Test of Welding Technique for Repair of Steel Highway Bridges

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ABSTRACT

A welding technique for the repair of steel highway bridges is described. Recent inspection of steel bridges indicates that there are considerable fatigue cracks, which need some repair work, in bridges that have been in service more than 10 years. The demand for field welding in lieu of bolting has increased. The main problems of field welding are welding under the influence of stresses, welding under the influence of vibration, and rewelding a weld with fatigue cracks. These problems are not common in new bridge construction. A series of laboratory and field tests on plate girder bridges has been conducted to assess the influence of these problems on the welding technique used in field repair. A recommended practice for field weld repair is proposed, and test results showing the effect of stresses, vibrations, and so forth are given. The welding technique proposed here has been successfully applied.

There are approximately 3,700 steel bridges on the Hanshin Expressway, which is 123.6 km long. According to the results of a recent inspection, there was considerable fatigue cracking in bridges that had been in service more than 10 years. This meant that an effective repair method was needed.

Field welding can be more practical than bolting, but there are differences between shop and field welding. For example, it is common practice to keep a bridge open to traffic during repair. This causes traffic-induced vibration and load-induced stresses. The influence of such vibration and stresses on welding was not clearly understood.

In the last 5 years a series of laboratory and field tests of plate girder bridges has been conducted to assess the influence of vibration and stresses on the rewelding of fatigue cracks. A recommended procedure for field weld repair is proposed here, and test results showing the influence of stresses, vibrations, and so forth are presented. The welding technique proposed here has been successfully used in the repair of bridges.

QUESTIONS

The main open questions about weld repair are

For field welding under the influence of stresses:

1. What is the possibility of cracking due to stresses and cold cracking due to restraint of plate?
2. Are there excessive residual stresses due to stresses and restraint of plate?
3. Is there deformation due to welding under stresses and restraint of plate?

For field welding under the influence of traffic-induced vibrations:

1. What is the possibility of hot cracking due to vibrations during weld solidification?
2. What is the effect of vibrations on bead shapes?
3. What is the effect on the strength of joints of welding under the influence of vibrations?

For butt rewelding of fatigue cracks, to what extent is fatigue strength recovered in rewelded joints?

TEST AND MEASUREMENT

<u>General Considerations</u>

Tests of field weld repairs of plate girders include

1. Fillet welding tests with tensile-stressed specimens,
2. Vibration measurement of actual bridges,
3. Fillet welding tests of vibrating specimens, and
4. Fatigue tests of original and repaired butt weld joints.

The chemical composition and mechanical properties of test steel SM50 and SM58Q, specified in the Japanese Industrial Standard (JIS), are shown in Tables 1 and 2, respectively. SM50 steel is equiva-

TABLE 1 Chemical Composition of Test Steel

steel grade	Thickness (mm)	Chemical composition %				
		C	Si	Mn	P	S
JIS SM50 (ASTM A572 Grade 50)	9	0.14	0.40	1.29	0.020	0.004
	30	0.18	0.41	1.37	0.020	0.015
JIS SM58Q (ASTM A678 Grade B)	9	0.14	0.34	1.30	0.014	0.003
	30	0.12	0.29	1.31	0.011	0.003
	40	0.13	0.29	1.28	0.018	0.003

JIS : Japanese Industrial Standard
ASTM : American Society for Testing and Material

TABLE 2 Mechanical Properties of Test Steel

Steel grade	Thickness (mm)	Mechanical properties			Allowable tensile stress (kgf/cm^2)
		Yield strength (kgf/mm^2)	Tensile strength (kgf/mm^2)	Elongation (%)	
SM50 (ASTM A572 Grade 50)	9	42 (≧33)	56 (50~62)	25 (≧17)	2,100
	30	41 (≧32)	58 (50~62)	27 (≧21)	
SM58Q (ASTM A678 Grade B)	9	58 (≧47)	63 (58~73)	27 (≧19)	2,600
	30	55 (≧46)	63 (58~73)	29 (≧26)	
	40	58 (≧46)	66 (58~73)	30 (≧26)	

( ) : JIS provision

lent to ASTM A572 grade 50, and SM58Q corresponds to ASTM A678 grade B. Electrodes for fillet welding are the super low hydrogen type, LBM-52 (JIS D 5016, equivalent to AWS E 7610), and super low hydrogen-low strength type, LB-47A (JIS D 4316), which were selected based on nonpreheated weld metal cracking tests. For butt welding, an ultra low hydrogen electrode, LB-62UL (JIS D 5816, equivalent to ASW E 9016-G), which has strength equivalent to that of the test steel, was used.

Fillet Welding Test of Tensile-Stressed Specimens

Welding tests of thirty specimens were carried out in the laboratory to examine the effects of tensile stresses and restraints transferred from the surrounding plate on cracks, residual stresses, and welding deformation. Test specimens and conditions are shown in Figure 1 and Table 3, respectively. Tensile stresses are $0.8\sigma_{all}$ for web plates (thickness, t = 9 mm), and σ_{all} for flange plates (t = 30 mm), where σ_{all} is allowable tensile stress. In the tests, constant stress was applied during welding; then deformation was kept constant by controlling the gauge length for 48 hours after welding, and finally stress and restraint of deformation were released.

TABLE 3 Test of Welding Under Stress

Specimen	Direction of weld line	Steel grade	Thickness t(mm)	Stress (kg/mm²)	Gauge length (mm)	Welding position	Electrode
V-1	Perpendicular	SM50	9	$\sigma_{all} \times 0.5$	400	Horizontal	(4ø) LBM-52
V-2	Ditto	SM50	9	$\sigma_{all} \times 0.8$	Ditto	Ditto	(4ø) LBM-52
V-3	Ditto	SM50	30	σ_{all}	Ditto	Ditto	(4ø) LB-47A
V-4	Ditto	SM58Q	9	$\sigma_{all} \times 0.8$	Ditto	Ditto	(4ø) LB-47A
V-5	Ditto	SM58Q	30	σ_{all}	Ditto	Ditto	(4ø) LB-47A
P-1	Parallel	SM50	9	$\sigma_{all} \times 0.8$	Ditto	Ditto	(4ø) LBM-52
P-2	Ditto	SM50	30	σ_{all}	Ditto	Ditto	(4ø) LB-47A
P-3	Ditto	SM58Q	9	$\sigma_{all} \times 0.8$	Ditto	Ditto	(4ø) LB-47A
P-4	Ditto	SM58Q	30	σ_{all}	Ditto	Ditto	(4ø) LB-47A
P-5	Ditto	SM50	9	$\sigma_{all} \times 0.5$	Ditto	Ditto	(4ø) LBM-52

Welding defects were sought using X-ray, magnetic particle, and microstructure and macrostructure tests in accordance with JIS. Welding residual stresses, $(S_x, S_y)_{RS}$, and maximum stresses, $(S_y)_{max}$, were measured by two-axis strain gauges. The maximum stress $[(S_y)_{max}]$ can also be expressed as

$$(S_y)_{max} = (S_y)_{TRC} + (S_y)_{RRC} \quad (1)$$

where $(S_y)_{TRC}$ is the stress due to constant applied load and welding, and $(S_y)_{RRC}$ is the stress increment at a constant gauge length. Deformation, or change in length, due to welding was measured using a gauge length of 40 cm that was marked on the specimens.

Vibration Measurement of Bridges

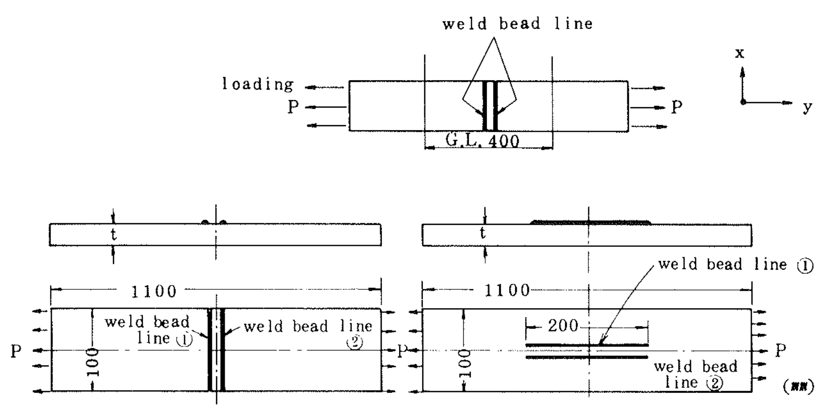
Before the test of fillet welding under the influence of vibration was conducted, field measurements of traffic-induced vibrations of main girders in three plate girder bridges were carried out to obtain basic data for laboratory tests. Span lengths of these bridges are 22.4 m, 34.3 m, and 37.4 m. Both vertical and horizontal vibration responses were measured in the frequency range of 0.3 to 300 Hz. The measured points were on the support, the flange plate, and the web plate (top, center, and bottom) of the main girder at midspan. The main girder vibrated irregularly in both the horizontal and the vertical direction.

The results of the frequency analysis of the measured horizontal and vertical irregular vibrations are shown in Figure 2. This figure depicts the relationship between mean and deviation of horizontal and vertical displacement ranges and frequencies.

Fillet Welding Tests Under the Influence of Vibration

The effect of vibration on bead shape and other weld defects were examined using specimens (Figure 3) that were connected by fillet welding under vibrating conditions.

The specimens are of SM50 and SM58Q steel with a thickness of 9 mm for the web plates and 30 mm for the flange plates of the plate girders. Frequencies in the tests were 0.3, 3, 30, 90, 150, and 300 Hz, and displacement ranges were Vs, 5 Vs, and 10 Vs. Vs is the mean value of the displacement ranges shown in Figure 2. In-plane or out-of-plane vibrations are applied to the specimens at each frequency.



(a) Perpendicular welding to stress axis (b) Parallel welding to stress axis

FIGURE 1 Specimens for test of welding under stress.

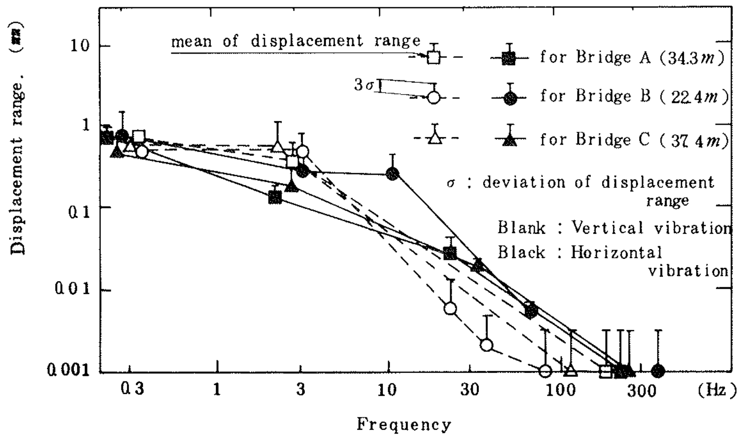


FIGURE 2 Results of frequency analysis of measured vibration.

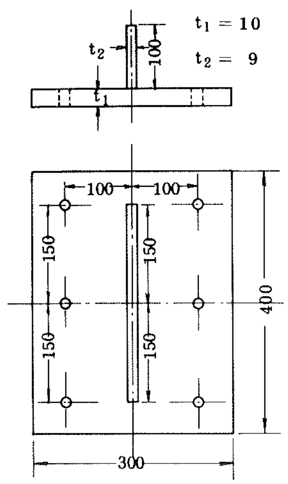


FIGURE 3 Specimen for test of fillet welding under vibration.

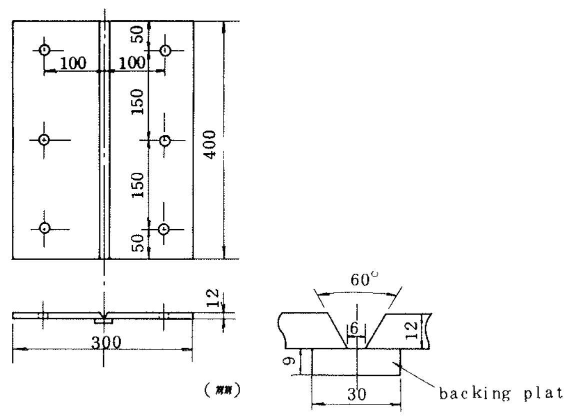


FIGURE 4 Specimen for test of groove welding under vibration.

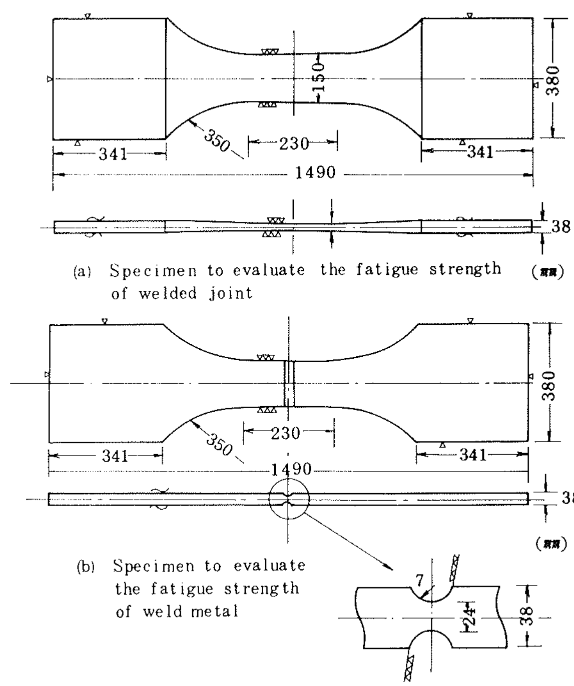


FIGURE 5 Specimens for test of fatigue strength of welded joint.

Vertical up-welding, vertical down-welding, and overhead welding were used on the specimens to examine the effect of directions and positions of welding. Welding defects were sought and inspected using the methods mentioned earlier.

In addition to the fillet welding test, a groove welding test was carried out to examine the influence of vibration on the strength of welded joints (Figure 4). The electrodes used in this test were LB-47A and LBM-52.

Fatigue Strength Test of Repaired Weld Joint

Fatigue cracks often occur in the weld zone of steel bridges. Fatigue tests were conducted on four specimens (Figure 5). Two specimens were used for the original weld joint and weld metal tests, and two were used for the rewelded joint and reweld metal tests. Each specimen was of SM58Q steel 40 mm thick and was connected by submerged arc welding at the center. For the rewelding tests, two specimens, which had received proper gouging treatment after some fatigue damage, were manually welded with an LB-62UL electrode. The number of stress cycles in this test is shown in Table 4.

TABLE 4 Stress Range and Number of Cycles

Test specimen	Stress range (kgf/mm^2)	Number of cycles n	Fatigue damage n/Nf
re-welded joint	40	2.0×10^5	0.30
re-weld metal	35	1.0×10^5	0.34

Nf : Number of cycles to failure

TEST RESULTS

Fillet Welding with Tensile-Stressed Specimens

No weld cracks occurred, but a few pinholes, detected by magnetic particle test, and a few blowholes at the weld toe were found. Maximum stresses, $(S_y)_{max}$, in the load axis were almost equal to yield stress of the base metal for both perpendicular and parallel weld cases.

Residual stress, $(S_y)_{max}$, in the load axis is shown in Figure 6. These values are lower than 8 kgf/mm² for both cases. The deformations, due to the influence of welding and tensile stress, $0.8\sigma_{all}$ and σ_{all}, are less than 0.87 mm for a single weld line (Table 5).

Test of Fillet Welding Under the Influence of Vibration

In Figure 7 the results of visual examination are shown. Undercuts, leg length, and throat depth of the fillet weld for both vertical up- and vertical down-welding conditions are shown. Unacceptable defects are indicated by a black portion of a circle.

The interpretation of the results of the visual examination is based on the provisions of Japan Road Association (JRA) specifications (1). As shown in Figure 7, poor bead shapes, especially on the undercuts, occurred at a frequency of 3 Hz for vertical up-welding. The effect of in-plane vibration on bead shape is more significant than that of out-of-plane vibration. It is assumed that the specimen vibrates with large displacement amplitude at this frequency and that weaving is difficult for in-plane vibration. However, the vibration did not affect the leg length or the throat depth of the fillet welding. In the case of the vertical down-welding test conducted only for in-plane vibrations, the bead shapes were in the acceptable range. In the overhead welding condition, there were undercuts at a frequency of 3 Hz.

Blowholes were detected by the X-ray test. Results of the evaluation of vertical up-welding are shown in Figure 8. The figure shows blowholes at a frequency of 3 Hz and 5 Vs or more displacement range. No blowholes were observed in the case of vertical down-welding.

In Figure 9 typical photographs of the weld zone are shown for different test conditions. The first crystal structures with round shapes occur at vibration frequencies of 30 Hz or more.

Figure 10 shows the relationship between the tensile strength of the joint connected by groove weld-

TABLE 5 Results of Test of Welding Under Stress

Specimen	Steel grade	Electrode	Thickness	Load (ton) (Stress) (kgf/mm^2) Initiation	Maximum	Deformation (mm) Weld Line (1)	(2)	Direction of weld line	Crack
V-1-C	SM50	LBM-52	9	8.5 (9.4)	21.7 (24.1)	0.17	0.40	Perpendicular	NO
V-2-C	SM50	LBM-52	9	13.7 (15.2)	25.8 (28.7)	0.31	0.68	Ditto	Ditto
V-3-C	SM50	LB-47A	30	57.3 (19.1)	82.8 (27.6)	0.37	0.55	Ditto	Ditto
V-4-C	SM58Q	LB-47A	9	18.1 (20.1)	33.6 (37.3)	0.36	0.73	Ditto	Ditto
V-5-C	SM58Q	LB-47A	30	78.2 (26.1)	105.0 (35.0)	0.51	0.71	Ditto	Ditto
P-1-C	SM50	LBM-52	9	13.3 (14.8)	334.2 (38.0)	0.26	0.75	Parallel	Ditto
P-2-C	SM50	LB-47A	30	57.3 (19.1)	90.0 (30.0)	0.36	0.55	Ditto	Ditto
P-3-C	SM58Q	LB-47A	9	18.2 (20.2)	445.0 (50.0)	0.36	0.87	Ditto	Ditto
P-4-C	SM58Q	LB-47A	30	78.0 (26.0)	116.7 (38.9)	0.51	0.70	Ditto	Ditto

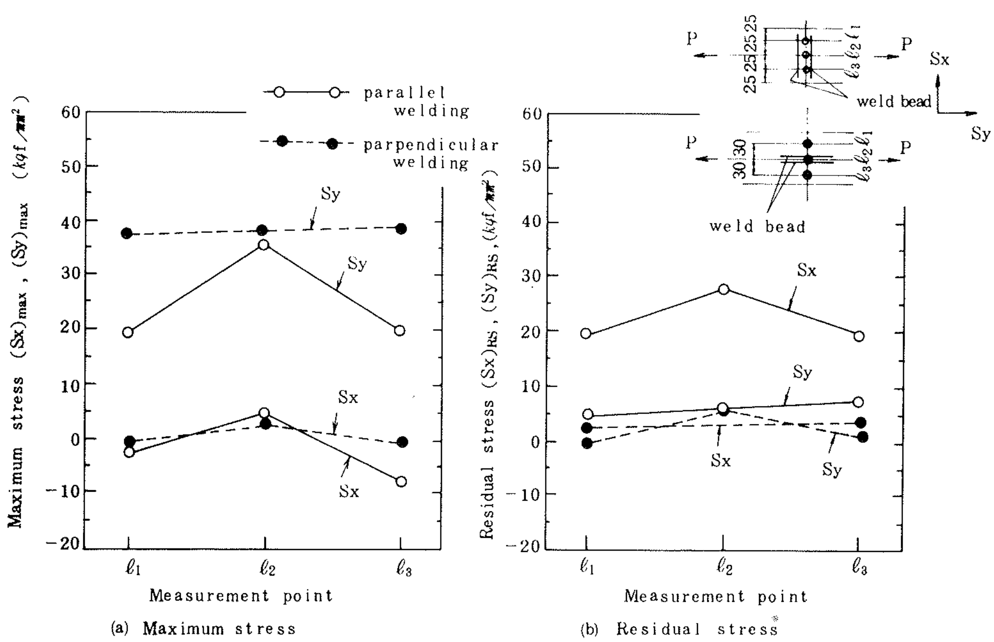


FIGURE 6 Maximum stress and residual stress of weld zone under stress.

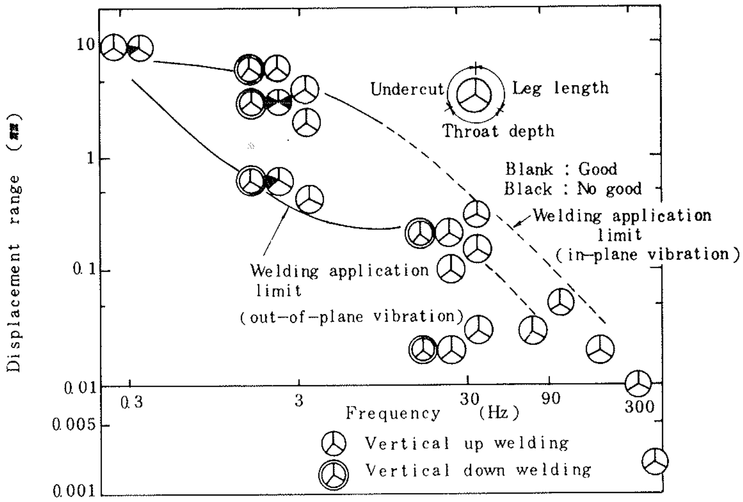


FIGURE 7 Visual examination results of test of welding under vibration.

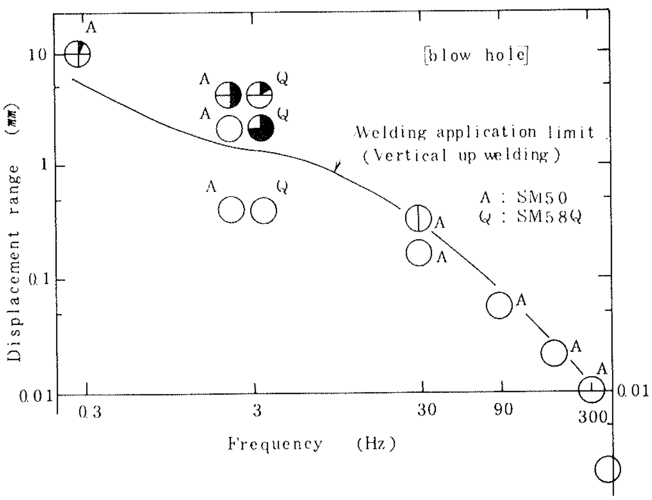


FIGURE 8 X-ray test result for welding under out-of-plane vibration.

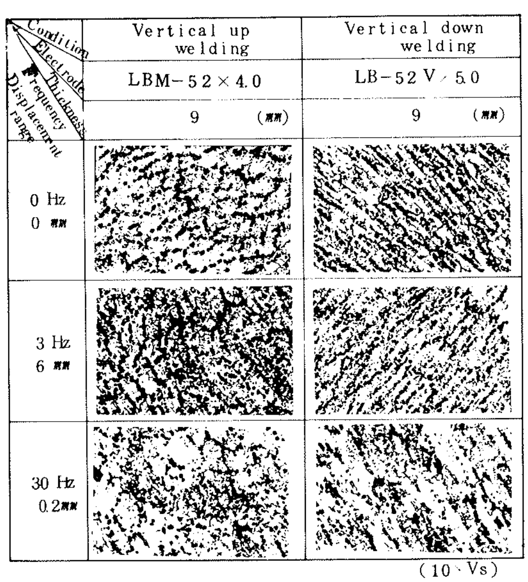


FIGURE 9 Microstructures of weld metal under vibration.

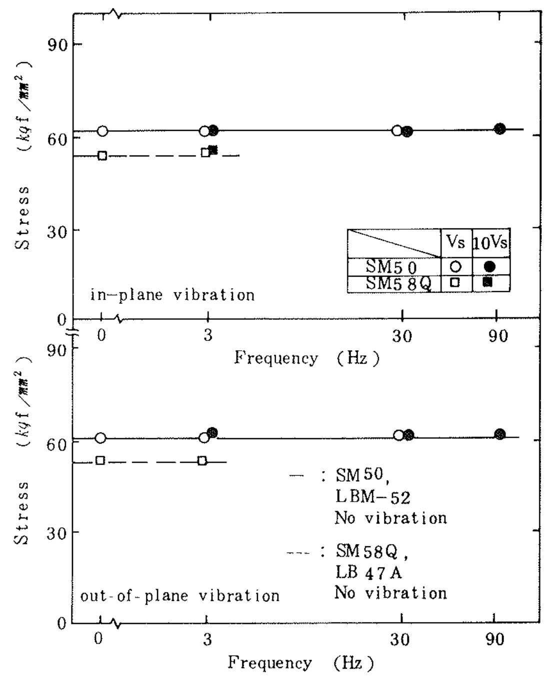


FIGURE 10 Tensile strength of groove welding joints under vibration.

ing and the frequencies of the vibration. The tensile strength of the welded joints is not influenced by vibration. There is a slight decrease in charpy impact at 3 Hz and 10 Vs vibration. The tensile strength of the joint welded with a super low hydrogen-low strength electrode, LB-47A, was lower than that of base metal, SM58Q.

In addition to the laboratory tests, field tests were carried out on a plate girder bridge with traffic-induced vibration. Three fillet test specimens (Figure 3) were welded to the plate girder. In spite of slightly poor bead shapes, weld defects were not detected and the strength of those fillet welded joints was in the acceptable range.

Fatigue Strength Test of Repaired Weld Joints

Stress number (S-N) lines for the fatigue strength test are shown in Figure 11. This figure illus-

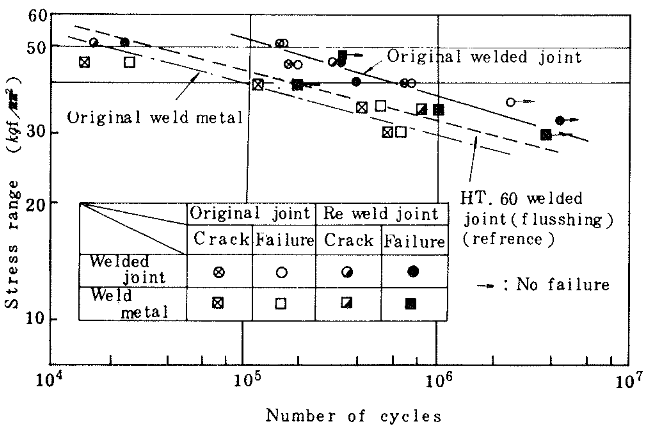


FIGURE 11 Fatigue strength test results.

trates the relationship among the applied stress range, $\Delta\sigma$, the number of cycles at the initiation of the crack, Nc, and the number of cycles at failure, Nf. Two straight S-N lines for the original welded joints and the weld metal can be approximately expressed as

$$Nf \cdot \Delta\sigma^{7.74} = 1.65 \times 10^{18} \quad (2)$$

$$Nf \cdot \Delta\sigma^{7.92} = 4.96 \times 10^{17} \quad (3)$$

The fatigue strength of the rewelded joint is almost equal to that of the original welded specimen, except in the case of the specimen for $\Delta\sigma$ = 50 kgf/mm². In the case of $\Delta\sigma$ = 50 kgf/mm², the fatigue strength was decreased by a weld defect (4 mm x 1 mm slag inclusion). The fatigue strength of repaired weld metal specimens is not lower than that of the original weld metal specimens.

SUMMARY AND DISCUSSION

Fillet Welding Under the Influence of Tensile Stresses

The influence of low tensile stresses on weld defects is almost negligible. Maximum stresses, $(S_y)_{max}$, approach the yield stress of the base metal, and the maximum residual stress is lower than 8 kgf/mm². Elongation of welding deformation is affected by tensile stresses, but deformation of this magnitude is not thought to cause problems in practical use. It is concluded from these results that fillet welding is applicable to repair work done under the influence of tensile stresses.

Fillet Welding Under the Influence of Compressive Stresses

Although a test for this condition was not conducted, it is assumed that welding deformation under compressive stresses will be magnified by shrinkages during the heat input and solidification process. Such deformation and welding heat will decrease the buckling strength of members under compressive stresses. These problems are being studied.

Fillet Welding Under the Influence of Vibration

For the web plate of the plate girder, vertical up- and vertical down-welding tests were conducted. In the case of vertical up-welding, blowholes that lower the strength of joints did not occur within the vibration range of 5 Vs, although bead shapes, especially undercuts, were slightly affected by vibration. In the case of vertical down-welding, bead shapes and blowholes were in acceptable ranges. The application limit of vertical welding is shown in Figures 7 and 8.

It is concluded that fillet welding can be used to repair web plates under the influence of vibration if visual examinations and the correction of defects are done carefully.

In the test of the fillet welding on the flange plate, blowholes were not detected although the possibility of undercuts exists. Fillet welding can be used in the repair of flanges under the influence of vibrations if visual examination and correction of defects are done carefully. It is noted that the leg length and the throat thickness of fillet welds have to be increased to give an equivalent strength to connections if it is necessary to use an electrode, such as an LB-47A, the strength of which is lower than that of the base metal.

Repaired Weld Joints

Recovery of fatigue strength to the original level in fatigue-damaged joints can be achieved by the butt rewelding method although the S-N data presented are based on only a few tests. The combined effect of the removal of the fatigue-damaged metal by gouging and the reheating of the weld metal is thought to be the main reason for this recovery.

CONCLUSION

The series of tests reported in this paper highlighted the importance of welding conditions and weldability considerations, limitations, and cautions for field weld repairs. It can be concluded that field weld repair is applicable to steel bridges, except in highly compressed areas, if there is sufficient assessment of the welding condition, inspection, and removal of defects.

RECOMMENDED PROCEDURE FOR FIELD WELD REPAIR

To secure the necessary quality of repair, field welding should be carefully done following the procedures given hereafter in addition to the provisions of the existing JRA specifications for the construction of new bridges.

1. Stresses of the bridge to be repaired should be checked and vibration should be measured and assessed.
2. A welding procedure test should be done using specimens attached to the actual bridge and under the same conditions as those under which actual repair work will be done. The same skilled welders should be employed for the test and the actual repair work.
3. Scaffolds for field weldings should be stiff enough to prevent the scaffolds from vibrating with a large amplitude.
4. Visual examination and correction of defects should be done carefully.
5. Bead shapes should be smoothed by grinding after welding to minimize stress concentration.

The Hanshin Expressway Public Corporation has recently published a manual for the repair of steel highway bridges (2) that includes field weld repair provisions based on these recommendations. The manual has been successfully followed in repairing damaged bridges of the Hanshin Expressway.

ACKNOWLEDGMENT

The authors would like to thank many colleagues with whom they have discussed these tests, and they will be pleased if this paper is helpful to people concerned with bridge repair.

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2. Manual for Repair of Steel Highway Bridges. Hanshin Expressway Public Corporation, Kobe, Japan, 1984.

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Scale-Model Tests for Full-Depth Precast Concrete Panel-Decked Composite Bridge Span

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ABSTRACT

The Texas Transportation Institute, under the joint sponsorship of the Texas State Department of Highways and Public Transportation and the FHWA, has undertaken a multiyear research program to investigate the strength and structural performance of a bridge deck system using full-depth precast concrete slab panel modules supported on steel stringers. The primary objective of the study is to evaluate the full range of behavior of a composite bridge deck system that uses epoxy mortar and shear studs as joint material for a modular construction using full-depth, full-width precast concrete panels with block-outs. Understanding this behavior would facilitate the use of such a method for rapid bridge deck construction and rehabilitation. The first phase of this program, a series of static load tests on a one-third scale laboratory model, has recently been completed. The design and detailed construction of the scale model, the details and results of the load tests, and the evaluation of the results are presented in this paper. The results indicate that, within the elastic stress range, the construction system described here would develop composite action in a satisfactory manner.

Use of full-depth precast concrete panels is a viable method of bridge deck construction and replacement. Such a method has been used since the early 1960s. More than a dozen transportation agencies have built at least twenty-five bridges using such a method. Many of the projects were for rehabilitation, and some were new constructions. Since 1973 many of these bridges have been constructed to provide composite action. Composite action is afforded, typically, by the use of mechanical shear connectors; or structural mortar based on epoxy, cement, or polymer; or both connectors and mortar. Except for some scattered and minor nonstructural failures, all the bridges are reported to be performing well.

In the case of noncomposite construction, the structural behavior of a span is not greatly dependent on the precast panel decks, although some incidental development of composite action has been reported. In the case of composite construction, on the other hand, structural behavior would be dependent primarily on the performance of the separate components as well as on that of the system as a whole.

Attempts to investigate structural behavior and strength of such construction have been rare. Because the rehabilitation and construction of structures on existing highways are done under extremely restrictive tactical constraints, a reluctance to engage in such research and investigation involving a bridge pressed for service is understandable. Because of the significance of such an investigation, the Texas State Department of Highways and Public Transportation, jointly with FHWA, has undertaken a research program conducted by the Texas Transportation Institute, Texas A&M University, involving laboratory tests, prototype construction, and subsequent tests of the prototype under field conditions. The first phase of this program, the design and construction of a one-third scale laboratory model and static load tests, has recently been completed. The progress and results of this research are described.

DESIGN PROTOTYPE

To establish a basis for design of the model, a typical design of a two-lane, 60-ft nominal span bridge was selected from the Standard Drawings of Steel I-Beam Bridges of the Texas Highway Department (1). A typical cross section is shown in Figure 1(a). The stringers are old standard 36WF150 rolled sections, spaced 8 ft center to center. One signif-