolerance for deficiencies in the rail-end preparation including geometry, roughness, and cleanliness
c) Short time requirement for completion in order to maximize production rate of both shop and field joining and minimize track blockage for in-track joining
d) Automation to reduce the variability accompanying manual operations, reduce labor content and skill requirements, reduce the joining time, and, if possible, incorporate in-process monitoring of quality
e) Minimal postjoining thermal or mechanical treatments.

On the basis of these requirements as well as other factors, priorities for future efforts are discussed in the following paragraphs.

The major deficiency of the conventional, bolted rail joint for mainline track service, which accounts for its high failure rate and associated maintenance costs, is its low stiffness and deterioration by deformation, batter, and wear. Because there are many bolted joints in track and because the majority of bolted joints will not be eliminated by the installation of continuous welded rail for several decades, some effort should be directed toward improving the performance of these joints. The specific recommendations were given earlier at the end of the section on mechanical joining (p.42).

However, because of the need to increase the amount of CWR in use, perhaps the highest priority for future development efforts should be given to in-track, rail joining methods, which presently have shortcomings that have been described in some detail in previous sections. Thermite welding, which works well for some railroads but is unacceptable to others, has the greatest potential for development of methods to achieve acceptable and reproducible properties in the near future. In-track flash welding also shows considerable promise and should be developed and refined further, along with thermite welding. Friction, arc, and gas-pressure welding also can contribute to in-track welding capability but, compared with thermite and flash welding, they probably will involve longer development time because there is less experience with the equipment for in-track welding.

In-plant flash welding to produce continuous welded rail has provided joints that more closely meet the requirements given above than any other joining method that has been used.

The special requirement of insulated joints appears to be satisfactorily met by adhesive bonding of the joint. Performance improvements for insulated and standard adhesive-bonded joints can be expected with relatively minor changes in joint design and improvements in mechanical fastening methods as proposed for bolted joints.

Taking the view that improvement in the performance of track structures can be accomplished not only by efforts directly associated with joining methods but also by more indirect activities, several additional actions are recommended.

The production of rails longer than 39 feet reduces the number of joints in track and the number of welds that must be made to fabricate a continuous welded rail string.[160,161] Because processing, handling, storing, and welding of longer rails would have to be modified, the overall economic effects would have to be evaluated. Continuous casting of vacuum degassed steel can be used to produce long rails that would not require control cooling. For example, a 1443-ft-long rail weighing 136 lb/yd could be rolled from a 35-ton bloom having appropriate dimensions, e.g. 9 in by 12 in, to allow a sufficient rolling ratio. Another advantage of continuous casting is that the yield is close to 95 percent compared to about 80 percent for ingot processing.

It also is recommended that, in addition to analyzing failures of rail joints, mechanical-property and metallurgical analyses of welded joints that have satisfactorily carried significant tonnage be conducted Such joints might be removed most economically when the rail is transported to a new track site or to a welding plant. These analyses would contribute to our understanding of features that favor satisfactory performance as well as defects or deficiencies that do not seriously impair performance.

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APPENDIX A

WELDING HIGH-CARBON STEELS

It is desirable in a report on railroad rail welding to review briefly the important metallurgical principles applied to welding of high-carbon steels. This review contains enough information to indicate the significance of these principles and the specific ones most important when welding rail steels. The application of a sound metallurgical understanding to welding will minimize the chances for making an unsatisfactory rail weld.

The changes in the properties of metals that result from welding occur principally because of the temperature excursions caused by the welding operation. During fusion welding, the temperature of the steel ranges from above its melting temperature in and near the joint to the ambient temperature outside of the welding area. In addition, there are a variety of heating and cooling rates. Complex metallurgical changes occur that alter the microstructure and, thereby, the mechanical properties in accordance with known principles. Even though the general principles are known, the specific details needed to predict the final properties of a particular weld are seldom known.

Assuming proper joint design and freedom from defects, all of the mechanical properties of a weld depend on its microstructure and composition. However, the properties vary within a weld because the thermal history and microstructure change with the location within the weld fusion zone or the heat-affected zone. The microstructures also are composition dependent. With knowledge of the effects of thermal history on microstructures and properties, it is possible to select welding processes and procedures that are most apt to produce satisfactory welds. The knowledge needed includes phase transformation and critical temperature data for the rail steel being welded. It also includes knowledge of the effects of rate of cooling, time at temperature, preheating, and postheating on the steel properties. A short discussion of the basic metallurgy involved follows.[A1-A10]

Steel is basically an alloy of iron and carbon. Thus, the first concern is how varying the carbon content alters the melting temperature and phase (microstructural) changes that occur after solidifying. This is shown by the simplified iron-iron carbide equilibrium diagram which is the right half of Figure A1.(A1) The left half of this figure relates the probable microstructure of the weld heat-affected zone to the equilibrium diagram and a typical maximum temperature reached in different areas of the weld.

Austenite and ferrite are each different microstructural forms of the iron-carbon alloys. There are also other microstructures such as pearlite, bainite, and martensite. Martensite is hard and brittle compared to ferrite and is an undesirable constituent unless it is tempered to increase its toughness.

The events that occur at the several points on the peak temperature curve are:

Point 1 has been heated in excess of 2400 F. The austenite that forms will be coarse grained because of the grain growth at this temperature.

Point 2 has been heated to 1800 F and fully austenitized. Grain growth has not occurred and some grain refinement may occur.

Point 3 has been heated to above a critical temperature not high enough to completely homogenize the austenite that has formed from ferrite and cementite.

Point 4 has been heated to approximately 1400 F, which is between two critical temperatures. Part of the structure is converted to austenite and the resulting mixture of products during cooling can result in poor notch toughness.

Point 5 has been heated to 1200 F, which is below the lowest critical temperature, and no austenite has formed. Instead, the metal may be softened.

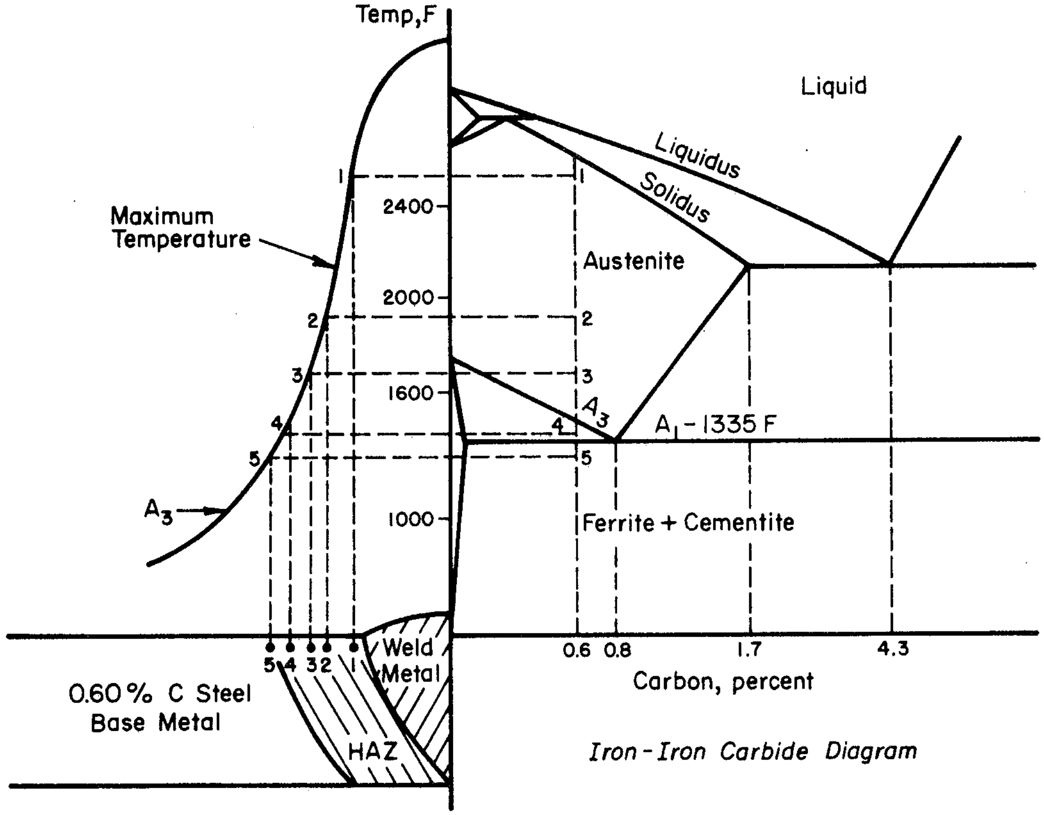


FIGURE A-1. BASE-METAL PEAK TEMPERATURE/WELD HEAT-AFFECTED ZONE/ IRON-CARBON EQUILIBRIUM DIAGRAM CORRELATION (A1)

The several areas in the heat-affected zone can appear as a number of structures with a range of properties. Because the structures that form depend largely on cooling rate, the properties can be controlled during welding. Important effects of welding conditions on cooling are:

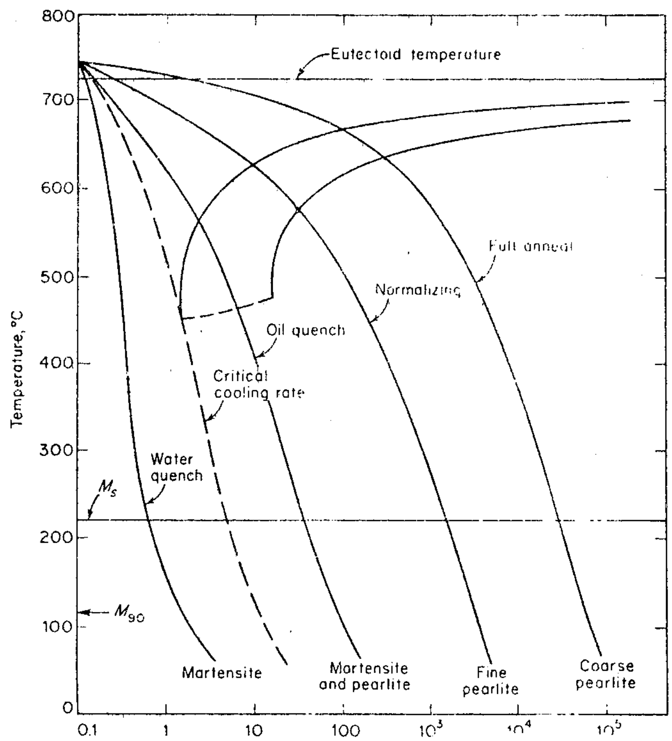
Heat input - Large heat inputs result in slow cooling. High welding speeds tend to reduce heat input and thus increase the cooling rates.

Base metal thickness - Thick sections cool more rapidly than thin sections because the surrounding heat sink is larger.

Base metal temperature, preheat - Preheating the base metal before welding reduces cooling rates.

To indicate the effect of cooling rate on microstructure, the continuous cooling transformation, CCT, diagram has been constructed. This diagram shows curves for the beginning and end of different transformations during continuous cooling on a temperature-versus-time graph. Figure A2 is the CCT diagram for a eutectoid steel (0.8 percent C) and shows the structures associated with various cooling rates. The actual cooling rate accompanying a welding operation depends on the welding process and procedures, heat input, component shape, and initial part temperature. In using these diagrams, it is important to recognize that (1) every alloy has a characteristic CCT diagram, (2) most common alloying elements shift the diagram to the right so that brittle martensitic structures are readily formed even at moderate cooling rates, and (3) cooling can be arrested or the weld can be reheated to produce more desirable microstructures.

The preceding discussion was given to show that a knowledge of the metallurgy of steel is necessary to the successful production of any weldment that involves substantial heating. The metallurgical complexity will vary with the steel and welding process and there usually are several ways to produce the desired properties.



Time, seconds

FIGURE A-2. THE VARIATION OF MICROSTRUCTURE AS A FUNCTION OF COOLING RATE FOR AN EUTECTOID STEEL (A-2)

Steels such as rail steels that range in carbon content between 0.60 and 1.00 percent are called high-carbon steels. They are difficult to weld because they tend to develop cracks in the weld area. Cracking is associated with the formation of martensite and also with absorption of hydrogen. Cracking can be prevented by reducing cooling rates. Hydrogen cracking in fusion welding also is controlled by using techniques and materials suited for keeping hydrogen away from the molten metal. Hydrogen cracking is most frequently encountered with improper arc-welding procedures.

It is evident from the discussion above that welding processes and procedures for welding rails must be used in ways that will minimize the formation of microstructures in and near the joint which contain brittle martensite. Yet in order to fulfill the need for wear resistance, fatigue resistance, and toughness, ways must be found to provide microstructures that are known to have these qualities. This means that welding techniques that provide continuous cooling rates such as indicated by the curve for water quenching in Figure A2, cannot be used unless postwelding heat treatments are acceptable. Rather, continuous cooling rates that are lower than the critical cooling rate or an isothermal treatment to provide a desired microstructure are needed.

There are varying opinions on how best to attain the desired properties when welding high-carbon-steel rail. Practices that employ preheating, low-hydrogen welding conditions, interpass temperature control (for arc welding) and postweld heat treatments seem most suitable. However, experience has shown that rail steels can be welded without either preheating or a postweld heat treatment. Fusion welds can be made without cracks by utilizing high heat inputs, carefully controlled atmosphere protection of the molten metal to avoid hydrogen embrittlement, retarding the cooling rate, multipass welds to get the tempering effect from each pass, and using a final tempering pass. Such practices are safer on steels containing 0.60 percent instead of 0.80

percent carbon as found in some rail steels. A safer procedure for either is to use a postweld heat treatment to lower the heat-affected-zone hardness and raise its ductility and toughness. This practice sometimes is used when welding continuous rail by such means as the resistance flash butt, thermite, submerged-arc, and other welding processes.

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APPENDIX B

ADHESIVE BONDING TECHNIQUES FOR RAIL JOINTS

The use of epoxy adhesives for all the bonding of rail joints can be successful only if proper attention is paid to the following considerations.

B.1 STORAGE OF RESINS

Epoxy resins are subject to deterioration if exposed to heat much in excess of room temperature. If the unmixed resins are left in the sun or hot storage areas, their useable life expectancy is greatly diminished. Manufacturers directions usually indicate a storage life of about six months. This can be significantly shortened if stored at temperature above 80°F.

B.2 MIXING PROPORTIONS

The proportions of resin and hardner are specified by the manufacturer, and are critical. If too much resin is used, the adhesive will remain soft and tacky. If too much hardner is used, curing or set-up time is shortened and excessive heat is generated which will diminish the effectiveness of the adhesive properties.

B.3 MIXING TECHNIQUE

The curing of a resin is a chemical reaction, and it is essential that the resin and hardner be thoroughly mixed so that the two components can interact. Insufficient mixing will result in a soft, tacky bond.

B.4 POT LIFE

The manufacturer usually stipulates the pot life of the resin mixture. This is the time, after mixing that the resin will remain sufficiently fluid to be applied properly. The pot life may vary from only a few minutes to as much as a half hour, and must be used during that time, otherwise the viscosity will increase to the extent that the adhesive will not flow properly and a poor bond will result.

B.5 APPLICATION

The key to a good bond is the preparation of the surfaces to which the adhesive is to be applied. The metal surfaces must be sand-blasted or ground so that all rust and dirt is removed. If necessary, a clean solvent may be used along with dirt or grease to remove any residual sand or grinding grit. Following this treatment, the surface should not be wiped. Any trace of oil or grease will prevent proper bonding. Only a thin coat should be applied, enough to wet both surfaces and fill any crevices. The mating parts are then clamped together tightly.

B.6 TEMPERATURE OF CURE

The cure temperature is also critical. If the bond is made in a cold ambient, the cure will be slow. On the other hand, a hot environment will hasten the cure. Warning, warmth will assist the cure, but if too much heat is applied, the adhesive may shrink or crack. The manufacturers directions should be carefully followed to obtain the optimum bond strength.

APPENDIX C

RAIL JOINING PATENT REVIEW

In order to acquire useful information and to determine the approximate number of patents concerning rail joining, the author of this report visited the U. S. Patent Office and examined the collections in pertinent subject classes. The results of this search are given in Table B-1. Among the total of approximately 4700 patents related to rail joining in Classes 104, 219, and 238 and the Subclasses given in Table C-1, there were about 1400 foreign patents.

TABLE C-1. SUMMARY OF PATENTS IN SELECTED CLASSES

Subject	Class	Subclasses	Approximate Number of Patents
Metal Working	29	470, 470.3, 470.5, 484, 486, 498.5	1250
Railways	104	2, 15 (Track Layers)	470
Metal Founding	164	53 (In Situ Reactive Heating)	150
Electric Heating	219	53, 54, 55 (Rail Bonds)	120
Railways, Surface Track	238	151-153, 158, 159, 162-164, 167, 169, 170, 177, 185, 186, 196, 197, 205-213, 227, 238-244, 257, 258, 260-263 (Rail Joints), 311 (Welded Fastenings)	4100

APPENDIX D

REPORT OF INVENTIONS

This report contains a comprehensive review of reported work on rail joining methods. After a diligent review of the work performed under this contract, it was found that no new inventions, discoveries, or improvements of inventions were made.

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