



Title	Bonding-assisted Prevention Methods for Weld Root Cracks in Steel Structural Members
Author(s)	Mao, Jiahao
Citation	大阪大学, 2026, 博士論文
Version Type	VoR
URL	https://doi.org/10.18910/105483
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The University of Osaka

Doctoral Dissertation

**Bonding-assisted Prevention
Methods for Weld Root Cracks
in Steel Structural Members**

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The University of Osaka

December 2025

**Bonding-Assisted Prevention
Methods for Weld Root Cracks
in Steel Structural Members**

by

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Dissertation submitted to the faculty of
Graduate School of Engineering,
The University of Osaka
in partial fulfillment
of the requirements for the degree of
Doctor of Engineering
2025

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ABSTRACT

Title of dissertation: Bonding-assisted Prevention Methods for Weld Root Cracks
in Steel Structural Members

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This dissertation examines fatigue cracks in steel bridge supports, particularly those that start at the fillet weld root between the bottom flange and the sole plate. These root cracks are troublesome due to their hidden location within complex structures, making detection and repair very difficult. To tackle this issue, the research combines structural analysis, experimental testing, and numerical simulations to find ways to prevent these root cracks from forming.

First, finite element analyses were performed on five existing bridges to identify which structural conditions are most susceptible to fatigue cracking. The results showed that bridges with larger girders or a high live load proportion are especially at risk. While increasing the fillet weld size and using bolt-weld hybrid connections can lower stresses at the weld root, practical factors like construction limits and cost issues hinder their effectiveness. These findings point to the need for alternative approaches beyond conventional structural adjustments.

To address this, the research proposes and thoroughly investigates bonding-assisted welding technology as a new method to improve weld root fatigue performance. Two types of bonding materials were tested: adhesive insertion (epoxy resin) and soft metal insertion (zinc). Experimental programs showed that both methods could significantly extend fatigue life by reducing stress concentrations at the weld root. Epoxy insertion led to more than a threefold increase in fatigue life, while zinc insertion provided over a tenfold improvement in certain cases. However, each method had its own advantages and drawbacks. Epoxy offered strong bonding and easy application without extra heat treatment but was vulnerable to welding defects and aging. Zinc ensured better welding

quality and durability but needed controlled heat treatment and had lower bonding strength than epoxy.

Multi-scale finite element simulations were used to broaden the application of bonding-assisted welding to various bridge cases. These analyses confirmed that bonding-assisted welding can significantly lower weld root fatigue stresses, with reductions of up to 62%, and can predict fatigue life extensions of more than 2.5 times in most situations. Suggested optimal parameter ranges include a recommended bond thickness of 0.5 mm material-specific safety distances to balance welding quality, bonding performance, and fatigue improvement.

Overall, this dissertation establishes bonding-assisted welding as a promising method for preventing weld root fatigue cracks in steel structural members. The findings clarify the structural mechanisms of weld root cracking and suggest practical design and fabrication strategies that engineers can apply. Future research will focus on long-term durability, bonding reliability, and exploring new bonding materials to further strengthen this technology for real-world bridges.

Keywords: Bridge engineering, Steel structures, Welding, Fatigue, Root cracks, Bonding

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CHAPTER 1

INTRODUCTION

1.1 Background

In 1779, the world's first iron bridge was built in Shropshire, United Kingdom, over the Severn River. The bridge used a total of 384.6 tons of cast iron for arch ribs, with a span of 30.6 meters [1]. It remains a symbol of the Industrial Revolution [2]. Cast iron, a ferrous alloy mainly made of iron and 2–4% carbon, is characterized by its brittleness and low tensile strength [3]. Even with these drawbacks, the bridge was the first large-scale use of iron-based materials in load-bearing bridge structures. This marked the beginning of the iron bridge era.

Starting in the 1850s, the rise of acid converter steelmaking (Bessemer process) and open-hearth steelmaking (Siemens–Martin process) caused a big increase in steel production. With its lower carbon content (less than 0.25%), greater strength, better toughness, and good weldability, steel gradually took over from cast iron as one of the main materials for building bridges [4].



URL: <https://artuk.org/discover/artworks/the-cast-iron-bridge-near-coalbrookdale-53234>

Figure 1-1 The world's first iron bridge (1779)

1.1.1 Steel Bridges

In the 1870s, rolled steel plates allowed factories to prefabricate bridge components, speeding up the creation of steel truss bridges. In 1874, the Eads Bridge (Figure 1-2 (a))

became the world's first steel truss bridge in the United States. It was the first large-scale use of steel in bridge construction, using 2,390 tons of steel in its arches [5]. In 1883, the Brooklyn Bridge (Figure 1-2 (b)), also in the United States, was completed with a main span of 486 meters, making it the longest suspension bridge span of the 19th century. It used parallel steel wire cables as the main load-bearing components, which significantly improved steel bridge technology [6].

The 20th century witnessed the use of high-strength steel, which allowed for longer bridge spans. In 1937, the Golden Gate Bridge (Figure 1-2 (c)) was finished in the United States. It featured a 1,280-meter main span and stiffening beams for greater rigidity. In 1964, the Verrazano-Narrows Bridge (Figure 1-2 (d)), a double-deck suspension bridge with a 1,298-meter main span, was completed. The Humber Bridge (Figure 1-2 (e)) in the United Kingdom followed in 1981, reaching a span of 1,410 meters. In 1998, Japan completed the Akashi Kaikyo Bridge (Figure 1-2 (f)), which set the world record for the longest suspension span at 1,991 meters in the 20th century. It featured a tower height of 297 m and used 1,800 N/mm² ultra-high-strength steel wire in its main cables for the first time [7].

In the 21st century, improvements in computer technology and finite element methods have made steel bridge design more precise and efficient. New materials and construction methods have also supported longer spans, faster speeds, and heavier loads. In 2012, Russia completed the Russky Bridge (Figure 1-2 (g)), the world's longest cable-stayed bridge with a 1,104-meter main span and an orthotropic steel box girder [8]. In 2020, China finished the Pingnan Third Bridge (Figure 1-2 (h)), the world's longest-span concrete-filled steel tube arch bridge at 575 meters. It used an intelligent system based on BeiDou satellite positioning during construction [9]. In 2022, Turkey completed the Canakkale Bridge (Figure 1-2 (i)), the world's longest suspension bridge with a main span of 2,023 meters [10]. Most recently, in 2024, China finished the Feilong Bridge (Figure 1-2 (j)), the longest corrugated steel web continuous rigid frame composite beam bridge, with a main span of 185 meters [11]. Building these steel bridges has relied heavily on welding, a key technology in modern bridge engineering. Even a typical 40-meter simply supported steel girder bridge can have thousands of meters of weld length.



(a) Eads Bridge (1874)



(b) Brooklyn Bridge (1883)



(c) Golden Gate Bridge (1937)



(d) Verrazano-Narrows Bridge (1964)



(e) Humber Bridge (1981)



(f) Akashi Kaikyo Bridge (1998)



(g) Russky Bridge (2012)



(h) Pingnan Third Bridge (2020)



(i) Canakkale Bridge (2022)



(j) Feilong Bridge (2024)

Figure 1-2 Steel and steel-concrete composite bridges

1.1.2 Welding

Welding is a process that uses heat, pressure, or a combination of both to achieve atomic bonding between materials, resulting in a permanent connection (Figure 1-3). Since the 1880s, arc welding has been used for metal joining (Figure 1-4). Over time, welding technology has evolved significantly, giving rise to various methods such as gas metal arc welding [12], submerged arc welding [13], resistance welding [14], laser welding [15], stir friction welding [16], and hybrid welding [17].

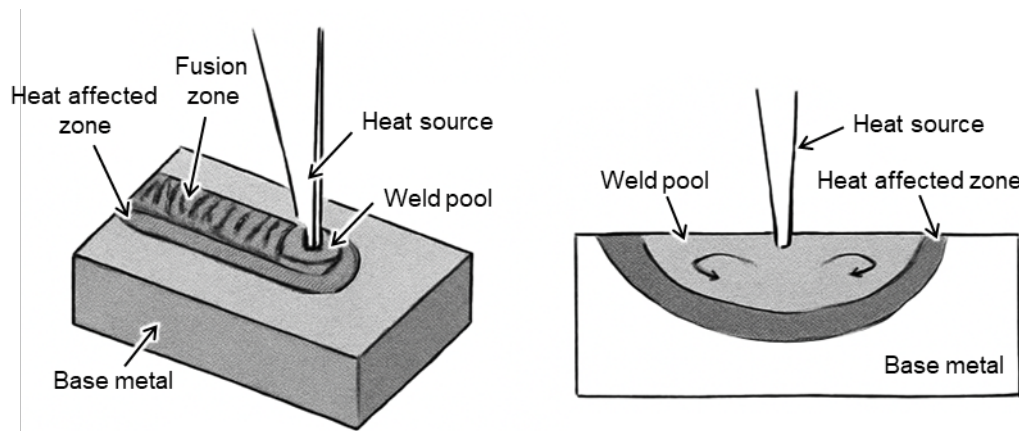


Figure 1-3 Schematic diagram of welding [18]

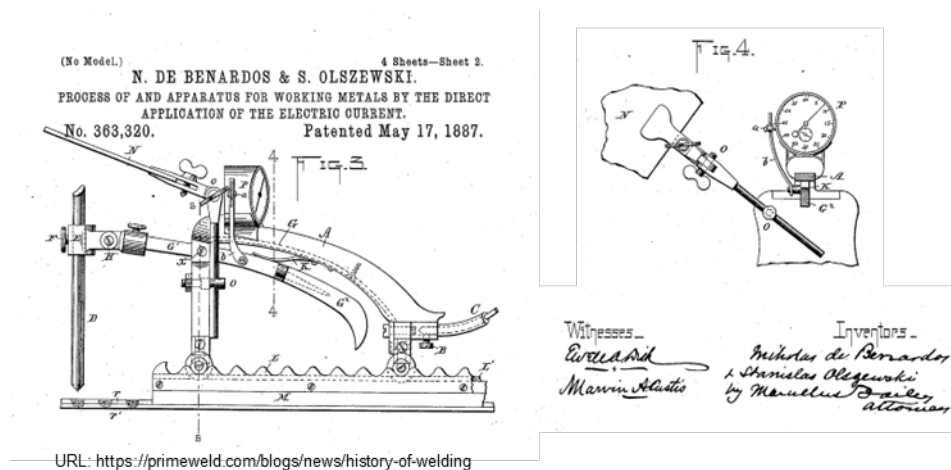


Figure 1-4 Original patent drawing for the first welding machine

Steel bridge components are usually prefabricated in factories and assembled on site. Welding plays a vital role in creating strong and precise connections. Compared to bolting, riveting, and adhesive bonding, welding has significant benefits for connection

performance, structural efficiency, and flexibility. It creates metallurgical or solid-state bonds between the base and filler materials, or between base materials, resulting in joint strengths that can be equal to or greater than that of the base material itself [19]. In contrast, bolted and riveted connections create holes that weaken the base material and are prone to loosening over time due to vibration or changing loads [20]. Furthermore, the continuous, gap-free joints made by welding effectively block liquids, gases, and corrosive substances, which helps improve the durability of the structure [21]. Welding also removes the need for extra connectors, cutting down structural weight, simplifying the design, and reducing material costs through integrated fabrication [22].

However, welding has its own challenges. The high localized temperatures can change the microstructure of the surrounding base metal in the heat-affected zone, which may lead to embrittlement, softening, or phase transformation. Uneven thermal cycles can create residual stresses and distortions, which often require adjustments after welding and increase fabrication costs. Despite improvements in process control, welding can still result in defects like cracks, porosity, incomplete fusion, or lack of penetration due to poor technique or inconsistent material quality [23, 24] (Figure 1-5).

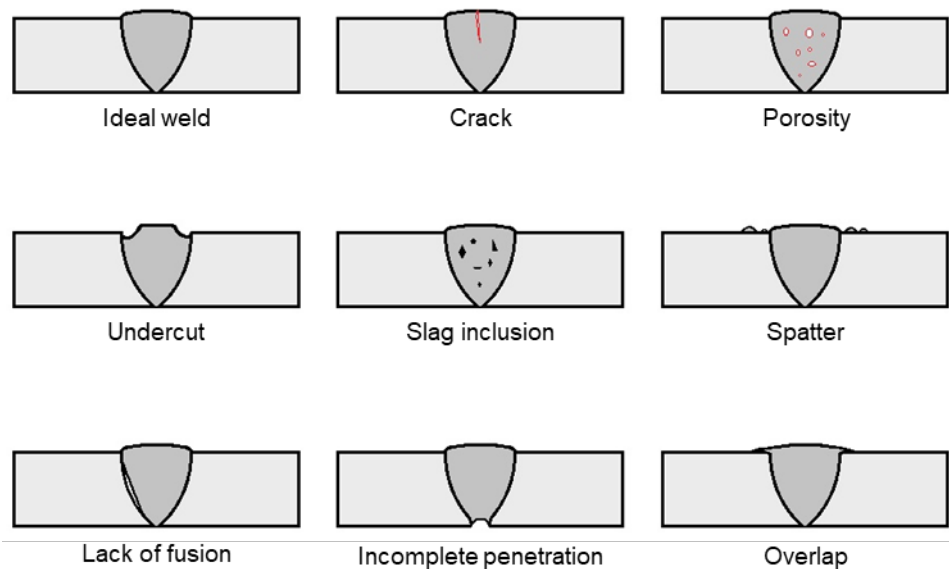


Figure 1-5 Welding defects

In some cases, limited space or thick sections lead to the use of non-penetrating welds. These joints often have geometric discontinuities that can increase stress concentration and lead to fatigue cracking, which can threaten structural integrity.

1.1.3 Fatigue

Fatigue is when a material fails because it gradually accumulates damage from repeated stress or strain, even if the applied stress is much less than its tensile strength [25]. Typical examples include metal parts failing from vibrations of bridge cables caused by repeated vehicle loads [26, 27], alternating stresses on aircraft wings during takeoff and landing [28], and the repeated bending of rotating mechanical shafts [29, 30]. Therefore, fatigue is a fundamental topic in the study of material mechanics and structural safety.

1.1.3.1 History

In 1837, Albert, a mining official in Clausthal, published the first known results of fatigue tests. Braithwaite first introduced the term “fatigue” in 1854 [31]. During the 1860s, Wöhler carried out a series of fatigue experiments. He compiled and organized data that led to the idea of the “fatigue limit.” This limit is the maximum stress below which a material can withstand an infinite number of load cycles without failing. His work laid the groundwork for the modern field of fatigue. Building on Wöhler’s findings, his successor Spangenberg plotted the first stress amplitude versus number of cycles curve (S-N curve), revealing the relationship that lower stress levels allow materials to withstand more cycles [32]. In 1910, Basquin introduced a logarithmic representation of the finite-life region of the S-N curve, using $\log \sigma_a$ (stress amplitude) versus $\log N$ (number of cycles), and proposed a simple empirical equation that remains in use today [33].

$$\sigma_a = CN^n \quad (1-1)$$

Where, σ_a is the stress amplitude; N is the number of cycles; C and n are material constants determined through tests.

In 1920, Griffith established the foundation of fracture mechanics and introduced the concept of the stress intensity factor K_I for Mode I loading. This development laid the

theoretical foundation for subsequent research into fatigue mechanisms [34, 35]. Building on Griffith's work, later researchers derived the stress intensity factors K_{II} and K_{III} , corresponding to Mode II and Mode III loading, respectively (Figure 1-6). These factors depend on the geometry of the structure and the type of loading and are essential for analyzing the stress field near the crack tip, as expressed in the following equations.

$$\sigma_{ij} = \frac{K_N}{\sqrt{2\pi r}} f_{ij}(\theta) \quad (i, j = x, y, z) \quad (1-2)$$

Where, σ_{ij} is the stress at the crack tip; K_N is the stress-intensity factor, when N takes I, II, and III, it corresponds to mode I, II, and III, respectively; r is the distance from the crack tip; and θ is the angle between the distance r and the x -axis.

In 1939, Orowan first applied the “dislocation pile-up” theory to explain the mechanism of fatigue crack initiation [36]. In 1945, Miner introduced the linear fatigue damage accumulation hypothesis, originally proposed by Palmgren, which became known as the Palmgren–Miner rule [37].

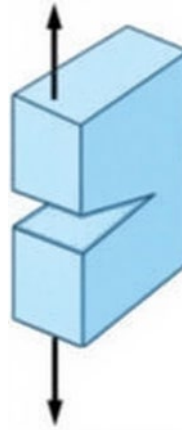
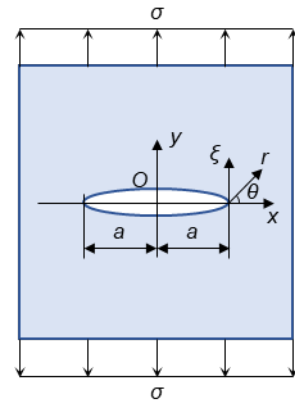
$$\sum_{i=1}^k \frac{n_i}{N_i} = 1 \quad (1-3)$$

Where n_i is the number of cycles applied at a given stress cycle; N_i is the fatigue life under the corresponding stress cycle; and k represents the distinct stress cycles experienced.

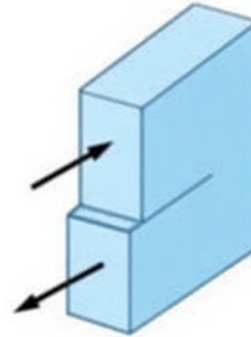
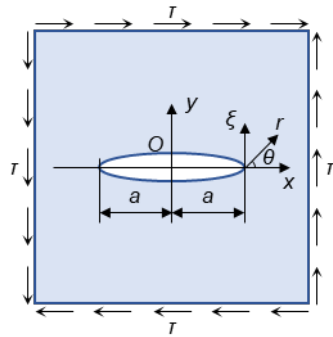
Since the 1950s, the advent of electron microscopy has enabled researchers to observe the microscopic evolution of fatigue cracks. In 1962, Paris introduced a crack growth rate model called the Paris law. While this model has limitations, especially in considering material differences and complex loading conditions, it represented an important advancement in the understanding of fatigue from a quantitative and microstructural perspective [32, 38].

$$\frac{da}{dN} = C(\Delta K)^m \quad (1-4)$$

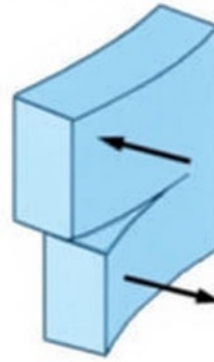
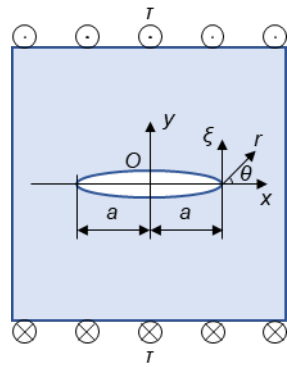
Where, a is the crack length (Figure 1-6); ΔK is the range of stress-intensity factor variation; C and m are material constants determined through tests.



(a) Mode I



(b) Mode II

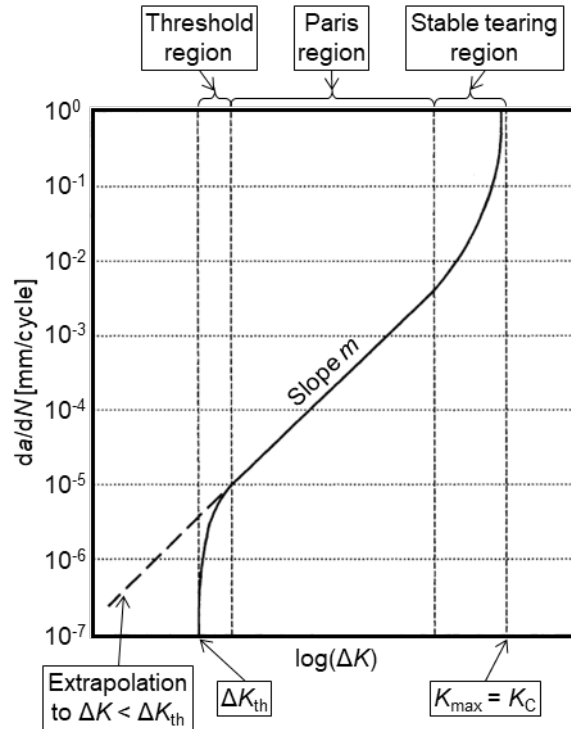


(c) Mode III

Figure 1-6 Calculation diagram of stress-intensity factors

1.1.3.2 Mechanism

It is widely recognized that the fatigue damage process in metals happens in three stages. These stages correspond to different areas in the $da/dN - \Delta K$ relationship (Figure 1-7) [39]. Essentially, fatigue arises from the formation and growth of tiny defects within the material due to repeated loading.

Figure 1-7 Typical da/dN - ΔK relationship [40]

(1) Threshold region

(ΔK near the fatigue threshold ΔK_{th})

Under cyclic loading, atomic movements and dislocation movements occur within the metal's crystal lattice. This leads to localized plastic deformation. This microscopic deformation starts surface or subsurface microcracks, usually a few microns to hundreds of microns in size. These areas, often linked to material flaws or inclusions, are at risk of stress concentration, which speeds up crack initiation.

(2) Paris region

(Linear relationship between da/dN and ΔK in logarithmic coordinates)

In this stage, microcracks grow steadily under repeated stress. Plastic deformation happens at the crack tip due to stress concentration, which lets the crack expand into the material at a rate that roughly matches the applied stress amplitude. This region usually makes up the bulk of the fatigue life, especially in high-cycle fatigue conditions.

(3) Stable tearing region

(ΔK approaching the material's fracture toughness K_{IC})

When the crack reaches a critical size where the remaining cross-sectional area can no longer handle the applied load, rapid crack growth takes place, resulting in sudden fracture. This failure is often brittle, even in ductile materials.

The fracture surface typically shows three different zones: the crack initiation zone (smooth and slightly concave), the fatigue propagation zone (marked by concentric “beach marks”), and the final fracture zone (characterized by coarse grains or crystalline features) (Figure 1-8).

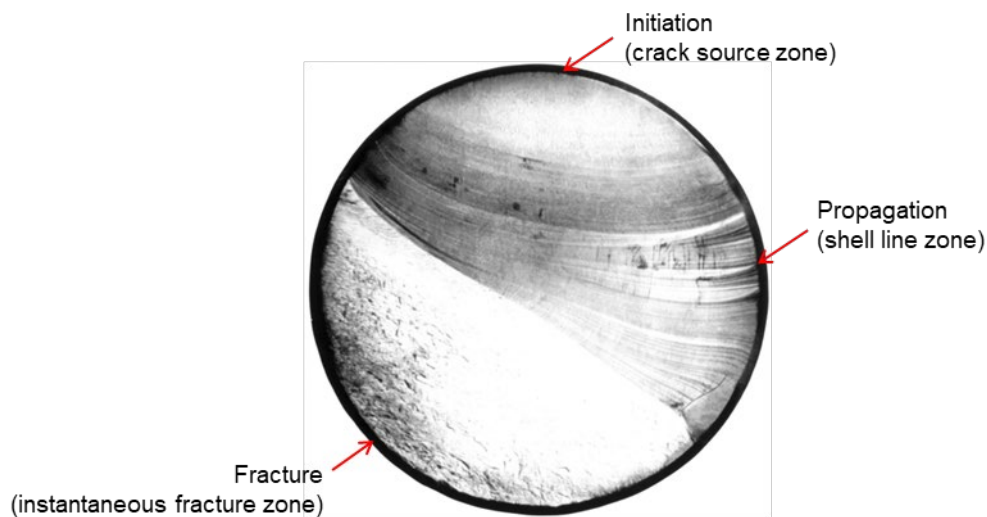


Figure 1-8 Macrograph of fatigue fracture surface of a steel shaft [41, 42]

1.1.3.3 Challenges

Although the mechanisms of fatigue are now relatively well understood, it is still hard to predict fatigue life in real engineering situations. This difficulty comes from uncertainties related to construction quality, material differences, and service conditions. For instance, steel bridges that face long-term cyclic loading from vehicles or trains are especially prone to fatigue cracking and structural deformation. This is particularly true at spots with high stress, like welded joints or internal defects.

Additionally, early fatigue cracks are usually very small and hard to spot. Once they begin, they can grow quickly and may cause sudden brittle fractures. Consequently, many

aging steel bridges get taken out of service too early because they don't resist fatigue well enough. At the same time, new steel bridges have to deal with tougher service environments, including heavier traffic loads, which make fatigue issues worse.

A structural failure analysis report outlined the main causes of damage for different types of structures [43], as summarized in Table 1-1. Unlike buildings and conveyors, where other issues are more common, fatigue is the top cause of damage in bridges. Overall, fatigue is the third most frequent cause of structural failure. This highlights the importance of focusing on fatigue prevention and control, especially in bridge design.

Table 1-1 Causes of structural damage [43]

Causes	Totality		Buildings		Bridges		Conveyors	
	No.	%	No.	%	No.	%	No.	%
Static strength	161	29.7	102	33.6	19	14.8	40	36.0
Environment	101	18.6	59	19.4	41	32.0	1	0.9
Fatigue	92	16.9	8	2.6	49	38.3	35	31.5
Stability	87	16.0	62	20.4	11	8.6	14	12.6
Rigid body movement	44	8.1	25	8.2	2	1.6	17	15.3
Thermal loads	23	4.2	23	7.6	0	0	0	0
Elastic deformation	15	2.8	14	4.6	1	0.8	0	0
Brittle fracture	15	2.8	9	3.0	5	3.9	1	0.9
Others	5	0.9	2	0.7	0	0	3	2.7
Summary	543	100	304	100	128	100	111	100

1.2 Literature Review

Extensive research has been conducted on the fatigue behavior of steel bridges. This section reviews the literature from four perspectives: (1) common causes of fatigue cracks, (2) fatigue crack detection, (3) fatigue crack repair and retardation, and (4) fatigue evaluation methods.

1.2.1 Common Causes of Fatigue Cracks

1.2.1.1 Weld Defects

Welded components are usually more prone to fatigue cracking than bolted or riveted ones. This is mainly because of imperfections in the welds and discontinuities, like undercuts, porosities, and lack of fusion. These defects create stress points and can start fatigue cracks. Additionally, harmful residual stresses, geometric issues at welds, and tiny cracks formed during welding can speed up fatigue damage in welded bridge parts.

These defects are often seen in older bridges built before modern quality control and nondestructive testing methods were developed. They also appear in components considered "secondary" in terms of load-bearing capacity, which means they don't always get strict inspections. One notable case is the fatigue cracking found in bridge girders with welded longitudinal stiffeners, as noted in several bridges across the United States [44, 45]. As shown in the case in Figure 1-9, cracks started from weld defects where the fillet weld connects the stiffener to the web, and the butt weld is at the stiffener's transverse joint. Since this detail wasn't seen as critical, it was left out of inspections. However, these connections can allow cracks to spread from the stiffener into the web, sometimes resulting in brittle failure [46].

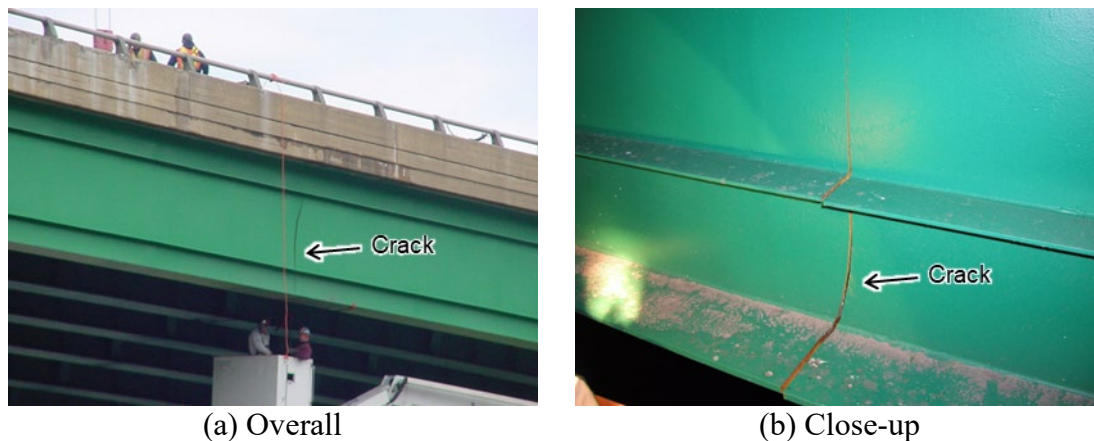


Figure 1-9 Cracking caused by lack of fusion [45]

In orthotropic steel decks, U-shaped stiffeners are often used. As shown in Figure 1-10, poor weld quality at the root can lead to incomplete penetration, which can create sharp defects that look like pre-existing cracks. When subjected to repeated wheel

loading, these features cause significant concentration. This makes the weld root especially vulnerable to fatigue cracking.

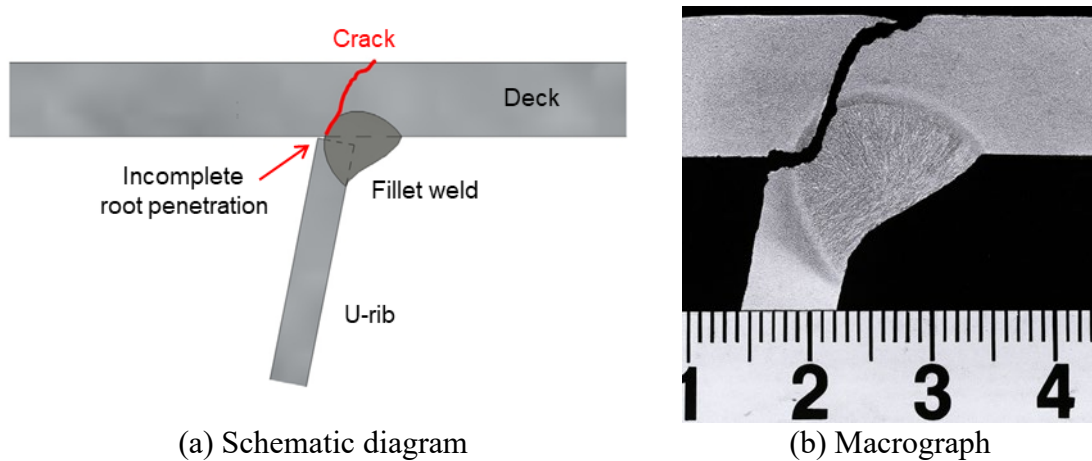


Figure 1-10 Cracking caused by incomplete root penetration [47]

1.2.1.2 Cross-sectional Changes

Cross-sectional changes in members are common details in steel bridge structures. A basic example is the transition in flange dimensions of a plate girder. It is well established that fatigue strength can be significantly enhanced by ensuring a smooth geometric transition. This is typically achieved by tapering or rounding the edges, which reduces stress concentrations at the transition zone (Figure 1-11).

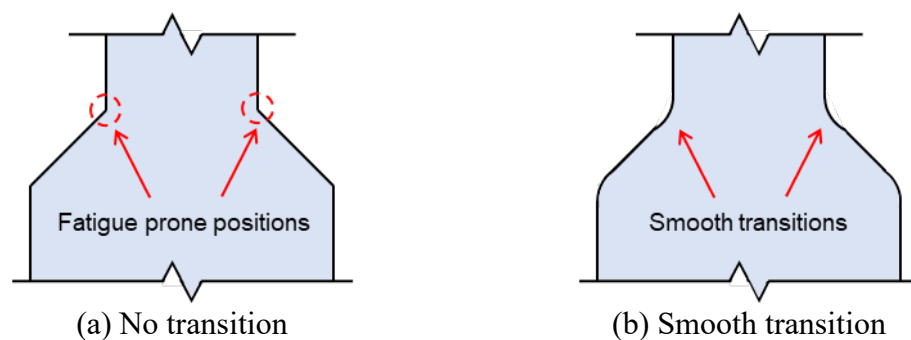


Figure 1-11 Smooth transitions to reduce stress concentration

However, more complex geometries involving abrupt section changes can create complicated stress states with additional stress components, which may be high enough to initiate fatigue damage. These secondary stresses are often difficult to predict through

simple analytical methods and may be overlooked during the design phase. Despite the design of smooth transitions, fatigue cracking has been observed in both the webs of main girders and the fillet welds connecting flanges to webs. In the latter case, cracks can propagate tangentially to the flange curvature due to radial stresses or radially due to tangential stress components (Figure 1-12).



Figure 1-12 Fatigue cracking at cross-sectional changes [46]

1.2.1.3 Vibrations

Vibration-induced fatigue cracking in steel bridges can result from both primary and secondary stresses and may be triggered by various external actions, including wind loads, traffic, and seismic activity. One of the most well-known cases highlighting the significance of wind-induced vibration is the 1940 collapse of the Tacoma Narrows Bridge (Figure 1-13). In this incident, wind-induced stress reversals in the bridge deck, composed of riveted steel beams, excited a 0.2 Hz torsional vibration mode. This resonance led to low-cycle bending fatigue and ultimately, catastrophic failure [48].

In addition to deck structures, fatigue cracks from vibration have also been seen in the hangers of arch bridges and the cables of cable-stayed bridges. These thin elements are especially sensitive to resonance because of their low bending stiffness. They can be affected by vibrations from traffic and/or wind loads [46, 49]. One example is a railway arch bridge built in 1943, with a span of 61 m, where fatigue cracks developed in the hangers at their connections to the steel arch (Figure 1-14). The bridge featured a steel arch and two tie girders connected by floor beams. The hangers, made of 79 mm diameter steel cables, began to fail in the early 1980s. Several hangers cracked and completely

detached from their connections. The fatigue damage was attributed to oscillation combined with secondary bending stresses at the threaded connection between the hangers and the arch [46].



Figure 1-13 Collapse of the Tacoma Narrows Bridge [50]

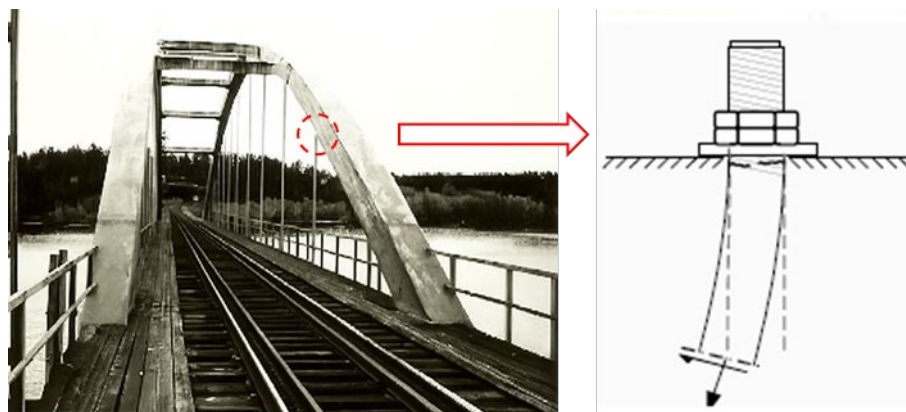
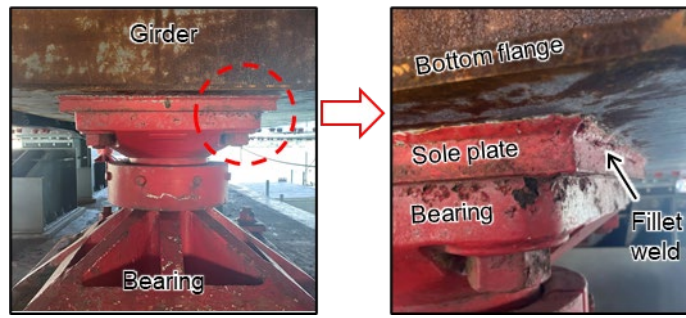


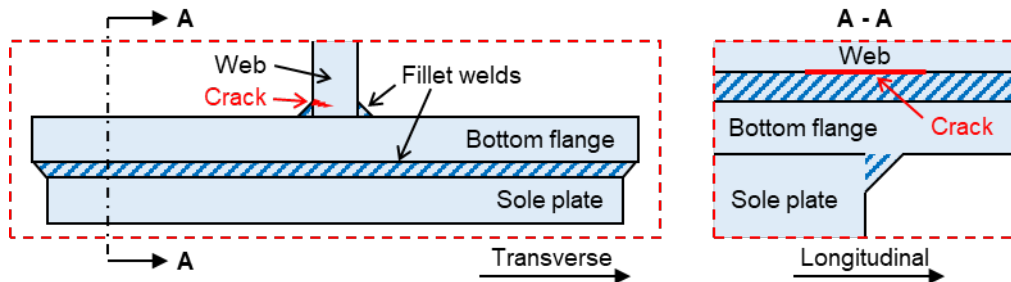
Figure 1-14 Vibration-induced fatigue cracking of the hangers [46]

1.2.1.4 Heavy Loads

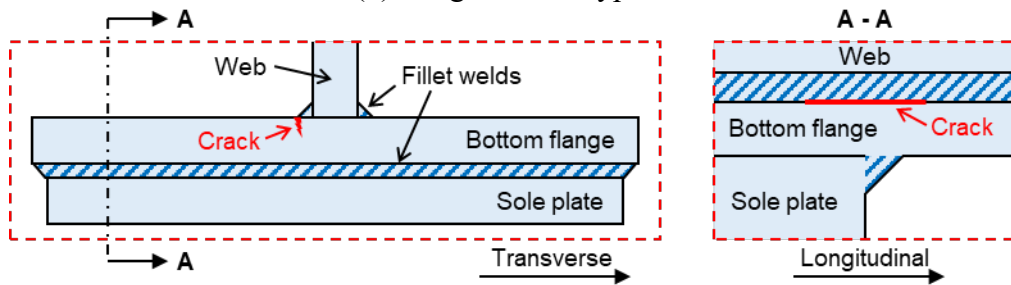
Key load-bearing components in bridges, such as supports, face high stress over their lifespan. Repeated traffic can worsen their fatigue performance. The structure around bridge bearings is often complex, with sudden changes in cross-section. When heavy loads are added, this complexity increases stress concentrations and makes fatigue issues worse. Various types of fatigue cracks have been reported near bridge bearings (Figure 1-15) [49, 51].



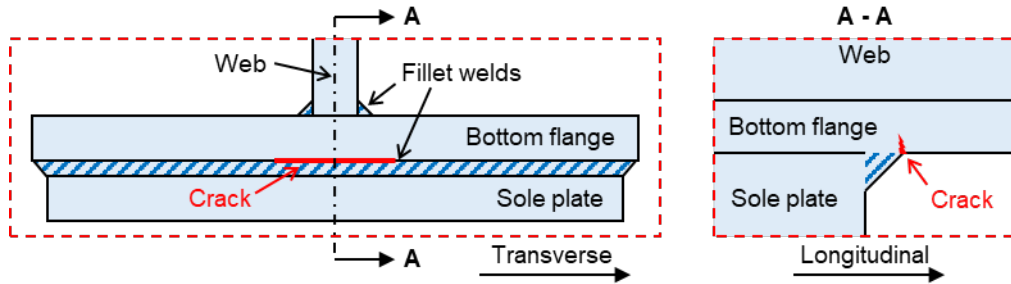
(a) Details of the bridge bearing



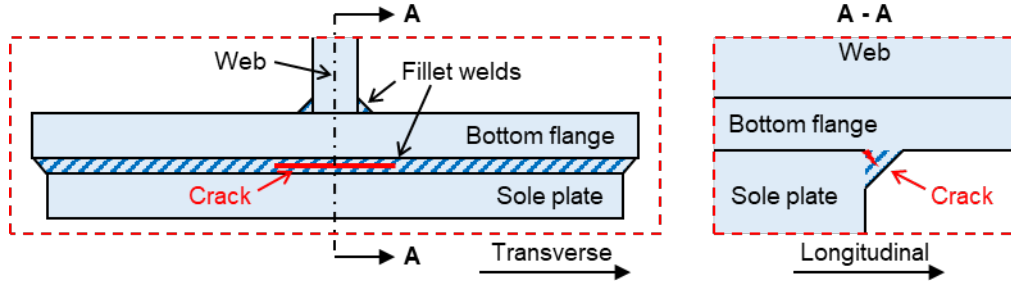
(b) Fatigue crack Type A



(c) Fatigue crack Type B



(d) Fatigue crack Type C



(e) Fatigue crack Type D

Figure 1-15 Common fatigue cracks near bridge supports

Among these, Type D fatigue cracks are particularly concerning. Unlike the other three types, which usually begin at the surface, Type D starts from the weld root inside the structure. These internal fatigue cracks are especially troublesome because they are hidden. They are hard to find before they reach the surface, which makes inspection and repair more difficult. As a result, their presence threatens structural integrity. Because of their concealed nature and the absence of established detection and prevention methods, internal fatigue cracks require special attention during both the design and construction phases. However, research on these cracks is still limited, and clear effective solutions have yet to be defined.

1.2.2 Fatigue Crack Detection

For steel bridges in service, it is essential to conduct regular inspections or monitoring to identify the initiation and progression of fatigue cracks and to ensure structural safety. To minimize disruption to bridge operation, non-destructive testing methods are commonly employed. The most widely used techniques include magnetic particle testing, penetrant testing, ultrasonic testing, and radiographic testing [52, 53]. These methods are introduced in the following.

1.2.2.1 Magnetic Particle Testing

Magnetic particle testing is widely used for detecting surface and near-surface cracks in welded components made of ferromagnetic materials. When such a component is magnetized, discontinuities perpendicular to the magnetic field produce a localized magnetic leakage field. This field attracts ferromagnetic particles, either in dry or wet suspension, to the discontinuity. The accumulation of particles forms visible indications at the defect locations [54]. Figure 1-16 illustrates the principle of magnetic particle testing, and Figure 1-17 shows an example of a detected surface crack.

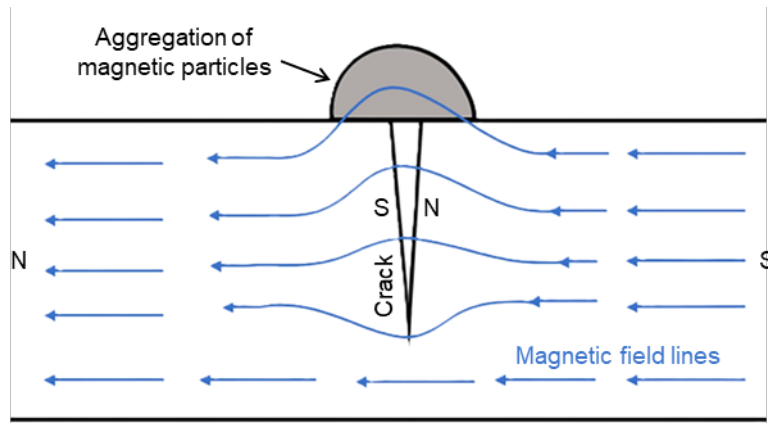


Figure 1-16 Principle of magnetic particle testing [54]

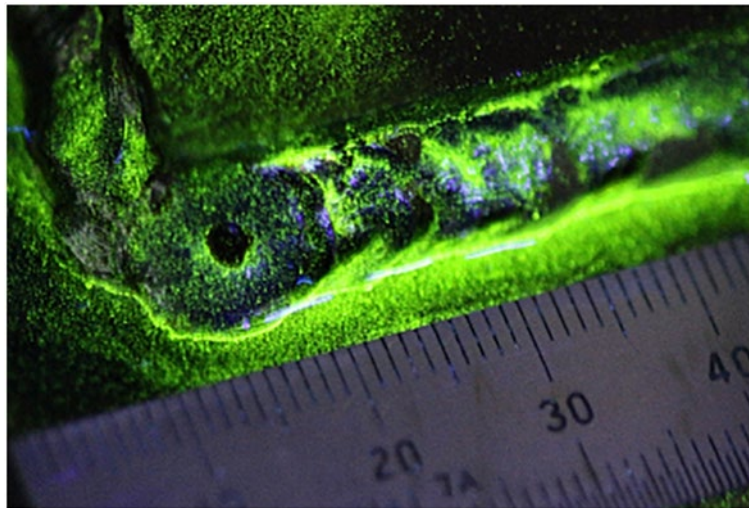


Figure 1-17 Detected crack using magnetic particle testing [55]

Compared to other non-destructive testing methods, magnetic particle testing is relatively simple and cost-effective. Yoke-based systems are portable and suitable for field use. However, this method has several limitations. It is applicable only to ferromagnetic materials and is primarily sensitive to surface and shallow subsurface defects. The surface must be relatively clean and free of excessive rust or scale, and proper demagnetization is required after testing.

1.2.2.2 Penetrant Testing

Penetrant testing is a non-destructive method that identifies surface-breaking defects in non-porous materials. It depends on capillary action, where a low-viscosity liquid penetrant is drawn into surface openings. After applying a developer, the penetrant seeps back out and reveals any flaws. This technique is commonly used in many industries because it is simple, cost-effective, and can find different types of surface imperfections. Figure 1-18 illustrates the general procedure, and Figure 1-19 shows an example of cracks detected on a gear using penetrant testing.

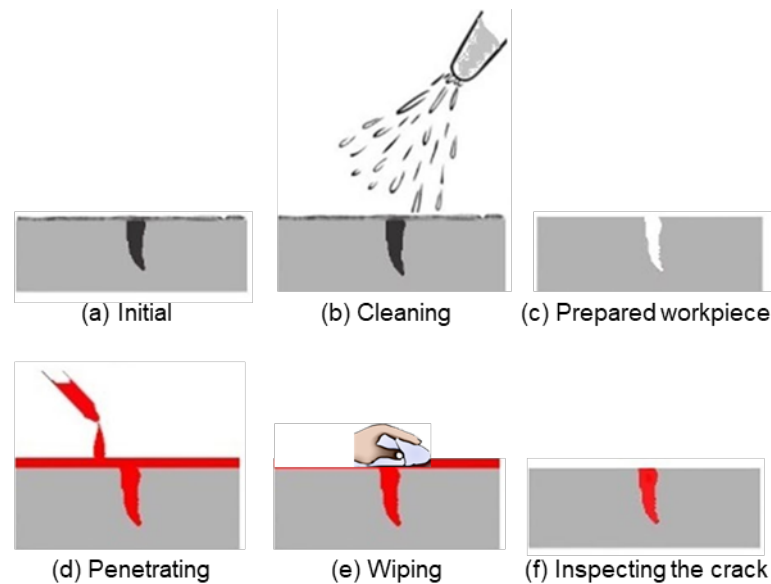


Figure 1-18 Procedure of penetrant testing [56]



URL: <https://pisl.co.tt/services/dye-penetrant-testing-pt/>

Figure 1-19 Detected cracks using penetrant testing

Penetrant testing is inexpensive and can be done in the field. It works on a variety of materials and components. The basic principles are simple, and the method can find small surface flaws that visual inspection might miss. With modern materials, it can reliably detect surface cracks as small as $0.5\ \mu\text{m}$ [57]. However, penetrant testing can only identify flaws that reach the material's surface; it cannot find internal or subsurface defects. The method's effectiveness can be affected by surface finishing and roughness. Uneven or rough surfaces may stop the penetrant from entering small cracks or make it hard to remove extra penetrant. It's crucial to clean the surface thoroughly before testing because contaminants can block the penetrant and cause false results. Furthermore, the process includes several steps: cleaning, applying the penetrant, allowing it to sit, removing excess penetrant, applying the developer, and conducting a visual inspection. These steps can make the process take a bit of time.

1.2.2.3 Ultrasonic Testing

Ultrasonic testing is a non-destructive method that identifies internal flaws by using differences in acoustic properties between the material and potential defects. It relies on reflecting ultrasonic waves and measuring changes in transmitted energy or time-of-flight to find discontinuities within the material [58]. The basic principle is shown in Figure 1-20.

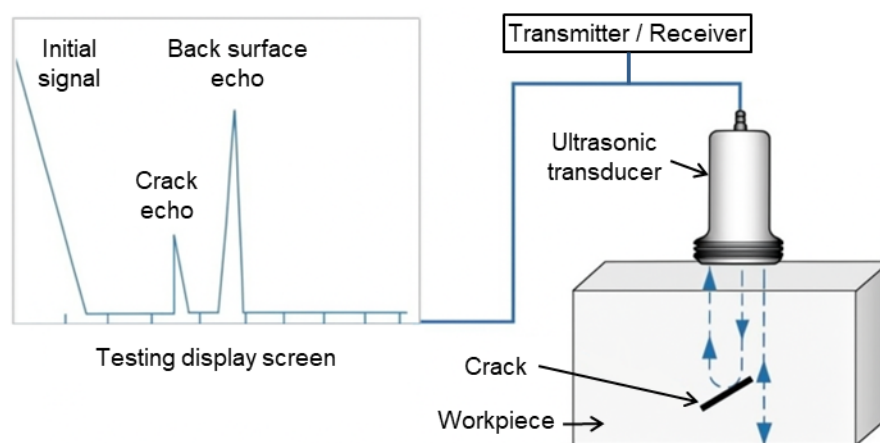


Figure 1-20 Principle of ultrasonic testing [59]

Ultrasonic testing can find very small flaws and works with many types of materials and shapes. It can penetrate deeply into the material, making it particularly useful for thick components [60]. This method provides immediate information about the internal condition of the structure. However, it has its limitations. Inspecting parts with irregular shapes or those that are very small, thin, or uneven can be difficult. Effective testing also needs good surface contact, which requires careful cleaning and preparation of the surface. Furthermore, cracks that are aligned parallel to the direction of the sound beam may not be detected.

1.2.2.4 Radiographic Testing

Radiographic testing is a non-destructive testing method that uses penetrating electromagnetic radiation, such as X-rays or gamma rays, to examine the internal structure of a material or component [61]. Its principle is shown in Figure 1-21.

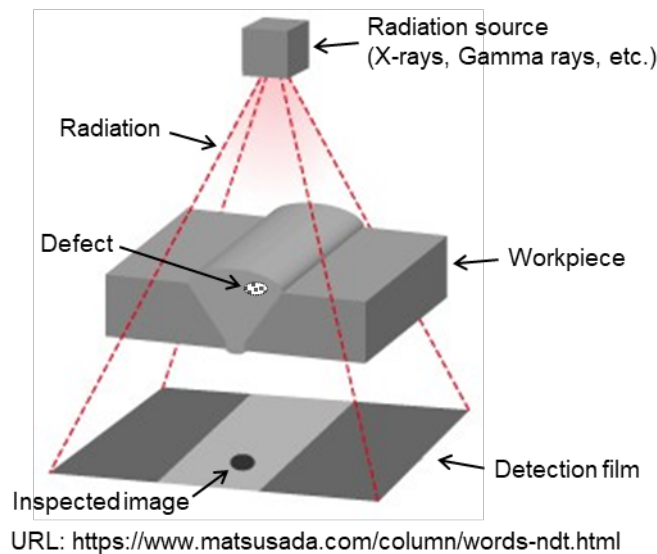


Figure 1-21 Principle of radiographic testing

This technique can reveal internal flaws, inconsistencies, or other hidden features that are not visible on the surface. Radiographic testing produces a film or digital image that serves as a permanent inspection record. This allows for future reference and analysis. One benefit of this method is that it usually requires minimal surface preparation, making it a relatively convenient inspection option. However, there are also significant limitations.

Since radiographic testing involves ionizing radiation, strict safety measures must be followed to protect personnel [62]. The equipment, especially for X-ray sources, can be expensive. Additionally, the method typically does not provide precise information about the depth of a defect. Like ultrasonic testing, radiographic testing is most effective for detecting flaws that are perpendicular to the radiation beam. Defects aligned with the beam direction may not be accurately detected, which can affect reliability.

1.2.2.5 Summary

The four common methods for detecting fatigue cracks mentioned above are not effective at identifying Type D fatigue cracks shown in Figure 1-15. Magnetic particle testing and penetrant testing only work on surface-breaking cracks. Therefore, they cannot detect internal cracks like Type D, which start beneath the surface and become a serious threat once they propagate outward.

Ultrasonic testing can find internal flaws, but it needs a very flat surface for reliable results. In the case of fillet welds between the bottom flange and the sole plate of a bridge, the surface is often uneven, as illustrated in Figure 1-15 (a). This unevenness reduces detection accuracy. Radiographic testing also encounters difficulties due to the complex shapes near bridge supports and the roughness of the weld surface. Moreover, using ionizing radiation restricts its use in field settings.

Other non-destructive testing methods, such as strain testing [63], eddy current testing [64], and infrared thermography testing [65], have been looked into but also do not reliably detect Type D cracks.

In summary, no current method allows for the accurate detection of Type D crack propagation before these cracks reach the surface. Therefore, it is necessary to focus on preventing these cracks through appropriate design and construction strategies to ensure structural safety.

1.2.3 Fatigue Crack Repair and Retardation

Fatigue cracks often appear in aging steel bridges. When they are found through regular inspections and monitoring, it is important to carry out repairs or take steps to slow their growth. This helps maintain the structural safety of the bridge and extend its

service life. The following introduces several common methods for repairing and retarding the propagation of fatigue cracks.

1.2.3.1 Stop-hole Drilling

Stop-hole drilling is a technique used to retard fatigue cracks by drilling a hole at or near the crack tip to relieve stress concentration and interrupt crack propagation [66, 67]. This method is illustrated in Figure 1-22. By redistributing stress around the crack tip, stop-holes can effectively retard or prevent further crack growth, thereby extending the fatigue life of the component.

Although stop-hole drilling can be effective in the short term, it is generally considered a temporary measure. More permanent repairs, such as patching or component replacement, may eventually be required. Additionally, the process may reduce the cross-sectional strength of the component, and the stop-hole itself can become a new point of stress concentration. Therefore, careful design regarding the size, shape, and placement of the hole is essential. This method is typically limited to retarding through-thickness cracks and may not be suitable for internal or hidden fatigue cracks.



Figure 1-22 Stop-hole on a bridge diaphragm [68]

1.2.3.2 Vee-and-weld

Vee-and-weld is a repair method for metal structures that involves grinding a V-shaped groove along the fatigue crack and then filling the groove with weld metal. This technique is especially effective for long, through-thickness cracks in welded joints [69]. Figure 1-23 shows an example of a fatigue crack that has been gouged and prepared for repair by welding.

The process involves removing the cracked material, preparing the groove, and then performing the weld, often with several passes to ensure full penetration and strength. When done correctly, vee-and-weld can restore the repaired section's static strength, ductility, fracture toughness, and fatigue resistance to levels comparable to the original structure [70]. However, this method requires more labor and time than some other repair techniques. If the welding is not carefully controlled, it can create new defects like cracks, porosity, or incomplete fusion, which may undermine the repair's effectiveness.

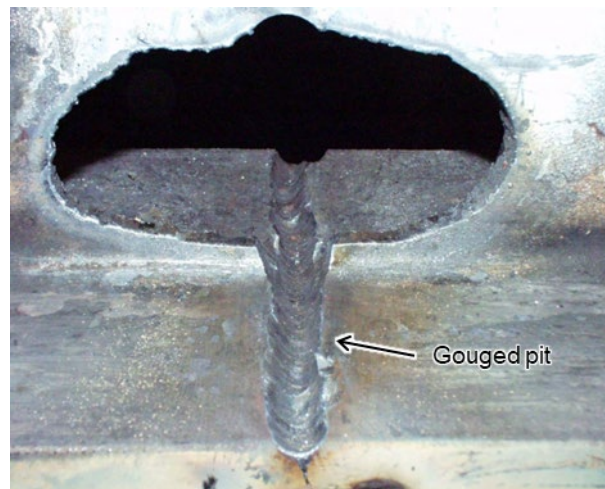


Figure 1-23 Air arc gouged flange of a beam for vee-and-weld repair [71]

1.2.3.3 Grinding

Grinding is a machining process that uses high-speed abrasive wheels, pads, or belts to remove material from a surface [72]. It is often used to smooth the weld toe area. This reduces stress concentrations and slows the formation of fatigue cracks, which helps to extend fatigue life, as shown in Figure 1-24.

In addition to improving fatigue life, grinding can also serve as a repair method. By carefully removing material around a fatigue crack, including at the crack tip, it helps prevent further growth. This process can change the distribution of the local stress, reducing stress concentration in critical areas like the weld toe [73]. One study investigated combining grinding with welding to boost the strength of the joint and prevent fatigue cracks from starting [74]. However, grinding is usually not effective for deep cracks or in critical areas with high stress concentrations. If done incorrectly, grinding can thin the component's cross-section, which might increase bending stress and hurt its fatigue performance.

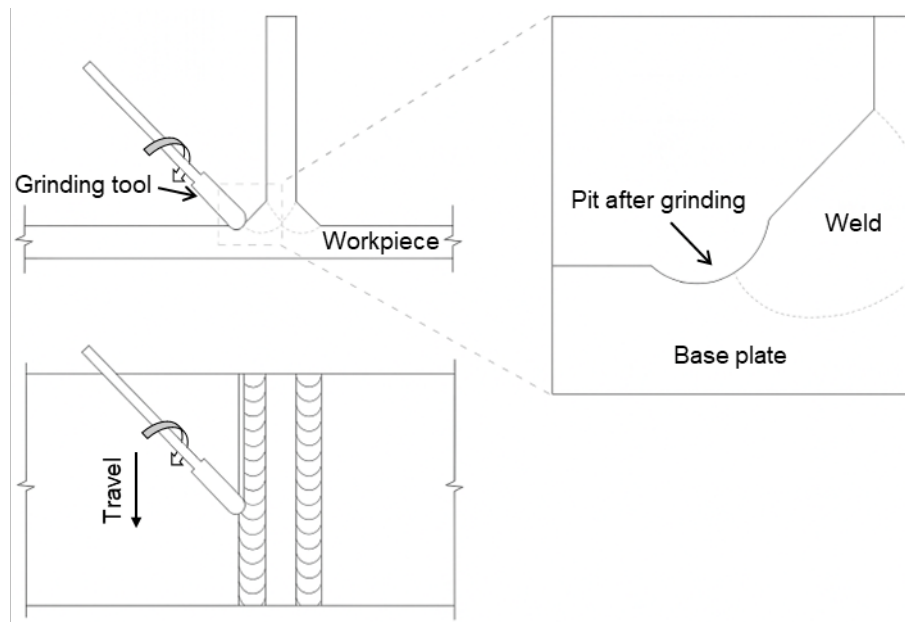


Figure 1-24 Grinding for improving fatigue life at the weld toe [71]

1.2.3.4 Peening

Peening is a metalworking process used to improve material properties by introducing compressive residual stresses at the surface [75]. This usually happens through mechanical methods like hammering or shot blasting, as shown in Figure 1-25. The process deforms the surface layer of the material, causing it to expand and generate beneficial compressive stresses that oppose the tensile stresses responsible for fatigue crack initiation.

By mitigating tensile stresses, peening significantly improves resistance to fatigue, cracking, and corrosion [76]. Figure 1-26 shows an example of a peened weld toe, where the fatigue performance has been enhanced through surface treatment. However, peening is limited to surface applications. It cannot address subsurface or internal cracks and is therefore most effective as a preventive surface treatment rather than a repair for existing deep flaws.

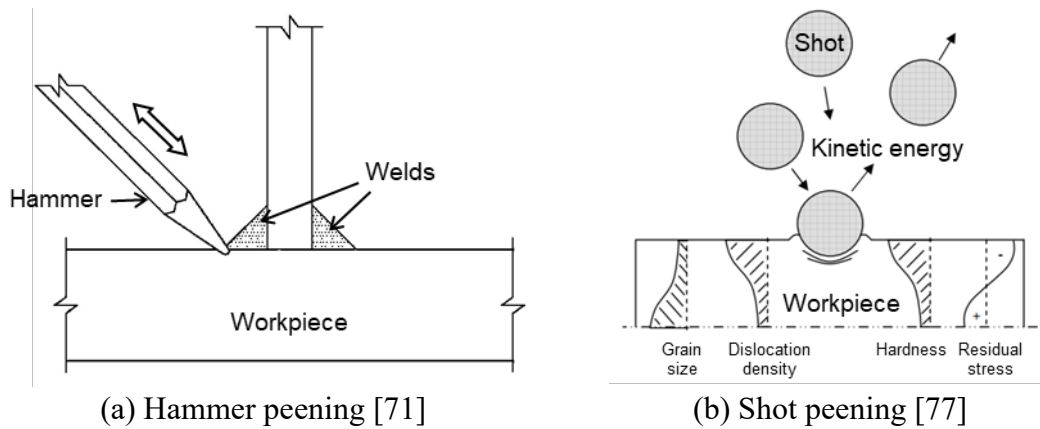


Figure 1-25 Principle of the peening process



Figure 1-26 Peened weld toe [71]

1.2.3.5 Summary

This subsection introduces four commonly used methods for fatigue crack repair and retardation. However, none of these methods are applicable to Type D cracks, as shown

in Figure 1-15. Stop-hole drilling requires precise positioning to be effective. Moreover, it requires the stop-hole to be perpendicular to the crack propagation direction and within the crack plane. Such conditions are not met with internal cracks like Type D. The other three methods, vee-and-weld, grinding, and peening, are limited to surface applications and are unsuitable for fatigue cracks that originate internally. Other approaches, such as carbon fiber reinforcement and single peak overloads [78], are also ineffective for addressing Type D cracks.

Therefore, there is an urgent need for new methods to prevent and control this type of fatigue damage. Several researchers have made initial attempts to address this challenge, which will be reviewed in detail in Section 1.3 “Previous Studies”.

1.2.4 Fatigue Evaluation Methods

As mentioned in Subsection 1.2.2, current detection methods for Type D fatigue cracks (Figure 1-15) are extremely limited. Therefore, accurately predicting the fatigue life of this structural detail is essential. If engineers can reliably estimate its fatigue life during the design phase, they can optimize the detail to ensure that fatigue cracks do not develop over the intended service life of the bridge. To this end, this subsection introduces four commonly used methods for evaluating the fatigue life of fillet weld roots: the nominal stress method, the structural stress method, the effective notch stress (ENS) method, and the equivalent displacement (ED) method.

1.2.4.1 Nominal Stress

The nominal stress method estimates stress based on external load parameters, including axial force and bending moment, and geometric properties, including cross-sectional area and moment of inertia. As shown in Figure 1-27, the nominal stress in the plate ($\sigma_{p,n}$) and in the weld ($\sigma_{w,n}$) can be calculated using the following equations, respectively.

$$\sigma_{p,n} = \frac{F}{A} + \frac{My}{I} \quad (1-5)$$

$$\sigma_{w,n} = \sqrt{\sigma_{\perp}^2 + \tau_{\perp}^2} \quad (1-6)$$

Where, F is the normal force; A is the cross-sectional area; M is the bending moment; $y = y_p - y_n$, with y_p being the vertical coordinate of the evaluation point and y_n that of the neutral axis; I is the moment of inertia; $\sigma_{\perp}$ and $\tau_{\perp}$ are the average normal and shear stresses in the weld, respectively.

In simple geometries, the weld stress components can be estimated using equilibrium conditions based on the nominal stress in the connected plate ($\sigma_{p,n}$). For more complex structures, they are typically evaluated through finite element analysis.

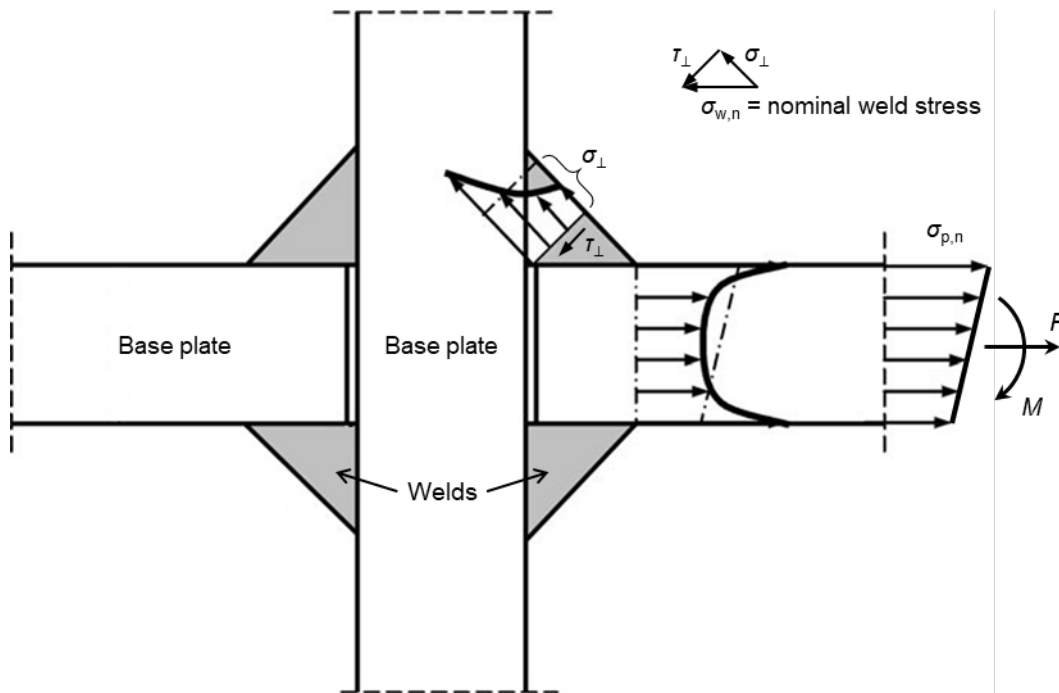


Figure 1-27 Nominal stresses in a cruciform joint [79]

The nominal stress method is widely used due to its simplicity. It accounts for certain secondary effects, such as misalignment, but it may neglect other influential factors like weld throat bending [79]. This method is thus usually applied only to the cases defined by standardized categories in design codes and fatigue assessment guidelines [80].

1.2.4.2 Structural Stress

Unlike nominal stress, structural stress accounts for stress increases caused by structural configurations, such as geometry-induced stress concentrations or additional

bending due to significant misalignment. However, it excludes local stress peaks induced by weld notches at the toe or root.

For weld toes, structural stress is typically evaluated by extrapolating surface stresses to the weld toe location, known as structural hot-spot stress [81]. However, surface stress extrapolation is unsuitable for evaluating fatigue at the weld root. Another method is through stress linearization across the thickness direction. Methods have been developed that utilize local nominal stress or structural weld stress based on internal force and moment distribution, or the stress field within the weld itself. These methods are sometimes limited to definite cases of application [79].

As shown in Figure 1-28, the structural stress in a fillet weld ($\sigma_{w,s}$) consists of two components.

$$\sigma_{w,s} = \sigma_{w,F} + \sigma_{w,M} \quad (1-7)$$

Two calculation approaches are commonly used for these components ($\sigma_{w,F}$ and $\sigma_{w,M}$), namely, using integral calculation or using equilibrium conditions.

(1) Integral calculation:

$$\sigma_{w,F} = \frac{1}{l} \int_0^l \sigma(y) dy \quad (1-8)$$

$$\sigma_{w,M} = \frac{6}{l^2} \int_0^l y \sigma(y) dy \quad (1-9)$$

Where, l is the weld leg length; $\sigma(y)$ is the normal stress along the weld leg; and y is the vertical coordinate. The shear stress ($\tau_{w,s}$) can be calculated similarly.

(2) Equilibrium conditions:

$$\sigma_{w,F} = \frac{F}{A} \quad (1-10)$$

$$\sigma_{w,M} = \frac{My}{I} \quad (1-11)$$

Where, the variables are defined as in Equation (1-5).

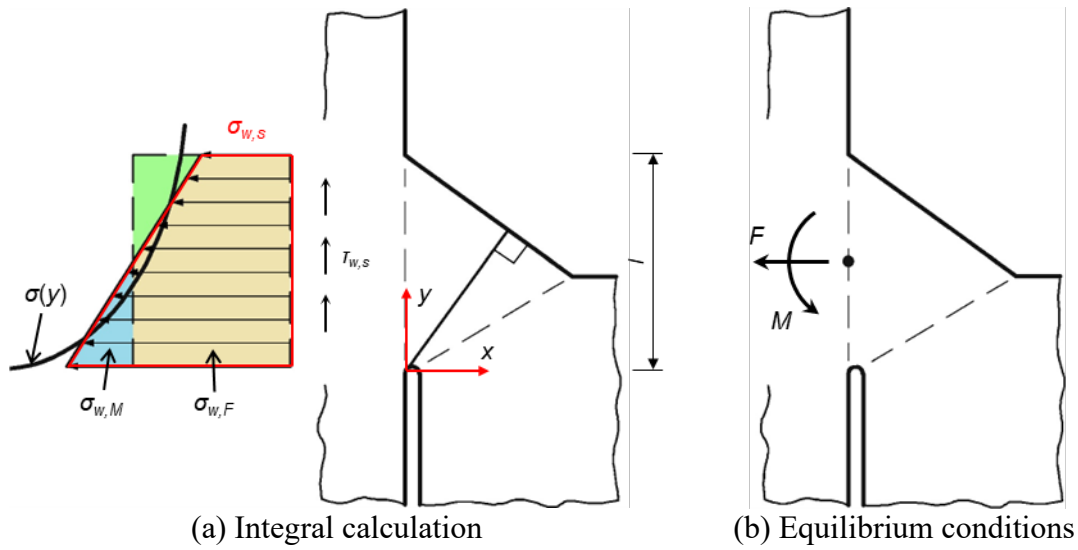


Figure 1-28 Structural stresses in a fillet weld [79]

Compared to the nominal stress method, the structural stress method offers better accuracy for assessing fatigue at the weld root. It clearly takes into account stress concentrations and weld throat bending caused by geometry or misalignment. However, this method still depends on simplified assumptions, such as averaged or linearized stress distributions within the weld. Its use has been confirmed in several case studies, but it is still limited to clearly defined situations.

1.2.4.3 Effective Notch Stress

The effective notch stress (ENS) method considers the local stress concentration at the weld toe or weld root using linear elastic theory. It does not include elastic-plastic material behavior. You can factor in the material's microstructural support effect, which shows how stress gradients impact fatigue behavior in uneven materials, by averaging the stress over a defined area.

The ENS method usually uses fictitious notch rounding, as shown in Figure 1-29 [82]. The basic concept is that the stress reduction from averaging over a small depth can be represented by adding a fictitious notch. Radaj suggested a conservative reference notch radius of $r_{\text{ref}} = 1 \text{ mm}$ for welded steel joints, and this was later extended to aluminum and magnesium alloys [83].

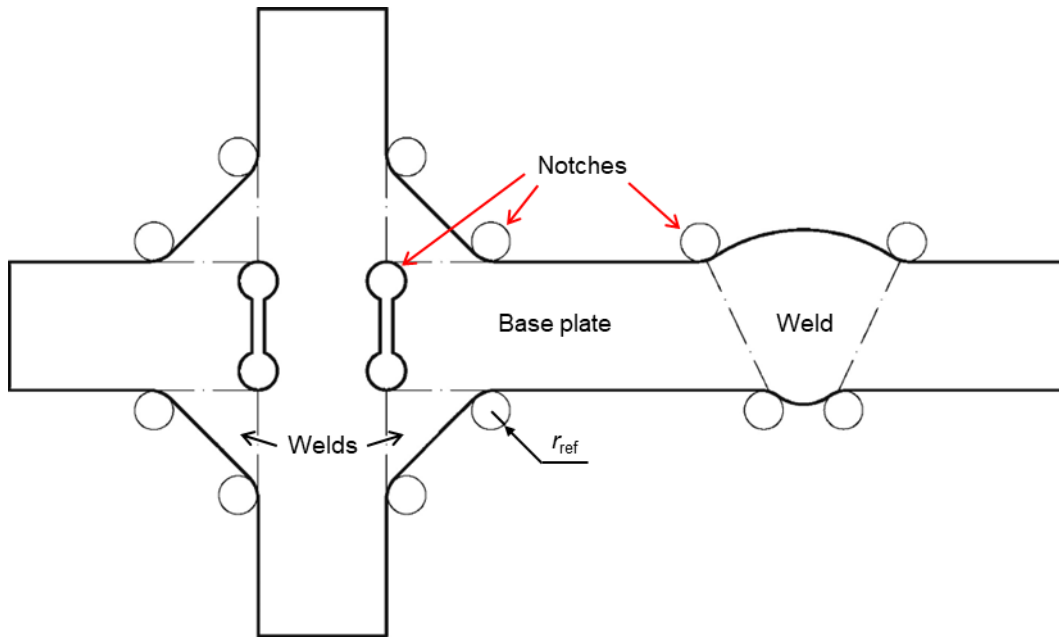


Figure 1-29 Fictitious notches in a cruciform joint and a butt joint [80]

The ENS method is mainly used through finite element analysis. An artificial circular notch is introduced at the stress singularity. The principal or von Mises stress around this notch serves as the foundation for evaluating the fatigue life.

The ENS method offers a major advantage over nominal and structural stress methods by explicitly considering local stress concentrations. It incorporates key influencing factors such as weld throat thickness and local geometric details. However, the weld geometry still needs to be idealized, especially at the weld root. Moreover, effects like residual stress or geometric irregularities are indirectly accounted for through the selected S-N curve [79].

1.2.4.4 Equivalent Displacement

The previous three methods are all stress-based methods that utilize S-N curves to evaluate the fatigue strength or life of structures or joints. However, these stress-based criteria have their limitations. Consequently, Tateishi et al. developed a displacement-based fatigue evaluation method known as the equivalent displacement (ED) method [84].

As illustrated in Figure 1-30, four nodes are selected near the weld root for displacement calculation. Nodes A_0 and B_0 are located at the weld root, while nodes A and

B are located 1 mm away in the unwelded region — node A on the sole plate and node B on the flange plate. The displacement components u and v represent the displacements in the x - and y -directions, respectively.

$$u = |(u_A - u_{A_0}) - (u_B - u_{B_0})| \quad (1-12)$$

$$v = |(v_A - v_{A_0}) - (v_B - v_{B_0})| \quad (1-13)$$

Where, the subscripts denote the corresponding nodes.

The displacement components u and v correspond to Mode II and Mode I loading (see Figure 1-6), respectively.

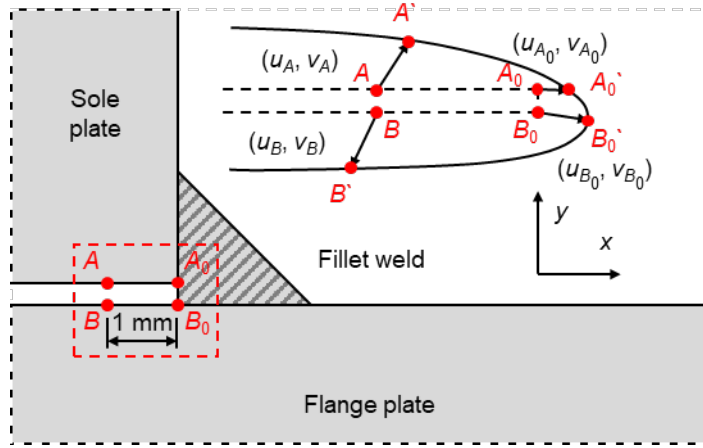


Figure 1-30 Diagram of equivalent displacement

The equivalent displacement (d) is calculated by the following equation.

$$d = \sqrt{3u^2 + v^2} \quad (1-14)$$

Because the ED method is displacement-based, it is less sensitive to mesh size, resulting in relatively stable outputs [84]. Additionally, the method allows modeling based on idealized structural configurations, which improves efficiency. However, its applicability has not yet been validated across a wide range of conditions, and it does not consider the impact of residual stress on fatigue life.

1.3 Previous Studies

Fatigue cracks that initiate within structures, such as the Type D crack shown in Figure 1-15, need special attention because effective detection methods are lacking. To

tackle this problem, several studies have aimed to improve the fatigue strength of these structural details and to retard or prevent the initiation of fatigue cracks. This section gives a short overview of these research efforts.

1.3.1 Bonding-assisted Welding

In 2016, Hirohata proposed a technique for strengthening welded joints using epoxy resin bonding, referred to as bond-assisted welding, as depicted in Figure 1-31. This technique involves first bonding steel plates together with epoxy, followed by welding. Compared to conventional welded joints, bonding-assisted welded joints exhibited an increase in elastic limit of approximately 70 N/mm² and a 12% improvement in joint stiffness, although the difference in ultimate strength was negligible [85]. In 2018, Hirohata et al. applied this technique to improve the fatigue performance of patched plate joints. Experimental results showed that under tensile loading at the weld root, the fatigue life of bonding-assisted welded specimens was at least four times greater than that of as-welded joints [86].

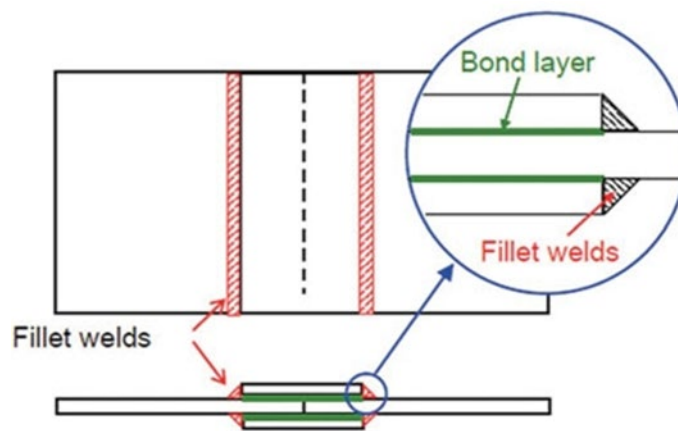


Figure 1-31 Bonding-assisted welded joint [85]

Despite the notable improvement in fatigue life, bonding-assisted welding exposed the problem of being prone to weld defects. As shown in Figure 1-32, the high heat input during welding can burn the epoxy, compromising its bonding performance. The resulting combustion may also release gases, leading to defects such as incomplete weld root penetration [85, 86]. Therefore, further research is needed to address the challenge of weld defects associated with this technique.

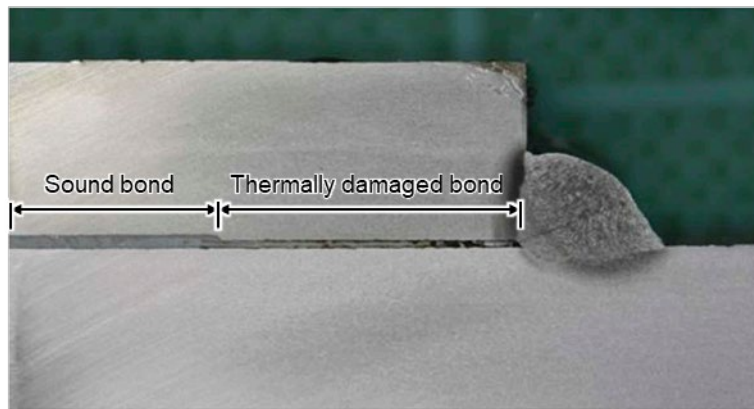
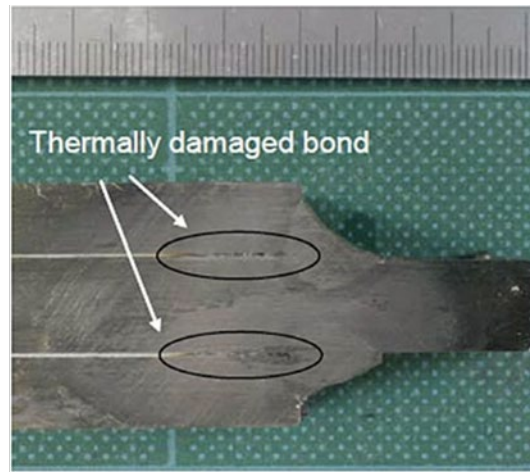


Figure 1-32 Thermal damage of epoxy layer [85, 86]

1.3.2 Patch Plates

In 2024, Jiang et al. proposed a patch plate reinforcement scheme targeting root-to-throat cracks (Figure 1-33) in the fillet welds between U-ribs and the deck of orthotropic steel decks. Since the interior of U-ribs is typically inaccessible for inspection, these fatigue cracks are also regarded as initiating from within the structure. Jiang et al. introduced a reinforcement strategy using epoxy resin to bond angle steel plates to the exterior surfaces of the U-ribs, as illustrated in Figure 1-34. Both experimental and numerical results demonstrated that this reinforcement method reduced fatigue stress by over 90%, thereby significantly enhancing fatigue life [87].

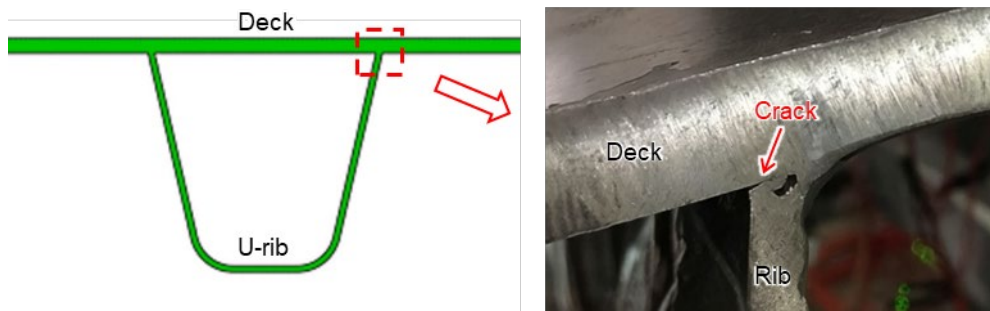


Figure 1-33 Root-to-throat cracks [87]

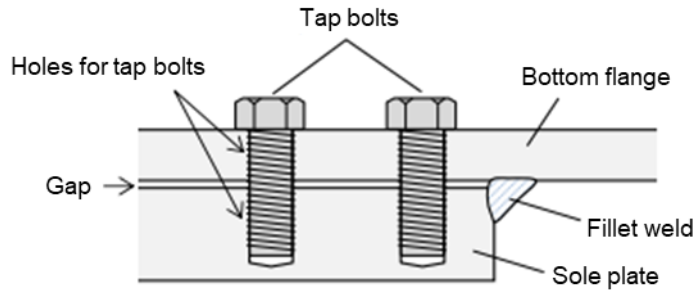


Figure 1-34 Strengthened specimen [87]

This method presents a novel bonding-based approach for preventing internal fatigue cracks and has shown excellent performance in orthotropic steel decks. However, it may not be suitable for regions near bridge supports, where the sole plate does not offer sufficient bonding area to effectively constrain deformation. Therefore, an alternative preventive strategy is needed for such cases.

1.3.3 Tap Bolts

In 2024, Niwa et al. proposed a method for preventing fatigue cracks near bridge supports by adding tap bolts, as shown in Figure 1-35. By constraining the relative displacement between the bottom flange and the sole plate, tap bolts effectively reduce stress amplitude at the weld root. This method is especially useful for dealing with Type D cracks shown in Figure 1-15. Experimental and simulation results suggest that this approach can extend the fatigue life of the weld root by about ten times, making it a practical solution [88].



(a) Schematic diagram of tap bolts



(b) Drilling holes



(c) After strengthening

Figure 1-35 Prevention method through tap bolts [88]

However, this method requires drilling additional threaded holes. It relies on the interlocking of bolt and hole threads to limit movement, which needs high machining precision. Additionally, the presence of holes may lower the load-bearing capacity of the bottom flange and sole plate, posing certain limitations.

1.4 Objective

In general, there is no effective method for detecting Type D fatigue cracks, as shown in Figure 1-15. As a result, if these cracks occur in bridges, they often go undetected until they reach the surface. By that time, the safety of the structure may already be at risk. Therefore, it is important to focus on preventive measures during design and construction to reduce the risk of these internal fatigue cracks.

Furthermore, most current techniques for repairing or retarding fatigue cracks do not work well for Type D cracks. Although recent studies have suggested new methods like bonding-assisted welding, patch plates, and tap bolts, these approaches still exhibit practical limitations.

This study aims to address these problems by adjusting structural design and improving the bonding-assisted welding technique to prevent Type D fatigue cracks. Through a series of experimental testing, numerical simulation, and theoretical analysis, this research seeks to offer better prevention strategies for internal fatigue cracks. The research gap addressed in this study is the use of non-destructive methods to improve the fatigue performance of weld roots near bridge bearings.

1.5 Outline

To achieve the objectives outlined above, a series of investigations was conducted and compiled into this dissertation, which consists of six chapters. The overall structure is shown in Figure 1-36. The bonding-assisted welding methods discussed in this dissertation are divided into two categories: adhesive insertion and soft metal insertion. These are covered in Chapters 3 and 4, respectively. The classification of these two categories is shown in Figure 1-37.

Chapter 1 introduces the background and motivation for this study. It reviews relevant literature and sets the research objectives.

Chapter 2 explores methods and limitations for improving the fatigue life of the weld root at the fillet weld between the bottom flange and the sole plate from a structural design perspective. Finite element models of five representative steel or steel-concrete composite bridges were created. The ENS method was used to evaluate the fatigue life of the weld root in the original bridge designs. This chapter examined the influence of weld size, sole plate thickness, and bolt installation.

Chapter 3 introduces an improvement to the existing bonding-assisted welding method using epoxy resin. A heat-resistant rubber barrier is added to confine the epoxy, which reduces thermal damage and weld defects. Experiments were conducted to verify the feasibility and effectiveness of this method.

Chapter 4 introduces a new bonding-assisted welding method that uses soft metal instead of epoxy. Due to its higher heat resistance, soft metal can minimize the risk of welding defects. The feasibility and effectiveness of soft metal insertion were experimentally investigated.

Chapter 5 discusses the applicability of bonding-assisted welding to actual bridge construction and provides optimization strategies. Simulation analysis was conducted on the representative bridges from Chapter 2 to assess the improvement in weld root fatigue life achieved through bonding-assisted welding.

Chapter 6 concludes the dissertation by summarizing the key findings and proposing directions for future research.

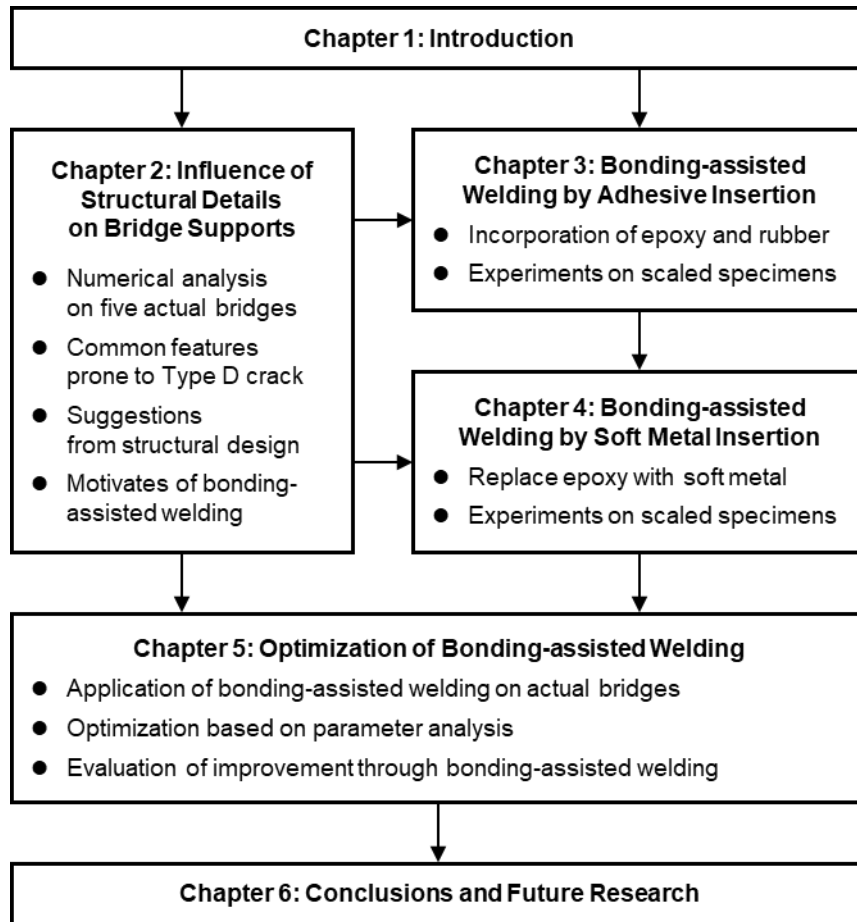


Figure 1-36 Outline of the dissertation

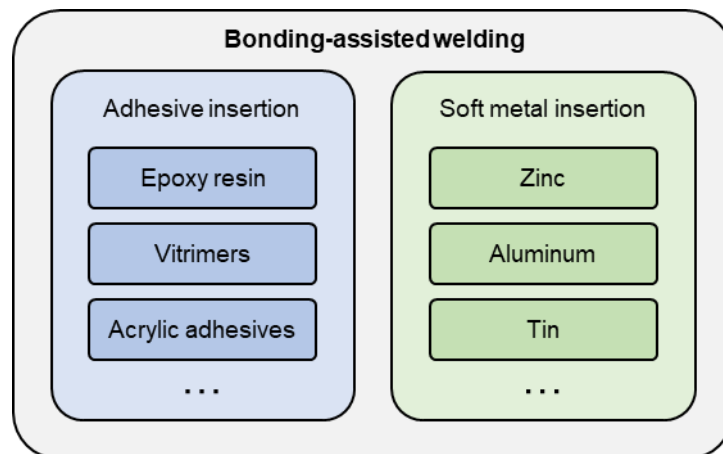


Figure 1-37 Classification of bonding-assisted welding

CHAPTER 2

INFLUENCE OF STRUCTURAL DETAILS ON BRIDGE SUPPORTS

2.1 Introduction

To investigate the specific causes of Type D crack (Figure 1-15), this chapter analyzes the loading conditions and stress distribution around bridge supports through finite element modeling based on actual bridge structures. Fatigue analysis of the fillet weld root between the bottom flange and the sole plate was conducted using the ENS method described in Subsubsection 1.2.4.3 “Effective Notch Stress”. Common design features prone to Type D cracking were identified, and potential improvements were proposed from a structural design perspective, including fillet weld size, sole plate thickness, and bolt installation. The limitations of adjusting the structural design alone were also identified, as this is insufficient to eliminate Type D cracks in some bridges. This provided the motivation for the subsequent studies into bonding-assisted welding.

2.2 Bridge Information

Four actual highway steel-concrete composite bridges and one railway steel bridge were selected to evaluate the fatigue life of weld roots around bridge supports. In the composite bridges, the girders are made of steel, while the decks are made of concrete. Basic information about the selected bridges is in Table 2-1. The five bridges located in the United States, China, and Japan. All five bridges adopt I-girder configurations, with the number of girders ranging from two to seven, as shown in Figure 2-1. Structural details at the cross-section of the intermediate supports are summarized in Table 2-2. The configurations of the bridge supports are shown in Figure 2-2, including dimensional definitions. The bottom flange, sole plate, and bearing are joined through both welding and bolting. The sole plate is fillet-welded to the bottom flange all around.

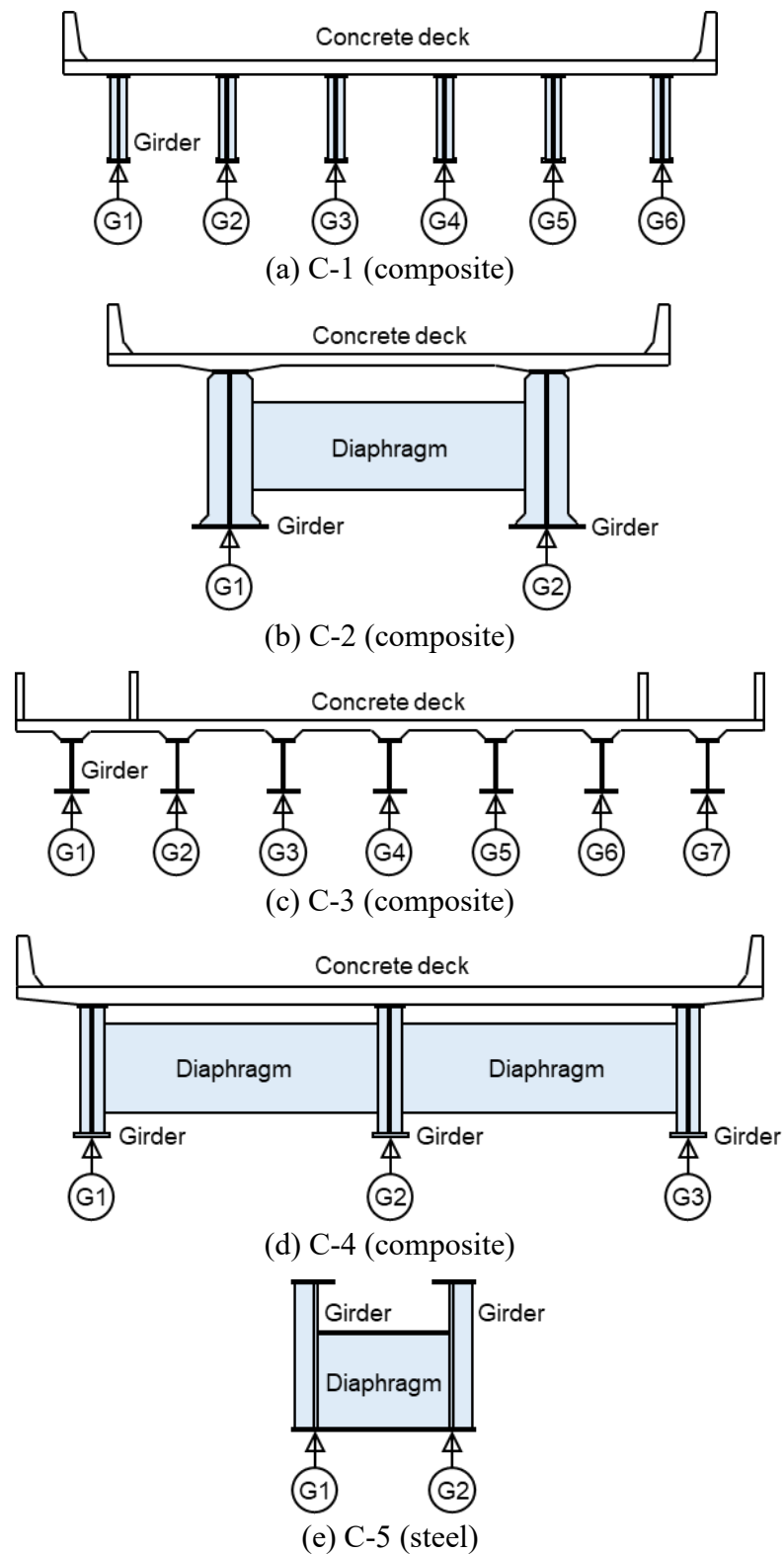


Figure 2-1 Cross-sections of the five actual bridges (unit: mm)

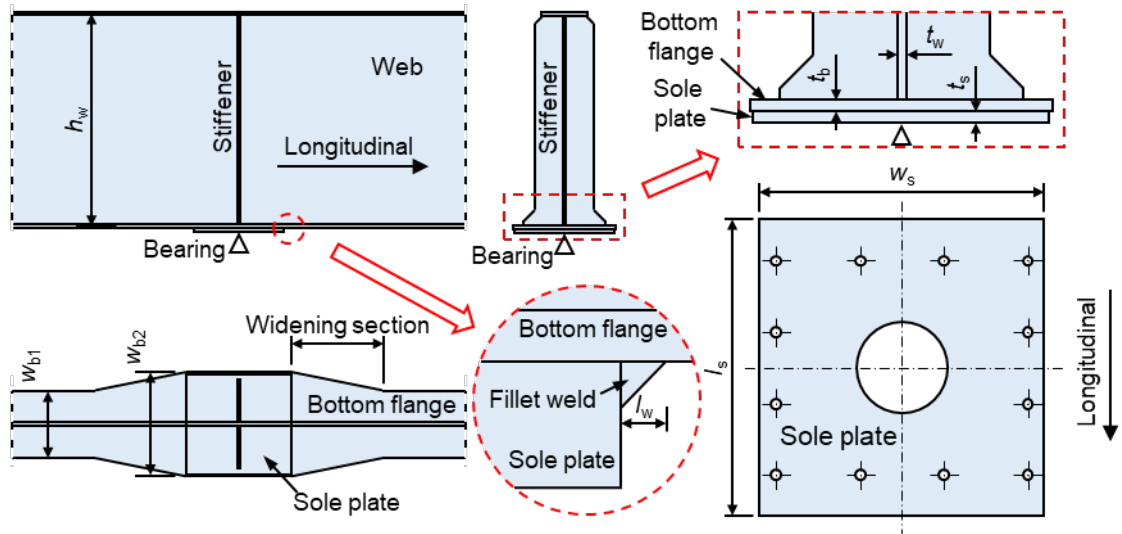


Figure 2-2 Structural details near the bridge supports

Table 2-1 Basic information of the five actual bridges

Bridge ID	Bridge Type	Span design			Bridge width [m]	Number of girders
		Number	Length [m]	Total [m]		
C-1	Highway	3	30.0 + 36.0 + 30.0	96.0	14.20	6
C-2	Highway	3	48.9 + 41.0 + 53.9	143.8	10.14	2
C-3	Highway	4	4 × 20.0	80.0	14.00	7
C-4	Highway	5	34.9 + 3 × 38.0 + 34.9	183.8	14.75	3
C-5	Railway	3	30.0 + 40.0 + 30.0	100.0	1.70	2

Table 2-2 Structural dimensions of intermediate supports (unit: mm)

Bridge ID	Web		Bottom flange		Sole plate			Weld leg
	Height h_w	Thickness t_w	Width $w_{b1}(w_{b2})$	Thickness t_b	Length l_s	Width w_s	Thickness t_s	Length l_w
C-1	1500	20	400	20	280	480	40	10
C-2	2800	20	870 (1350)	34	1370	1320	50	10
C-3	900	14	600	32	340	340	30	16
C-4	2200	12	620 (740)	35	700	700	39	10
C-5	1300	12	850 (1010)	22	1000	680	50	10

Note: Values in parentheses is the widened width near the support.

2.3 Finite Element Models

This research focuses on the behavior of weld roots near bridge supports. Using a high-resolution model is crucial for fatigue evaluation. However, creating such a detailed model for the whole bridge requires excessive resources and time. To overcome this, a two-stage modeling strategy was employed: a coarse global girder model and a refined local support model. The coarse model was first analyzed to capture the stress patterns around the supports. These stress data were then applied to the refined model to assess weld root stresses. This hybrid approach balances between computational efficiency and the accuracy required for detailed analysis.

2.3.1 Girder Models

The girder models were developed to evaluate stress distributions in the vicinity of the supports. The five representative bridges under study each have between two and seven girders. For computational efficiency, single-girder models were employed, with scaled loads, as shown in Figure 2-3. Loads were determined following the relevant national design codes: AASHTO LRFD Bridge Design Specifications (USA) [89], JTG D60-2015 General Specifications for Design of Highway Bridges and Culverts (China) [90], TB 10002-2017 Code for Design on Railway Bridge and Culvert (China) [91] and Specifications for Highway Bridges (Japan) [92]. The fatigue load case included both dead load and vehicular (or train) load, with load factors set to 1.0.

Each model comprises three-dimensional eight-node cubic elements with reduced integration (C3D8R). The mesh adopted a global size of 200 mm. The corresponding numbers of nodes and elements are given in Figure 2-3. The analysis focused on intermediate support. Concrete tensile strength was neglected, and only the steel girder was modeled, while the concrete deck was replaced by an equivalent uniformly distributed load acting on the top flange. To simplify the geometry, diaphragms, stiffeners, longitudinal slope, transverse slope, and curvature were excluded. Boundary conditions assumed one pinned support at one end and roller supports at the others. Linear elastic material properties were applied, with steel characterized by an elastic modulus of $2.06 \times 10^5 \text{ N/mm}^2$ and a Poisson's ratio of 0.3. The support showing the highest fatigue

stress was identified on each bridge, and corresponding refined local models were developed for further study.

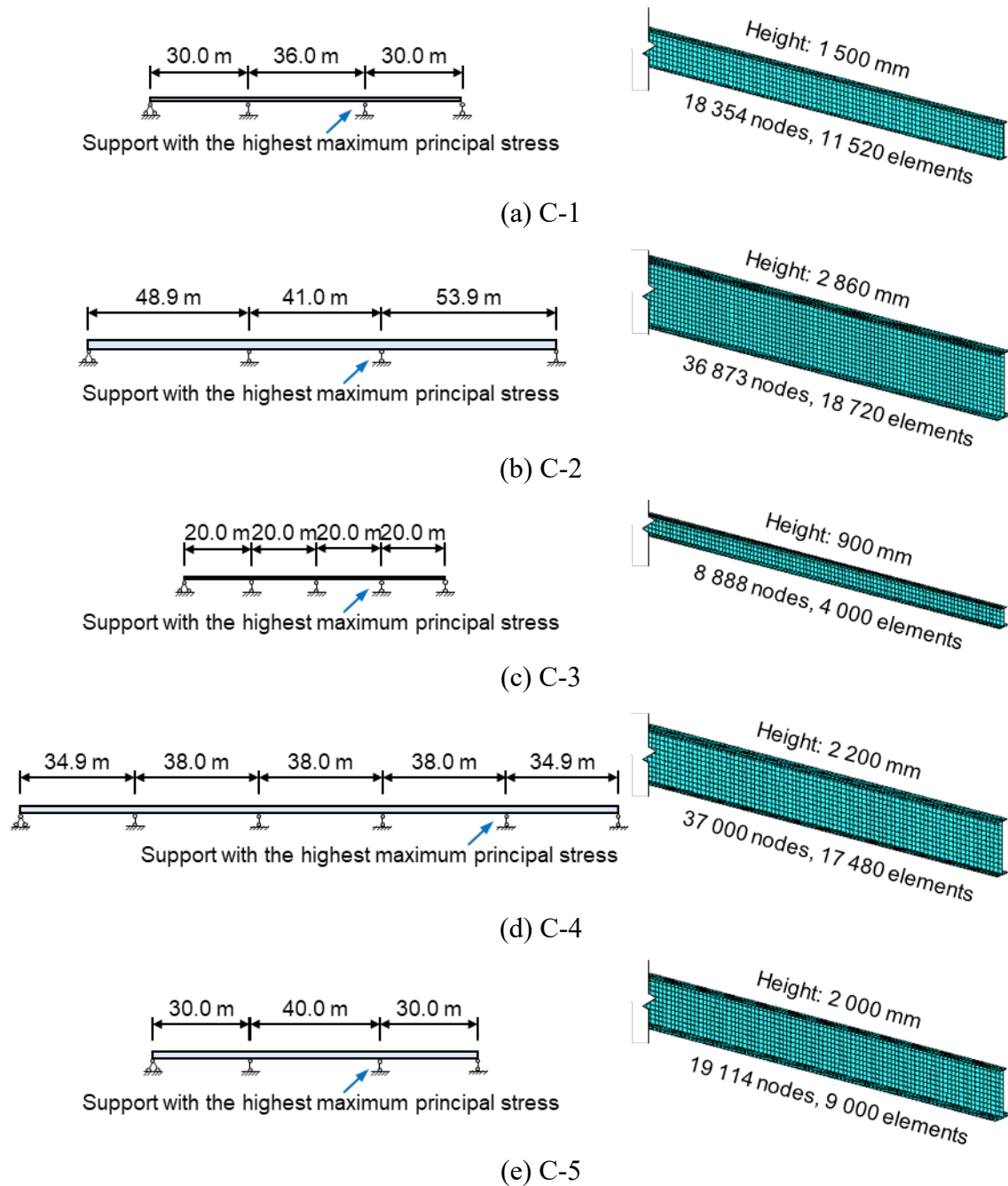


Figure 2-3 Overview of the girder models

2.3.2 Support Models

2.3.2.1 Meshing Strategies

The geometry around bridge supports is intricate, especially at the weld root that connects the sole plate to the bottom flange. This area experiences significant stress concentration. Because of this, the nominal stress method cannot reliably predict weld root fatigue strength in such area [93]. In finite element simulations, stresses at the weld root are strongly affected by mesh density, a phenomenon often referred to as stress singularity. To overcome this, Radaj D. et al. proposed the ENS method, which was derived from Neuber H.'s concept of microstructural support [83]. In this approach, an artificial circular notch is inserted at the singularity, and the fatigue strength is evaluated using the stress around the notch. A standard notch radius of $r_{\text{ref}} = 1 \text{ mm}$ is typically adopted, as illustrated in Figure 2-4. Although the ENS method is suitable for weld root fatigue analysis, it also has limitations. For example, it does not well account for the influence of residual stress and increases the modeling workload.

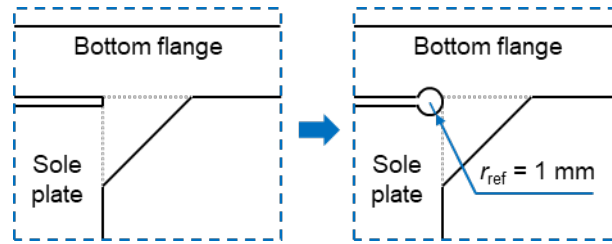


Figure 2-4 Effective notch stress method

A series of high-precision models of selected bridge supports were built in ABAQUS software. As shown in Figure 2-5, these models capture the local support region of the girder, and symmetry was applied to create a half-model. For simplification, longitudinal and transverse slopes as well as bridge curvature were omitted. At the weld root between the sole plate and the bottom flange, a notch with radius $r_{\text{ref}} = 1 \text{ mm}$ was introduced for ENS evaluation. The meshes were generated using C3D8R element. Around the notch, an element size of 0.2 mm was applied, gradually coarsening to 20 mm in the surrounding region. A mesh refinement study confirmed convergence: reducing the notch mesh to 0.1 mm changed the ENS value by less than 1%. All analyses assumed linear elasticity,

with material parameters consistent with those in the girder models. The weld and base metals were modeled with identical mechanical properties.

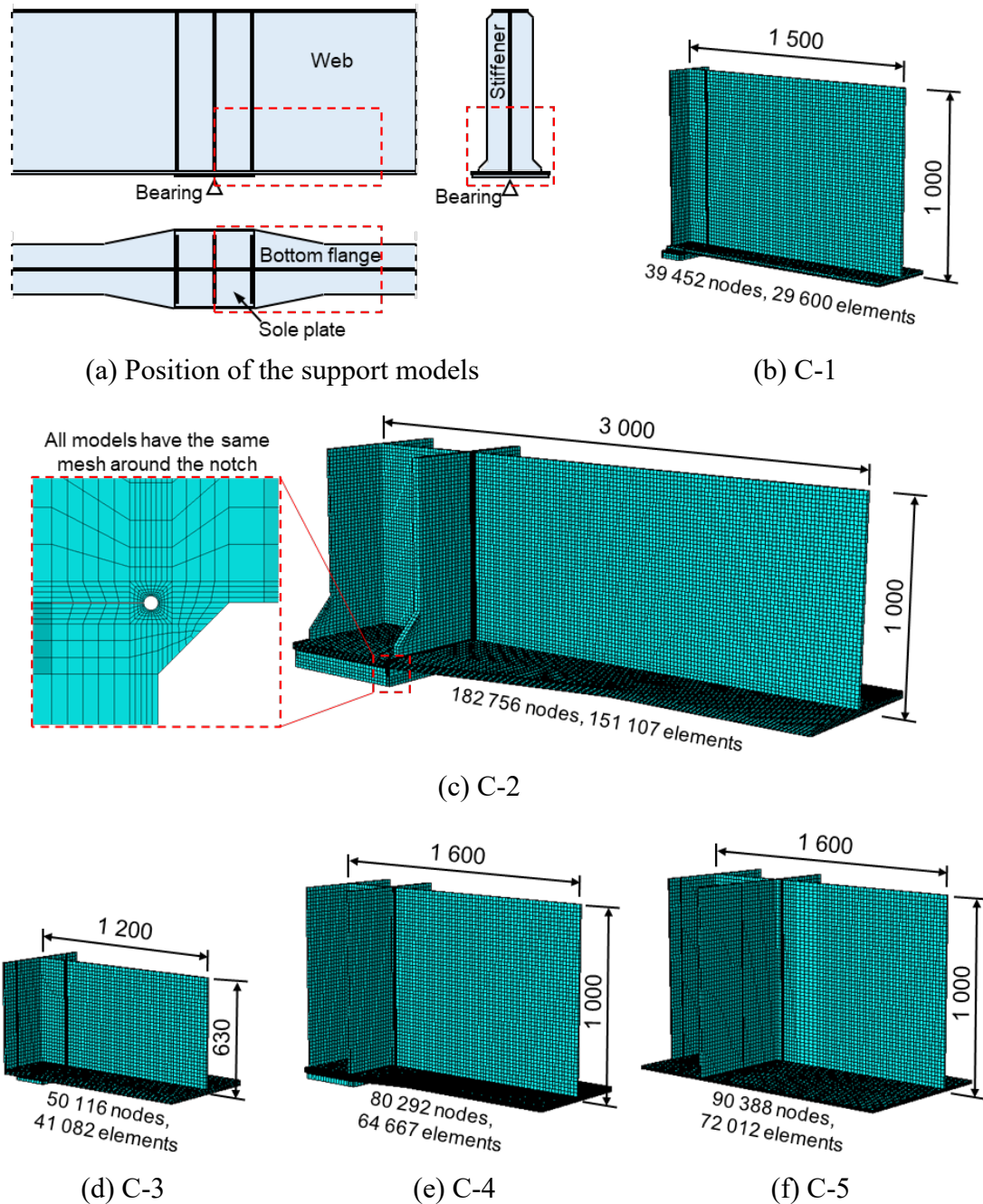


Figure 2-5 Overview of the support models

2.3.2.2 Boundaries and Loads

The boundaries and applied loads of the support models are shown in Figure 2-6. Symmetry was imposed along the central vertical plane, while the sole plate was constrained with a pin support. The normal and shear loads were simplified as distributed loads with linear variation applied on the cut surfaces of the support model. These load distributions were obtained from the girder model and can be expressed as:

$$N_x(y) = k_1 y + \sigma_1 \quad (2-1)$$

$$N_y(x) = k_2 x + \sigma_2 \quad (2-2)$$

$$S_{xy}(y) = k_3 y + \tau_1 \quad (2-3)$$

$$S_{yx}(x) = k_4 x + \tau_2 \quad (2-4)$$

Where, $N_x(y)$, $N_y(x)$, $S_{xy}(y)$, and $S_{yx}(x)$ denote the distributed normal and shear stresses in the x - and y -directions, respectively. The coefficients k_1 , k_2 , k_3 , k_4 , σ_1 , σ_2 , τ_1 , and τ_2 are load constants derived from the girder models, with their values listed in Table 2-3.

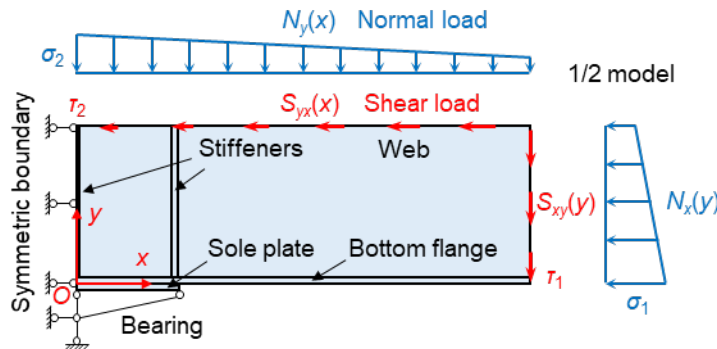


Figure 2-6 Boundaries and applied loads of the support models

Table 2-3 Values of load constants for each bridge

Bridge ID	$N_x(y)$		$N_y(x)$		$S_{xy}(y)$		$S_{yx}(x)$	
	k_1	σ_1	k_2	σ_2	k_3	τ_1	k_4	τ_2
C-1	0.0719	-54.172	0.0024	-1.600	-0.0059	-3.324	-0.0038	-3.429
C-2	0.0401	-51.858	0.0133	-30.561	-0.0151	-5.592	-0.0011	-17.219
C-3	0.0941	-27.998	0.0076	-7.893	-0.0167	-4.682	-0.0061	-7.582
C-4	0.0514	-60.092	0.0269	-39.122	-0.0195	-6.338	-0.0091	-12.796
C-5	0.0562	-50.747	0.0184	-26.016	-0.0144	-4.991	-0.0075	-8.422

2.3.2.3 Simulation of Bolts

The bolts were not explicitly modeled using solid elements. Instead, their behavior was represented through displacement constraint equations applied to paired nodes, as shown in Figure 2-7. Bolt pretension was not considered. Bolts were assumed to restrict the relative displacement between the bottom flange and the sole plate. The relative displacement vector $\vec{d}$ is defined as:

$$\vec{d} = \vec{d}_A - \vec{d}_B \quad (2-5)$$

Where $\vec{d}_A$ and $\vec{d}_B$ denote the displacement vectors of the paired nodes on the bottom flange and the sole plate, respectively, expressed as:

$$\vec{d}_A = (u_A, v_A, w_A) \quad (2-6)$$

$$\vec{d}_B = (u_B, v_B, w_B) \quad (2-7)$$

Where u , v , and w denote the displacement components in the x -, y -, and z -directions, respectively.

The bolt constraints were separated into normal and tangential components. In the normal constraints, the bolts were modeled as tension-only connections, permitting compressive deformation. To prevent penetration under compression, “hard” contact was specified in ABAQUS at the bottom flange–sole plate interface. In the tangential constraints, the bolts were regarded as rigid connections, fully restricting relative displacement between paired nodes. The equations of constraints are summarized as:

$$\begin{cases} u_A - u_B = 0 \\ v_A - v_B \leq 0 \\ w_A - w_B = 0 \end{cases} \quad (2-8)$$

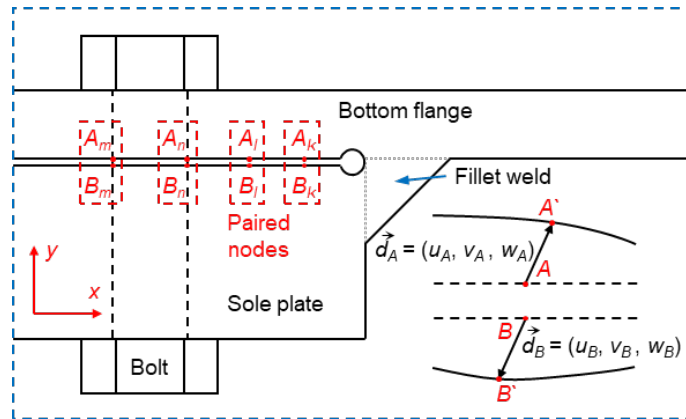


Figure 2-7 Relative displacement between paired nodes

2.4 Fatigue Evaluation Results

2.4.1 Results of Original Design

Local finite element models of the intermediate supports were used to investigate the ENS at the weld roots connecting the bottom flange and sole plate in five bridges. Residual stress was not considered. Fatigue life estimation was carried out according to the FAT 225 design curve (Figure 2-8), as specified in the IIW Recommendations [80]. Figure 2-9 presents the calculated ENS values and the corresponding fatigue life predictions. Among the five cases, Bridge C-3 showed the lowest ENS of 127.8 N/mm², while three others exceeded 225 N/mm², suggesting fatigue lives shorter than two million cycles. Particularly high stress values were found in Bridge C-2 (331.7 N/mm²) and Bridge C-4 (275.9 N/mm²), which correspond to predicted fatigue lives of only 6.24×10^5 and 10.85×10^5 cycles, respectively. Because both Bridges C-2 and C-4 are highway bridges, the high stress levels indicate a considerable risk of fatigue cracking under heavy traffic, which could eventually lead to increased maintenance requirements.

Although Bridges C-1 to C-4 have comparable deck widths and live loads, their ENS results vary greatly. Bridges C-1 and C-3, which adopt multiple smaller girders, exhibited much lower ENS values compared with Bridges C-2 and C-4. Additionally, the weld root ENS of Bridge C-2 was about 120% of Bridge C-4. While large-girder designs simplify fabrication and maintenance, they lead to higher support loads, which shortens the fatigue life of the weld roots. A closer comparison between Bridges C-1 and C-3 reveals another important factor. Despite only a slight difference in the number of girders, Bridge C-3 showed an ENS 45% lower than that of Bridge C-1. The key structural difference is the fillet weld size. Bridge C-1 used a 10 mm weld leg, while Bridge C-3 employed a 16 mm weld leg. The effect of fillet weld size on fatigue resistance is discussed in the next section in detail. Bridge C-5 is a railway bridge. Although its dead load is lower than that of the other four highway bridges, its live load is larger and accounts for a greater proportion of the total applied load. Consequently, the weld root ENS of Bridge C-5 is relatively high, meaning fatigue cracking between the bottom flange and the sole plate remains a concern.

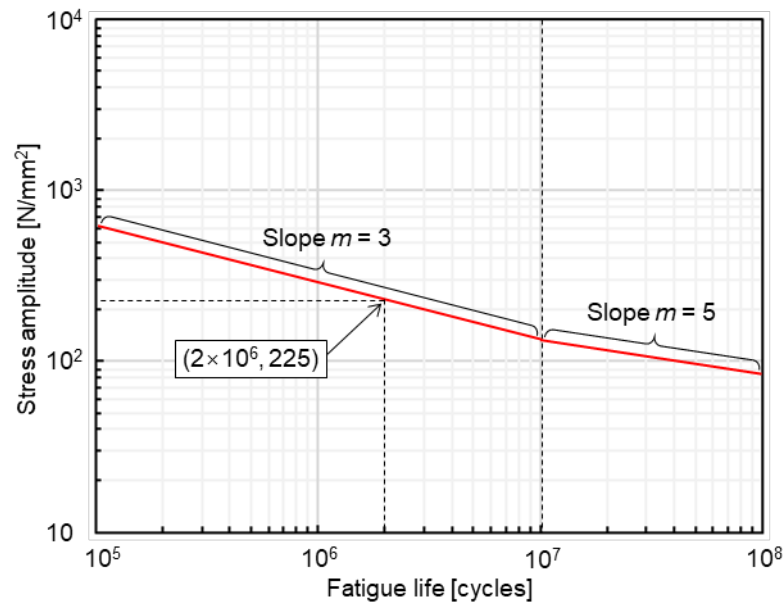


Figure 2-8 FAT 225 design curve

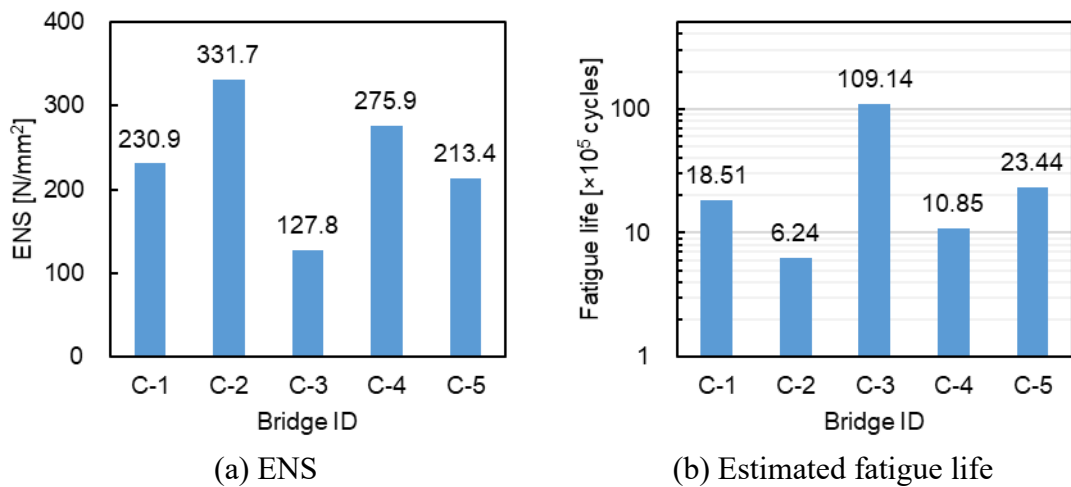


Figure 2-9 Fatigue evaluation result of selected bridges

Overall, these results demonstrate that many bridges face fatigue risk at the fillet weld root of the bottom flange–sole plate connection. To solve the problem, parametric analyses will be carried out to evaluate potential design improvements. Since Bridge C-2 showed the highest ENS, it is chosen as the reference model for the following parametric study.

2.4.2 Structural Design

In girder design, the primary focus is typically on ensuring sufficient load-bearing capacity, which makes altering the overall girder configuration for the sole purpose of improving weld root fatigue performance unrealistic. A more practical strategy is to refine the details of the fillet welds and sole plate. Analysis of the five case-study bridges indicated that fillet weld size has a strong impact on the fatigue behavior at the weld root. Additionally, the five bridges also showed noticeable differences in sole plate thickness. The thickness ratio of the sole plate to the bottom flange may have effect on relative stiffness and the stress distribution at the weld root. This subsection investigates how the leg length of fillet weld l_w and the sole plate thickness t_s affect fatigue resistance of the weld root. Furthermore, the role of bolts in hybrid bolted–welded connections is evaluated with respect to their effect on ENS at the weld root. The definitions of these key parameters are shown in Figure 2-10.

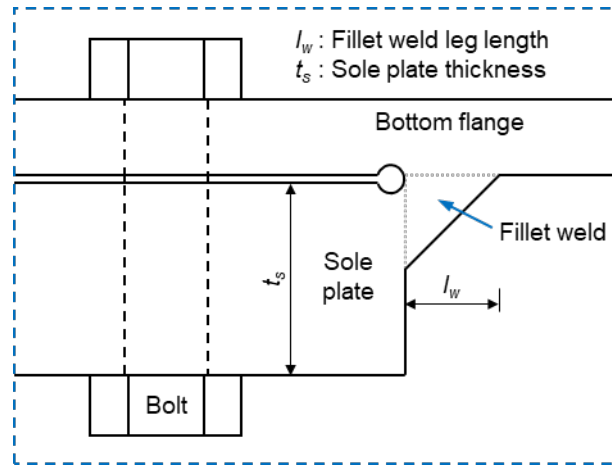


Figure 2-10 Key parameters for structural design

2.4.2.1 Effect of Weld Size

Using Bridge C-2 as the reference case, the weld size was examined by varying the fillet weld leg length l_w . The parametric range extended from 6 mm to 20 mm while the original design specifies a 10 mm leg length. The resulting weld root ENS values and the corresponding fatigue life estimates are demonstrated in Figure 2-11. A clear trend was observed that the ENS consistently decreased as l_w increased. At a leg length of 20 mm,

the weld root ENS dropped to 227.1 N/mm², about 68% of the baseline value. This reduction led to a threefold increase in predicted fatigue life, although it still did not reach two million cycles. Despite these benefits, enlarging the weld size does have some drawbacks. The improvement effect diminishes as welds become larger, and oversized welds are difficult to achieve with single-pass welding. They also complicate quality control, while raising construction challenges and costs. Therefore, although increasing weld size can improve fatigue resistance to some degree, the method alone cannot ensure long-term prevention of weld root cracking in bridge structures where fatigue life is already insufficient.

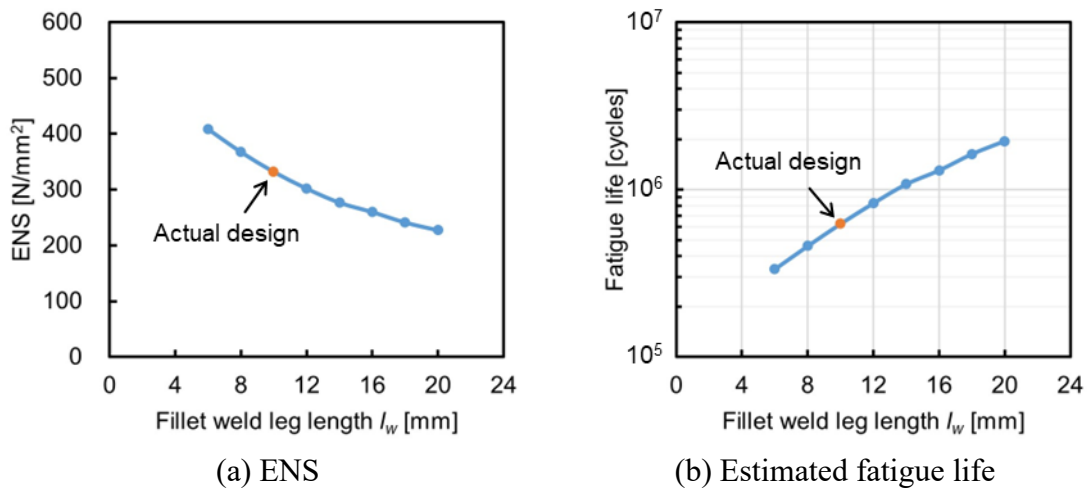


Figure 2-11 Influence of weld size on ENS and fatigue life

2.4.2.2 Effect of Sole Plate Thickness

The effect of sole plate thickness t_s on weld root ENS and the corresponding fatigue life is illustrated in Figure 2-12. In the parametric study, the sole plate thickness t_s ranged between 30 mm and 100 mm, compared with the 50 mm thickness adopted in the original bridge design. The results show that increasing the sole plate thickness produces only a marginal reduction in ENS. Even when the thickness is more than tripled, the weld root ENS decreases by less than 2%, leading to a fatigue life improvement of under 6%. These findings suggest that increasing the sole plate thickness t_s has little effect in improving weld root fatigue resistance. On the contrary, since its influence is minimal, an appropriate reduction in sole plate thickness might be acceptable in practice without significantly compromising fatigue performance.

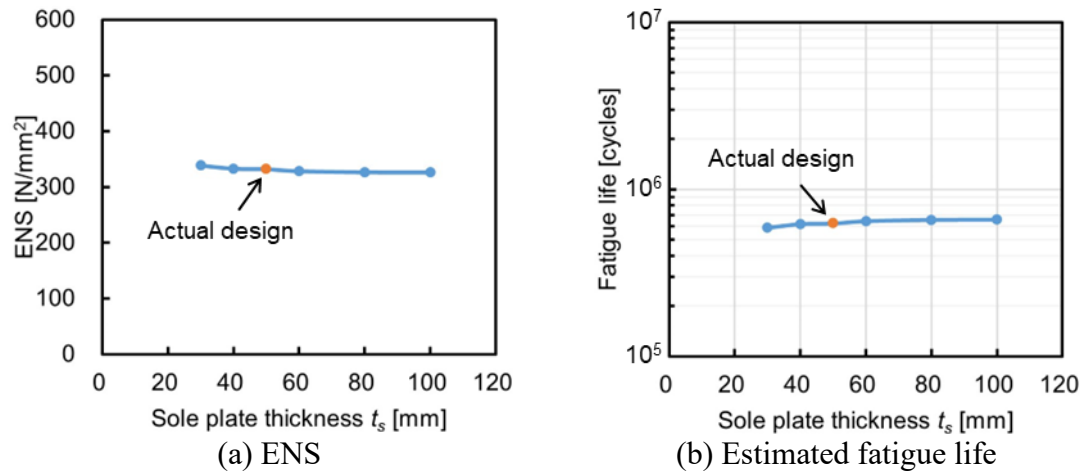


Figure 2-12 Influence of sole plate thickness on ENS and fatigue life

2.4.2.3 Effect of Bolts

The comparison of weld root fatigue performance with and without bolts for the five bridges is shown in Figure 2-13. Across all cases, models that considering only welding exhibited higher ENS values at the weld root than those with hybrid bolting–welding connections. The increases ranged from 2% to 26%. This rise in ENS correspondingly reduced the predicted fatigue life. Adding bolts effectively restricts relative displacement between the bottom flange and the sole plate, thereby lowering ENS and improving fatigue resistance. The effect is particularly notable for Bridges C-2, C-4, and C-5, which adopt bigger sole plates and therefore experience greater relative displacement compared with Bridges C-1 and C-3.

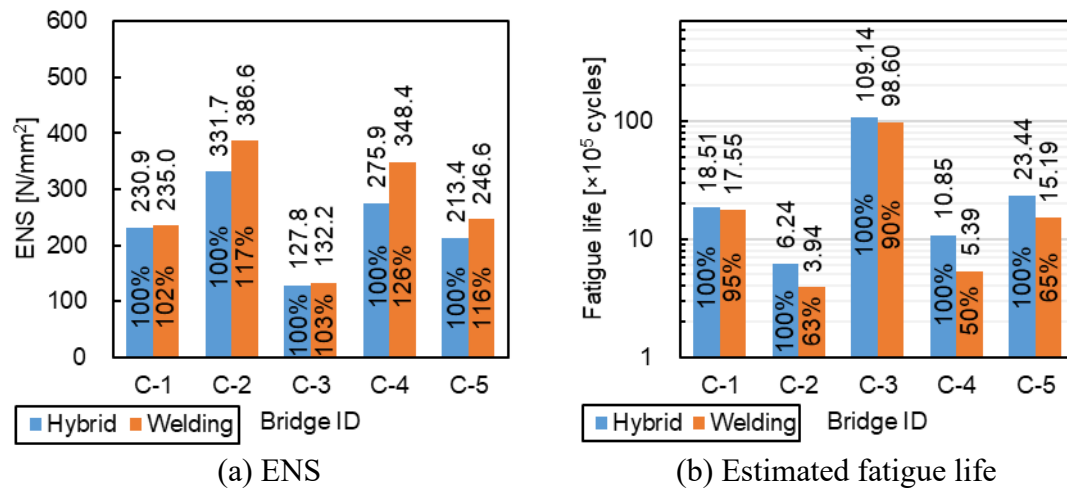


Figure 2-13 Comparison of hybrid and welding-only connection

Using C-2 as an example, Figure 2-14 shows the distribution of relative displacement between the bottom flange and the sole plate. In the hybrid connection model, displacement is noticeably lower around the bolts. If compared to the welding-only model, the hybrid case consistently shows less overall displacement, highlighting the positive role of bolts in managing ENS at the weld root. However, the impact of bolts is limited to a specific area. The difference in relative displacement between the two connection types decreases beyond a distance of 50 mm from a bolt. Therefore, simply adding more bolts or positioning bolts closer to the welds is not a feasible design strategy since enough safety clearance from the edge of the sole plate must be maintained.

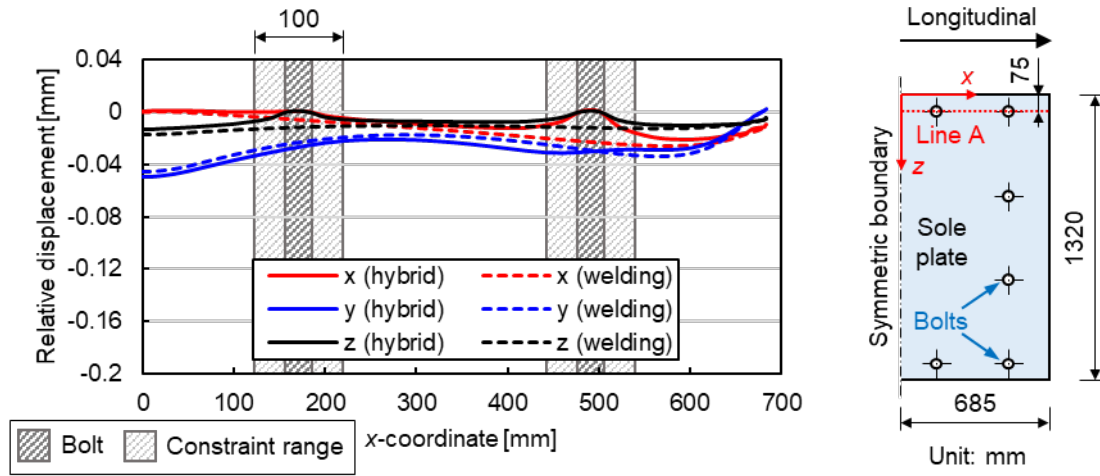
2.5 Summary

This chapter examined five existing bridges to identify structural characteristics associated with Type D fatigue cracking, supported by parametric numerical analyses. Based on the findings, several design recommendations were proposed to improve weld root fatigue performance.

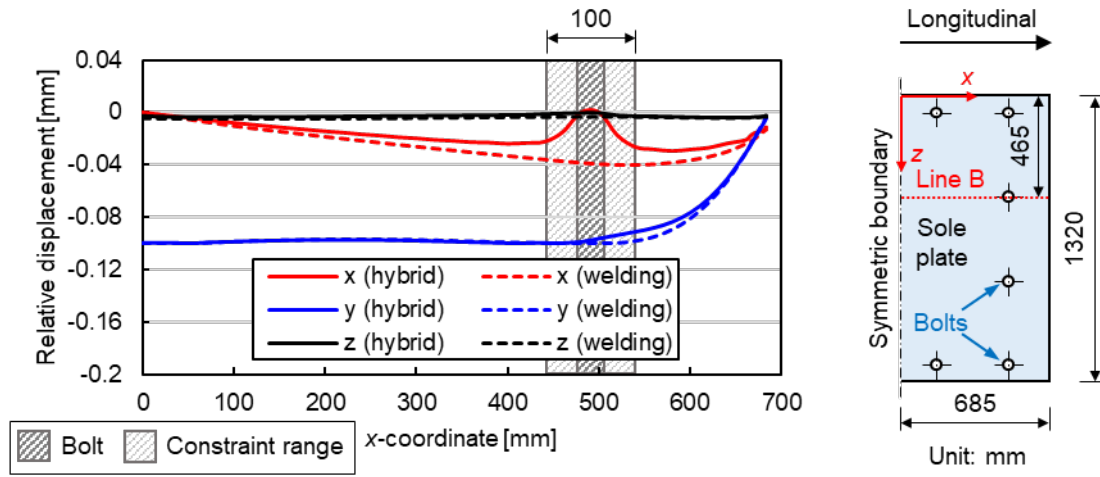
(1) The weld root connecting the bottom flange and the sole plate near intermediate supports was identified as a critical location. It experiences high tensile stresses that lead to fatigue problems. The problem is more severe in those bridges with fewer but larger girders and in those carrying a high proportion of live load. For certain cases, the estimated weld root fatigue life is less than 2×10^6 cycles.

(2) Among the parameters evaluated, enlarging the fillet weld size showed a notable improvement in fatigue resistance, while varying sole plate thickness had little effect. For bridges with relatively thick sole plates, hybrid connections combining bolts and welds are necessary. The bolts should be positioned as close to the weld root as safety clearances allow to maximize fatigue resistance.

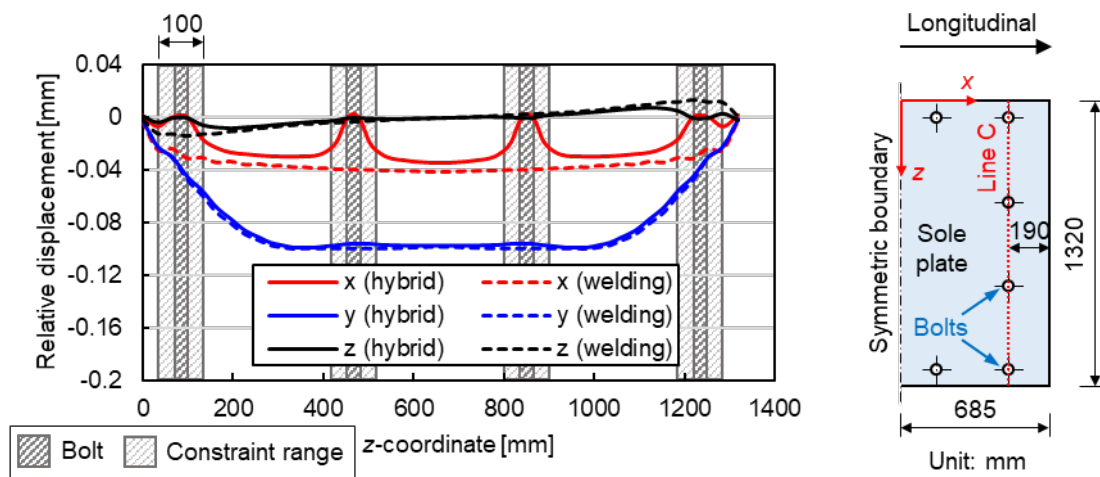
Overall, careful structural adjustment can lower the weld root ENS. However, it remains challenging to eliminate the risk of fatigue cracking entirely over the service life in some bridges. For example, bolt placement is often constrained by structural requirements, preventing its installation near the weld root. These limitations highlight the need for alternative approaches, motivating the concept of bonding-assisted welding.



(a) Line A



(b) Line B



(c) Line C

Figure 2-14 Relative displacement in Bridge C-2

CHAPTER 3

BONDING-ASSISTED WELDING BY ADHESIVE INSERTION

3.1 Introduction

In Chapter 2, it was found that bolt constraint has a significant effect on preventing Type D fatigue cracks. However, the constraint range of bolts is limited, and in some cases, it cannot cover the weld root. Therefore, a method of using adhesive to constrain the bottom flange and sole plate is proposed (adhesive insertion), using adhesive to supplement the position where bolts cannot constrain, as shown in Figure 3-1. This method is called bonding-assisted welding and was first proposed by Hirohata in 2016 to improve the static load performance of the joint. However, during the application process, it was found that there was a risk of adhesive burning. Although grooves were set to contain adhesive to maintain a safe distance, a small amount of adhesive still overflowed, causing combustion and generating gas to cause welding defects [85, 86], as shown in Figure 3-2.

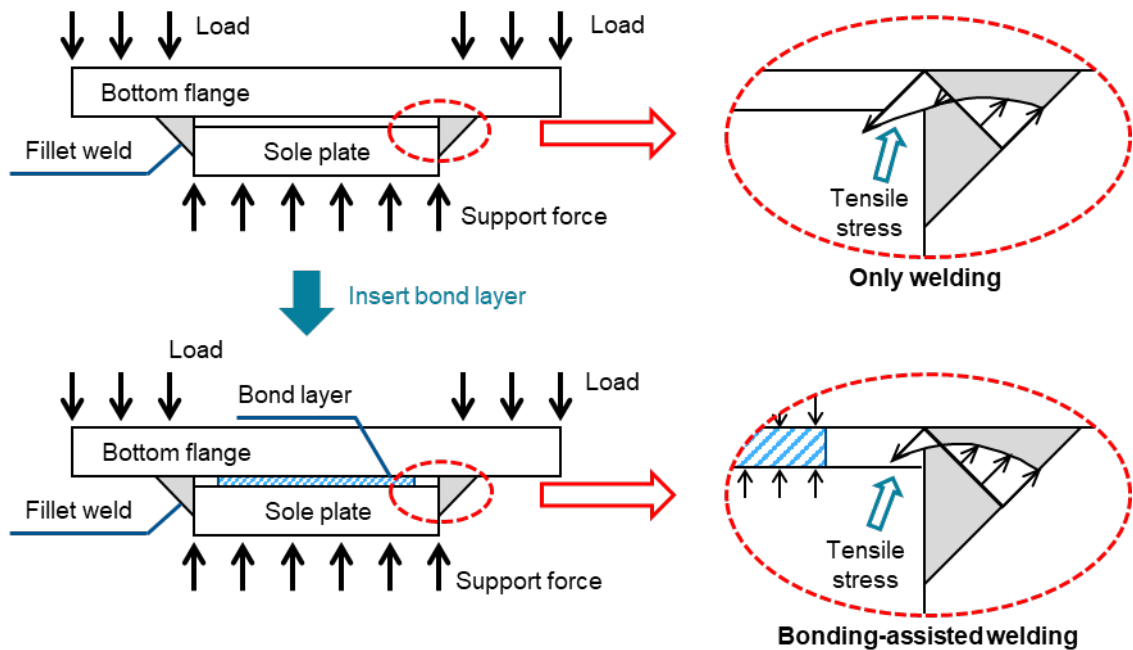


Figure 3-1 Concept of bonding-assisted welding

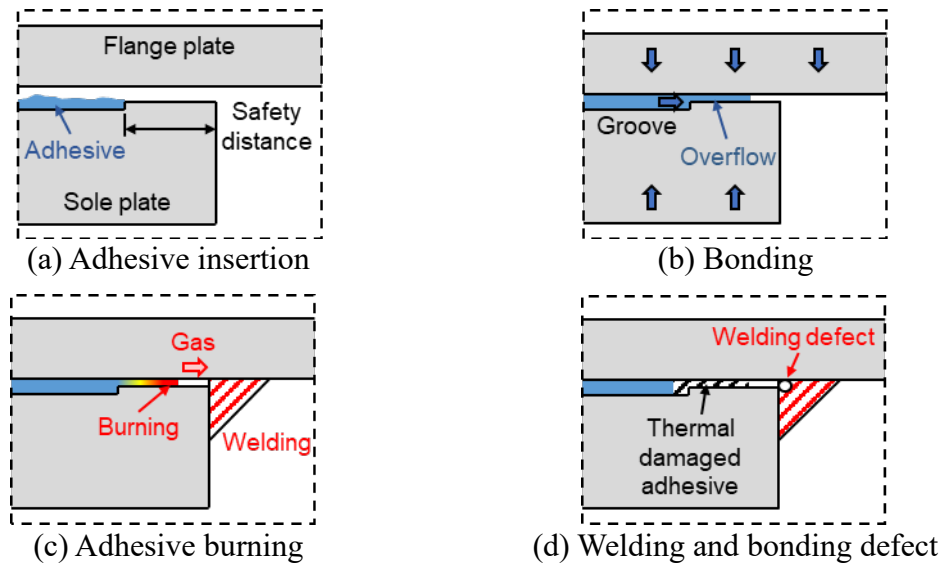


Figure 3-2 Mechanism of defect generation

Therefore, this chapter proposes an improved method in which heat-resistant rubber is placed around the adhesive, as shown in Figure 3-3. The rubber barrier prevents the adhesive from flowing out before solidification, maintains a safety distance, and prevents burning during welding. Based on this concept, a series of experimental and numerical studies was designed and conducted.

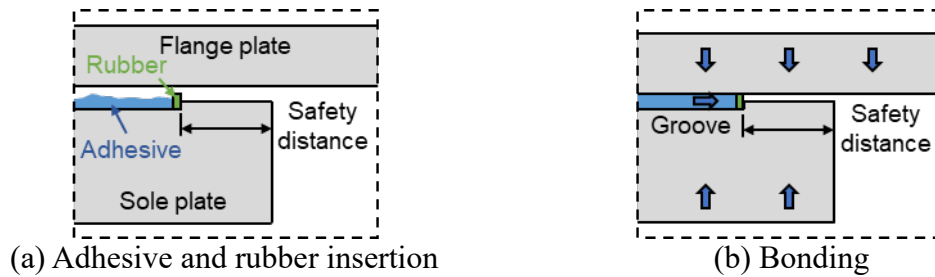


Figure 3-3 Effect of rubber

3.2 Experiments

3.2.1 Specimens

Bridge supports are particularly vulnerable to fatigue due to their high-stress environment. Therefore, the specimens were designed to replicate the stress conditions near bridge supports, focusing on the cyclic tensile stress acting on the fillet weld root between the bottom flange and the sole plate (Figure 3-4). Detailed dimensions are shown

in Figure 3-5. Each specimen measured 300 mm in length, 40 mm in width, and 31 mm in height, with flange and sole plate thicknesses of 9 mm and 22 mm, respectively. At the center of the sole plate, the thickness was locally reduced by 0.5 mm, and heat-resistant rubber was placed around this area to confine epoxy and prevent burning during welding [85]. The reduced region commenced 15 mm from the weld root, with the choice of this safety distance explained in Subsection 3.2.4. For comparative analysis, specimens without epoxy were also designed. For comparison, weld-only specimens without epoxy (labeled as W) were also fabricated. Specimens incorporating both welding and bonding were labeled as WB, with further classifications according to rubber thickness: WB0 (no rubber), WB0.5 (0.5 mm-thick rubber), and WB1 (1 mm-thick rubber).

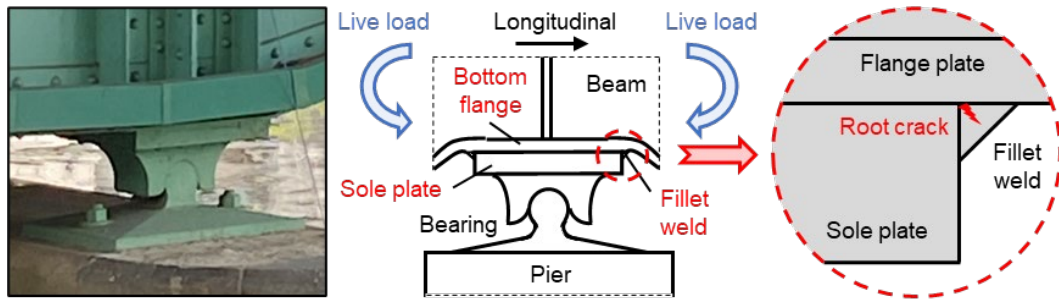


Figure 3-4 Bridge support

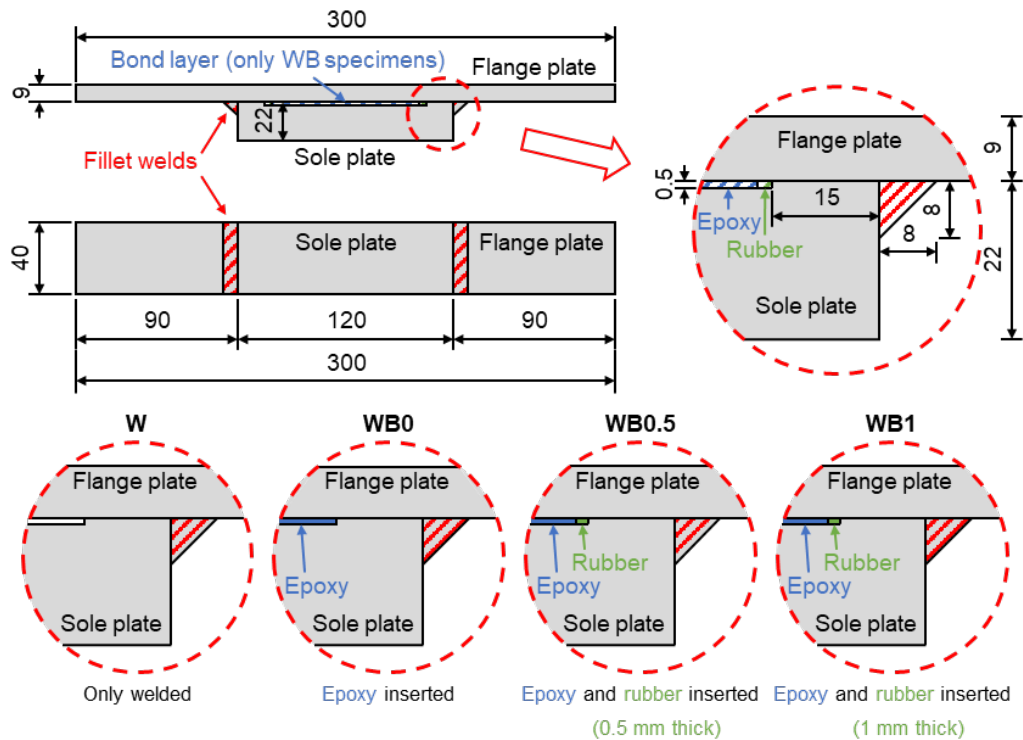


Figure 3-5 Details of specimens for adhesive insertion (unit: mm)

3.2.2 Fabrication

A total of 36 specimens were prepared, divided into four categories: W, WB0, WB0.5, and WB1, with nine specimens of each type. For the WB series, the sole plate was machined to create a 0.5 mm thickness reduction at its center. Both the flange and sole plates were then heat-treated ($600^{\circ}\text{C} \times 8\text{h}$) to relieve internal stress. After blasting the surface, epoxy was uniformly applied to the reduced area, followed by a 24-hour curing period at room temperature according to manufacturer's recommendation. The plates were subsequently joined by fillet welding using gas metal arc welding. For the W specimens, the sole plate also contained a thickness reduction but without epoxy. Welding parameters for all specimens were identical: 120 A current, 20 V voltage, and 5.2 mm/s travel speed. The specimens were initially fabricated from steel plates that were 200 mm wide and then cut into 40 mm wide specimens (Figure 3-6). Since the cutting process partially relieved the residual stress from fabrication, these stresses were considered negligible in the subsequent analysis.

3.2.3 Materials

The flange plate and sole plate were fabricated from SM400A steel [94]. They had thicknesses of 9 mm and 22 mm, respectively. Welding was performed with a YGW12 wire of 1.2 mm diameter, which belongs to the 490 N/mm^2 strength class as specified in JIS Z3312:2009 [95]. Table 3-1 shows the detailed chemical compositions and mechanical properties of these steel materials. The adhesive selected for the WB specimens was a heat-resistant epoxy resin, designated as E258R and manufactured by Konishi Chemical Ind. Co., Ltd. Table 3-2 summarizes the mechanical properties of the epoxy after heating, including its elastic modulus, tensile strength, and tensile lap-shear strength. Standard deviations for each measurement are also provided to indicate the variability of the test data. In addition, the heat-resistant rubber was FB780N manufactured by Akitsu Industry Co., Ltd, the material properties are summarized in Figure 3-3.

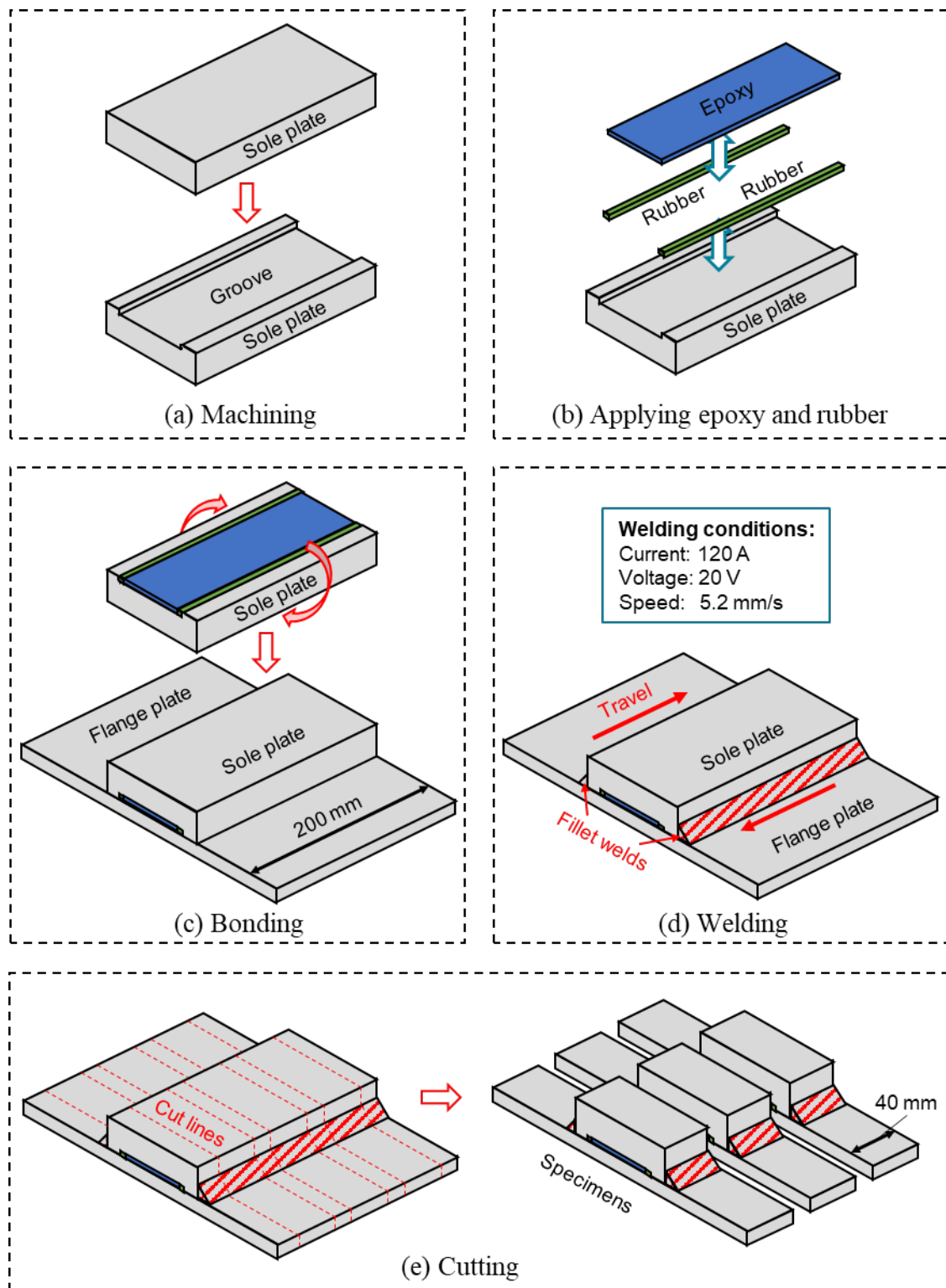


Figure 3-6 Fabrication of the specimens

Table 3-1 Chemical compositions and mechanical properties of steel materials

Category	Chemical compositions [mass %]					Elastic modulus [N/mm ²]	Yield strength [N/mm ²]	Tensile strength [N/mm ²]	Elongation [%]
	C	Si	Mn	P	S				
SM400A (9 mm)	0.08	0.25	0.82	0.018	0.003	2.06×10^5	341	450	31
SM400A (22 mm)	0.07	0.20	0.84	0.016	0.007	2.06×10^5	290	430	32
G49A0C12	0.09	0.44	0.94	0.012	0.012	2.06×10^5	460	540	-

Note: The data of G49A0C12 are nominal values.

Table 3-2 Mechanical properties of epoxy after heating [93]

Heating temperature [°C]	Elastic modulus [N/mm ²]	Tensile strength [N/mm ²]	Tensile lap-shear strength [N/mm ²]
Unheated	3 800 (25.6)	35.7 (2.3)	29.3 (0.6)
150	3 700 (123.6)	40.0 (7.1)	32.4 (1.2)
200	2 900 (69.9)	34.2 (6.1)	32.7 (0.4)
250	3 100 (119.5)	38.4 (4.7)	32.4 (0.3)

Note: The data in parentheses are standard deviations.

Table 3-3 Material properties of rubber

Category	Tensile strength [N/mm ²]	Elongation [%]	Shear modulus [N/mm ²]	Heat resistance limit temperature [°C]
FB780N	17.8 (-7%)	260 (±0%)	1.50	230

Note: The data in parentheses are the change after heating (250°C × 72h).

3.2.4 Safety Distance

The temperature rise within the specimen caused by welding heat input was evaluated using a simplified heat conduction approach to determine the minimum safety distance between the applied epoxy and the weld line. For this purpose, a single weld was

considered, with the arc welding process idealized as an instantaneous line heat source. Under these assumptions, the maximum temperature can be expressed by the following equation [96].

$$\frac{1}{T_{\max} - T_0} = \frac{1}{T_m - T_0} + \frac{c\rho r^2}{0.234Q} \quad (3-1)$$

Where, $T_{\max}$ is the maximum temperature; T_0 is the initial temperature; T_m is the melting point of steel; c is the specific heat; ρ is the material density; r is the distance from the weld root; and Q is the welding heat input, which is determined by the following equation.

$$Q = \eta \frac{IU}{v} \quad (3-2)$$

Where, η is the thermal efficiency, which is usually taken as 0.7 for gas metal arc welding; I is the arc current; U is the arc voltage; v is the welding speed.

To more accurately represent heat partitioning between the steel plates, previous research [97] provides an estimation method where the heat inputs to the flange plate Q_f and sole plate Q_s are given by the following equations, respectively.

$$Q_f = Q \frac{2t_f}{2t_f + l_s} \quad (3-3)$$

$$Q_s = Q \frac{l_s}{2t_f + l_s} \quad (3-4)$$

Where, t_f is the thickness of the flange plate; and l_s is the length of the sole plate.

Using this procedure, the estimated temperature distribution during welding was derived and is presented in Figure 3-7. According to Table 3-2, the epoxy exhibits stable tensile strength and lap-shear strength even after heating up to 250 °C, with only a reduction in elastic modulus observed. Therefore, 250 °C was adopted as the critical temperature threshold for defining the safe distance. Based on the thermal analysis, this threshold corresponds to a minimum clearance of approximately 15 mm between the epoxy and the weld. Thus, all specimens were fabricated following this requirement.

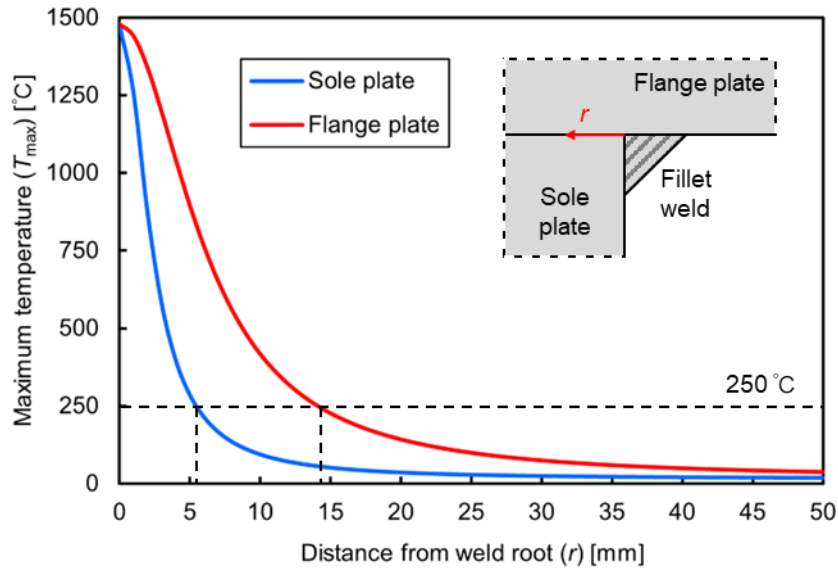


Figure 3-7 Maximum temperature distribution during welding (adhesive)

3.2.5 Setup and Measurements

To replicate the stress condition acting on the sole plate of bridge bearings in service, a four-point bending experiment was conducted on the fabricated specimens. The overall experimental arrangement is illustrated in Figure 3-8. In this setup, the load was applied cyclically at a frequency of 5 Hz, producing tensile stress at the weld root. The load ratio (minimum load / maximum load) was set to 0.1. For strain monitoring, two strain gauges were installed on each side of the flange plate. This led to four measurement points per specimen near the weld roots. Furthermore, to confirm the stress distribution from the weld root measurements, partial specimens were instrumented with two additional strain gauges. These were located near the weld toe at the middle of the flange plate. These extra measurements helped verify the reliability of the experimental results.

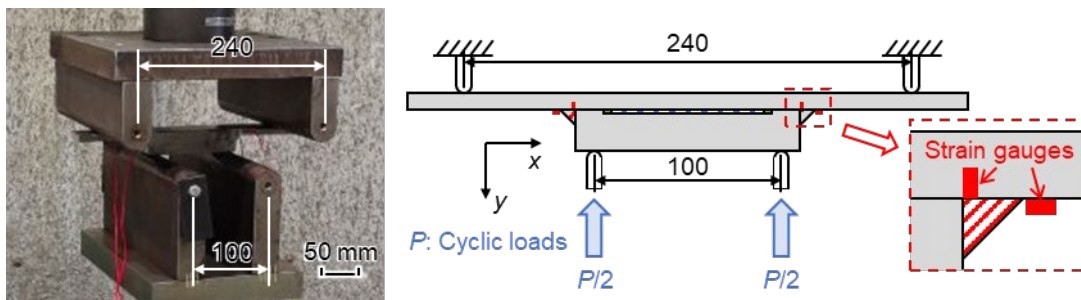


Figure 3-8 Setup and measurements (unit: mm)

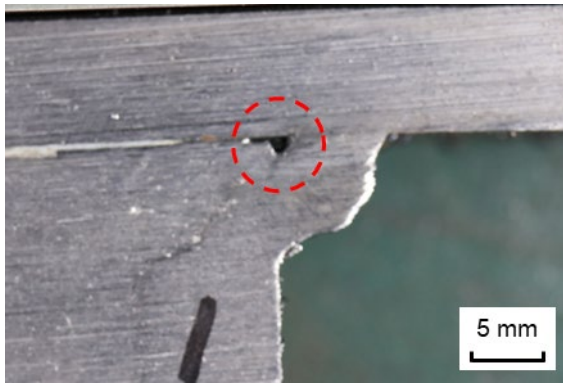
3.3 Experimental Results

3.3.1 Defects of the Specimens

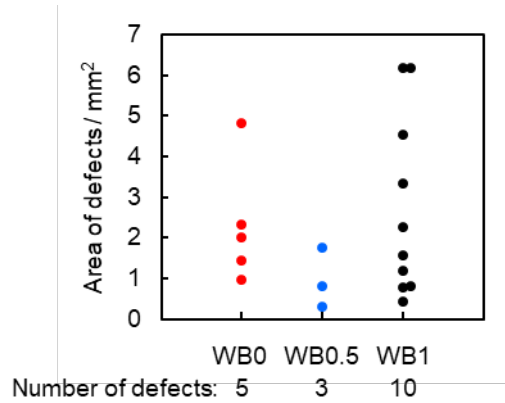
Welding defects were found during the inspection of specimens, with a representative case shown in Figure 3-9 (a). These defects were attributed to the gases produced by epoxy burning. The overall number and area of defects for each specimen type are summarized in Figure 3-9 (b). Although rubber was used as a barrier to prevent epoxy leakage, it did not stop all outflow. As a result, defects related to burning epoxy were still found in some WB specimens. The WB0.5 specimens showed fewer defects, both in number and area, than the WB0 specimens. This indicates that a moderately thick rubber layer effectively reduced epoxy outflow and burning. In contrast, the WB1 specimens had more defects in both incidence and size compared to the WB0 specimens. This outcome suggests that while the insertion of rubber is beneficial when applied with an appropriate thickness, excessive thickness may enlarge the gap between the flange and sole plates, leading to increased epoxy leakage and worse weld defects.

Since ISO 5817 [98] does not specify evaluation criteria for this type of welding defect, an approximate assessment was made by referring to the requirements for quality level B. This quality level prohibits incomplete penetration at the weld root. Based on this criterion, all specimens exhibiting defects visible to the naked eye were eliminated from the fatigue experiment program. Furthermore, because the WB1 specimens displayed a significantly higher incidence of epoxy-burning defects, this group was entirely excluded from the fatigue experiment. As a result, the number of valid specimens was reduced to 21, all of which satisfied the weld quality requirements of ISO 5817 [98].

In addition to the epoxy-burning defects, another type of imperfection was identified. Specifically, one WB0 specimen was found to contain an incomplete epoxy filling defect, as shown in Figure 3-10. The potential influence of this defect type will be discussed in detail later.



(a) Incomplete root penetration



(b) Number and area of welding defects

Figure 3-9 Welding defects attributed to epoxy burning

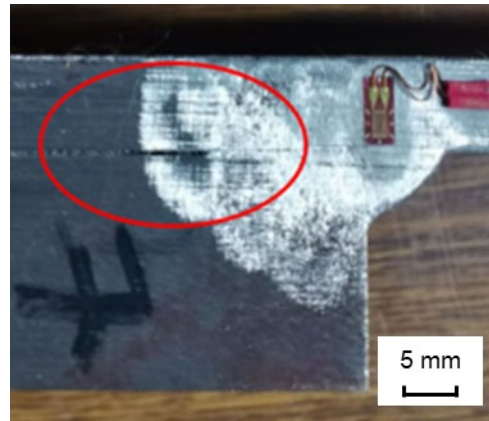


Figure 3-10 Incomplete epoxy filling

3.3.2 $S-N$ Relationship

A representative example of a fatigue crack observed is shown in Figure 3-11. All fatigue cracks were classified as brittle in nature and consistently initiated from the weld root. Because there is no simple method for calculating fatigue stress at the weld root, an alternative position, top of the flange plate, was used for calculating nominal stress amplitude. The $S-N$ relationship is presented in Figure 3-12. Based on previous research, fatigue failure was defined as the occurrence of a fatigue crack exceeding 0.2 mm in length from the weld roots [93]. Data points indicated by arrows correspond to specimens in which either no fatigue cracks were detected on the surface or fatigue failure did not occur even after being subjected to more than two million loading cycles.

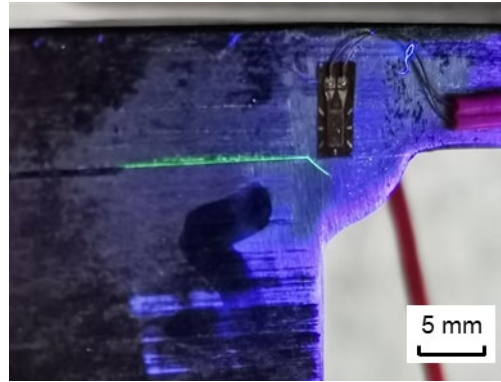
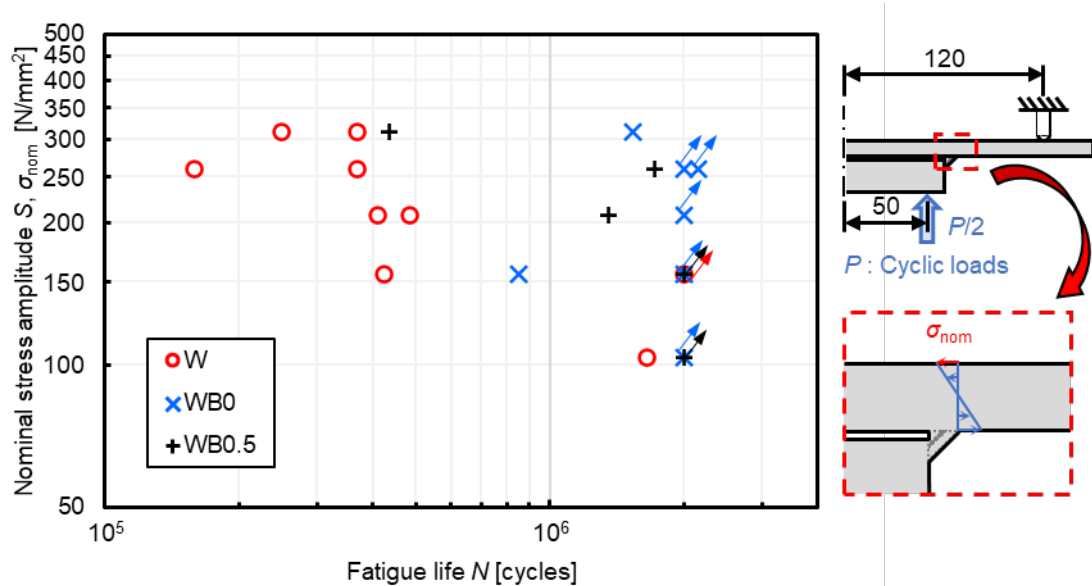


Figure 3-11 Detected fatigue crack

Figure 3-12 S – N relationship (adhesive insertion)

The experimental results demonstrate a clear improvement in fatigue life for specimens with epoxy insertion (WB series) compared to those without epoxy (W series). At nominal stress amplitudes above 200 N/mm², the fatigue lives of WB specimens were extended to more than three times that of W specimens. Notably, WB specimens subjected to stress amplitudes between 210 and 260 N/mm² exhibited fatigue lives comparable to W specimens subjected to only about 100 N/mm². This result implies that the insertion of epoxy reduces the fatigue stress at the weld root by approximately 50% to 60%. The fatigue lives of the WB0.5 group were slightly shorter than that of WB0 group. This result

will be analyzed in detail later. Nevertheless, it can be concluded that inserting a 0.5 mm thick rubber does not contribute significantly to extending fatigue life. Instead, its principal function is to improve welding quality by reducing epoxy leakage and lowering the chances of welding defects. It is also important to note that the results discussed here were obtained under the condition that the epoxy material was not aged. Since epoxy properties can decline over time due to aging, more studies are needed to assess the long-term reliability and durability of epoxy-assisted welds in real-world applications.

An exceptional case was observed in one WB0 specimen at a stress amplitude of 155 N/mm^2 , which exhibited a considerably shorter fatigue life than other WB specimens. This abnormal result is believed to stem from incomplete epoxy filling caused by shrinkage of the adhesive material, as previously shown in Figure 3-10. To confirm this hypothesis, numerical simulation analyses were conducted and are discussed in a later section.

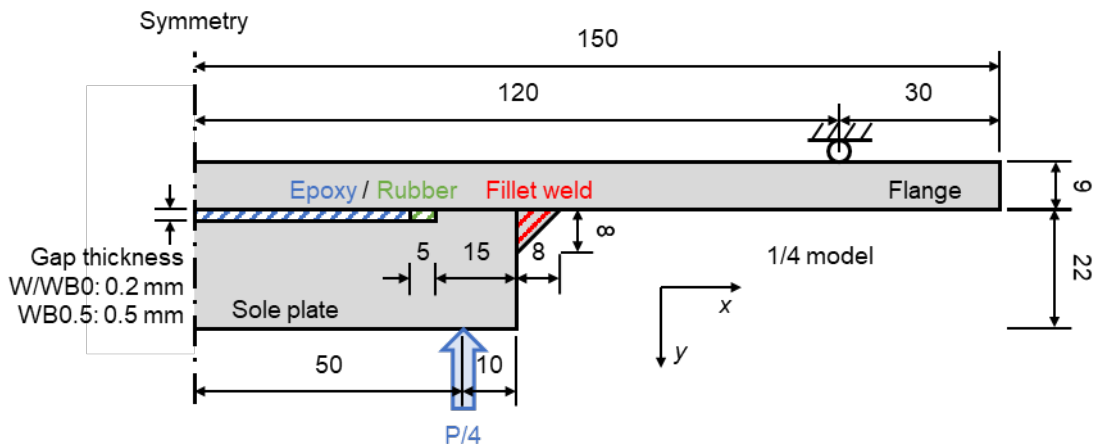
3.4 Numerical Analysis

3.4.1 Finite Element Models

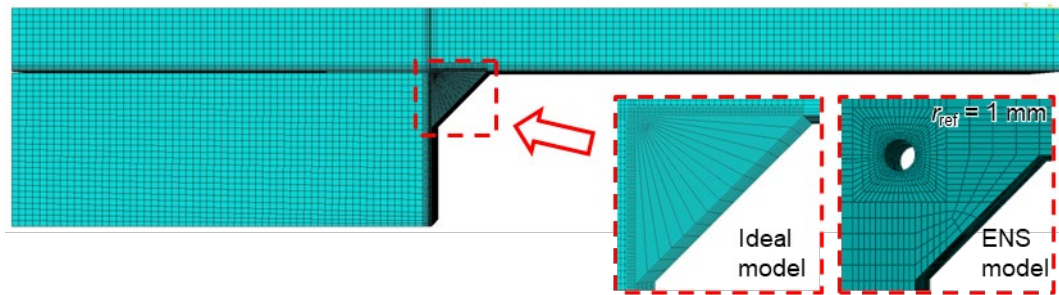
Finite element analysis was carried out using ABAQUS CAE to better understand the stress distribution around the weld roots. A quarter-symmetric model was adopted to reduce computational costs while maintaining accuracy, as shown in Figure 3-13. The gap thickness between the steel plates was set at either 0.2 mm or 0.5 mm, reflecting the values measured directly from the specimens. The numerical model was constructed using 3D 8-node solid elements with reduced integration (C3D8R). While the maximum element size was limited to 1 mm in the general region, the element size was gradually reduced to as small as 0.1 mm near the weld root. This meshing strategy resulted in a total of 131,660 elements and 148,491 nodes. Furthermore, to evaluate the improvement in weld root fatigue strength achieved by bonding-assisted welding, a finite element model was constructed using the ENS method. The difference between the original model and the ENS model is shown in Figure 2-4. The reference radius r_{ref} of the notch was 1 mm.

Material properties of the three types of structural steel were defined according to the values listed in Table 3-1, while the epoxy properties were set based on the unheated condition reported in Table 3-2. Poisson's ratios of 0.3 for steel and 0.35 for epoxy were

adopted. The weld and the steel plate were connected using tie constraints to simulate full fusion between them. For the contact definition, the normal interaction between the flange and sole plate was modeled as hard contact, whereas the tangential interaction was represented by a penalty function method with a friction coefficient of 0.3. In the case of WB specimens, the epoxy layer was modeled using solid elements of the same type as the steel plates. Tie constraints were imposed between the epoxy and the adjoining steel surfaces to simulate the bonding effect. Loading was applied as a monotonic concentrated force distributed over the mesh nodes, and its magnitude was set equivalent to the experimental load amplitude. Residual stress from welding was not considered.



(a) Details of the finite element model (unit: mm)



(b) Mesh of the finite element model

Figure 3-13 Fatigue analysis model

3.4.2 Load–Strain Relationship

To confirm the reliability of the finite element model, numerical strain results were compared against experimental measurements obtained from strain gauges, as illustrated in Figure 3-14. The comparison showed that the predicted strain responses matched

closely with the experimental data. This demonstrated that the modeling approach used was accurate enough for evaluating local stress conditions.

Under similar loading conditions, the WB specimens showed about a 60% reduction in strain at the weld root compared to the W specimens. This reduction highlights the effectiveness of the epoxy insertion in alleviating stress concentration at the weld root. On the other hand, the strain responses near the weld toe were similar for both types of specimens. This suggests that the positive effect of epoxy mainly occurs in the weld root area.

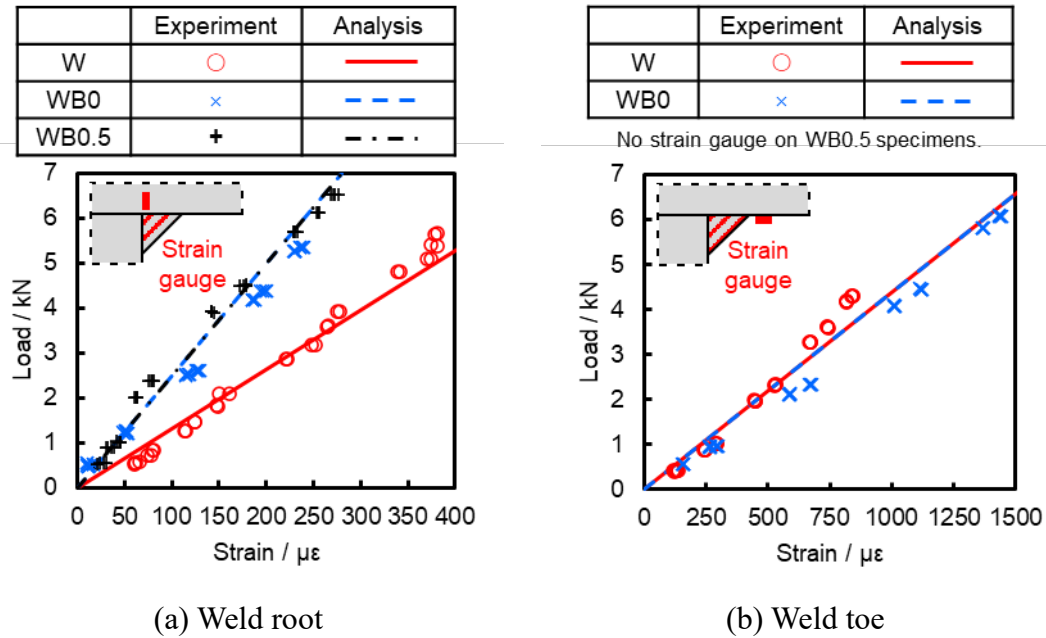


Figure 3-14 Comparison of load–strain relationship

3.4.3 Stress Mitigation

To evaluate the effect of epoxy insertion on local stress conditions, five load magnitudes corresponding to the fatigue experiments were applied in the finite element models. Although the absolute stress values varied with load magnitude, the overall patterns remained consistent across the different load magnitudes. Representative results under a nominal stress amplitude of about 210 N/mm^2 are shown in Figure 3-15. Moreover, the weld root ENS are summarized in Figure 3-16.

In all three models (W, WB0, and WB0.5), clear stress concentrations appeared at the weld root, with the maximum tensile stress being notable. Among these, the W model had the highest weld root ENS, highlighting the problem of Type D crack when epoxy was not used. In contrast, both the WB0 and WB0.5 models showed significant reductions in ENS, with values around 52% lower than those in the W model. These findings match the experimental fatigue performance, where WB specimens had longer fatigue lives compared to W specimens. A closer examination of stress components revealed that the effect of epoxy insertion was more significant in reducing the normal tensile stress in the y -direction (σ_y). Specifically, the reductions in σ_x and σ_y were about 55% and 70% respectively.

The WB0 and WB0.5 models displayed similar stress distribution. However, the weld root ENS in the WB0.5 model was approximately 18% higher than that of the WB0 model. This observation aligns well with the experimental results shown in Figure 3-12, where WB0.5 specimens exhibited slightly shorter fatigue lives compared with WB0 specimens. This could be explained by the role of the inserted rubber, which created a larger gap between the steel plates in the WB0.5 specimens, resulting in a thicker epoxy layer compared to the WB0 specimens. Since the elastic modulus of epoxy is much lower than that of steel, a thicker epoxy layer results in a weaker constraining effect on the surrounding plates. This reduced constraint diminishes the ability of epoxy to alleviate stress concentration, thereby resulting in somewhat higher stress in WB0.5 compared with WB0 specimens. In the finite element analysis, gap thicknesses of 0.2 mm and 0.5 mm were accordingly assigned to the WB0 and WB0.5 models, respectively, based on measured specimen data, as indicated in Figure 3-13 (a).

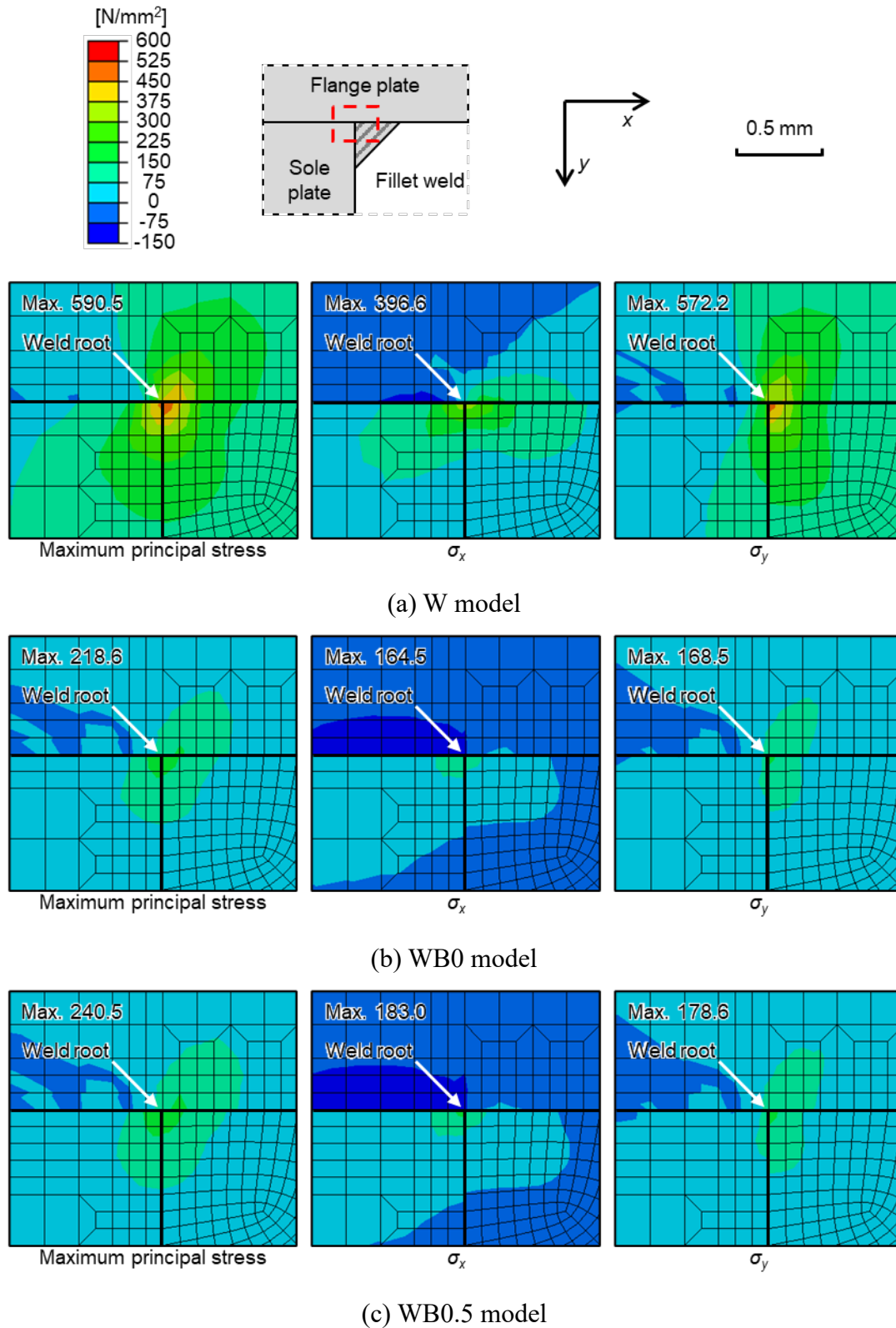


Figure 3-15 Stress distribution around the weld root (unit: N/mm²)

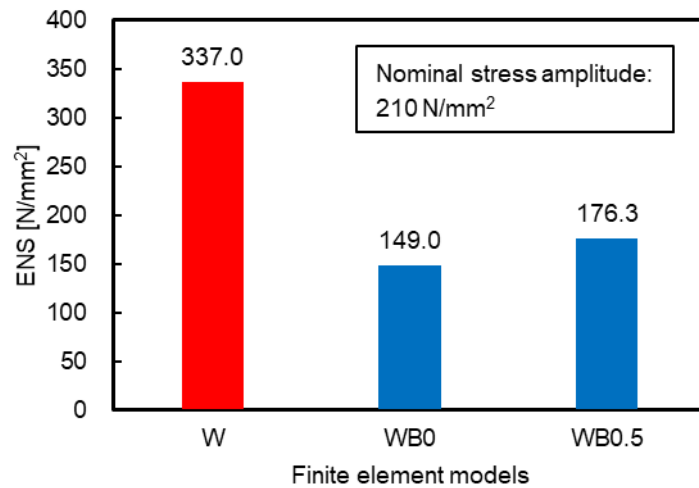


Figure 3-16 Comparison of weld root ENS (adhesive insertion)

3.4.4 ENS–N Relationship

Figure 3-17 presents the relationship between ENS and fatigue life for all specimens, referred to as the ENS–N relationship. Compared with Figure 3-12, the data points in Figure 3-17 exhibit a clearer concentration. A trendline was fitted to the dataset of effective specimens, excluding those without fatigue cracks and one with incomplete epoxy filling. The correlation coefficient between the experimental results and the fitted trendline was calculated as 0.65, much higher than the 0.10 correlation coefficient obtained with nominal stress fitting. These results indicate that the ENS method can effectively evaluate the fatigue strength of the weld root. Moreover, it can reliably assess the improvement in weld root fatigue performance achieved by adhesive insertion.

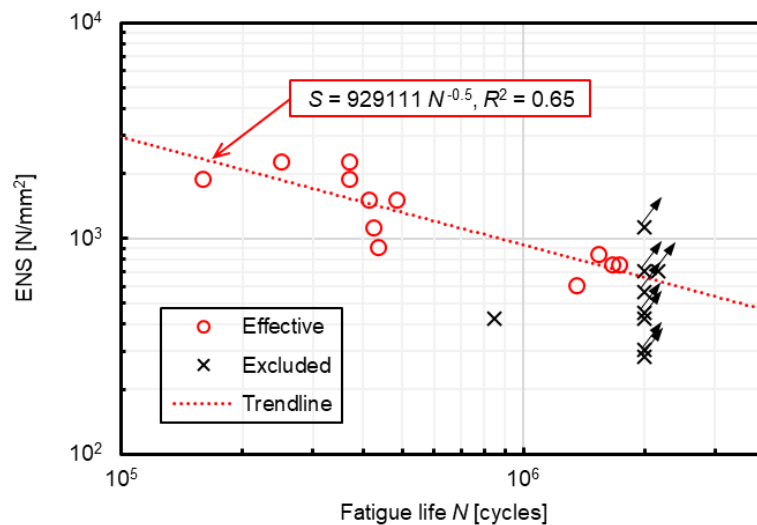


Figure 3-17 ENS–N Relationship

3.4.5 Incomplete Epoxy Filling

As discussed in Section 3.3, one of the WB0 specimens exhibited incomplete epoxy filling. This defect was associated with a significantly shorter fatigue life compared to the other WB specimens. To further examine this phenomenon, finite element models were developed to replicate the presence of unfilled areas within the epoxy layer. The analytical models were configured such that the gap between the two steel plates was not completely filled, as illustrated in Figure 3-18. In these simulations, it was assumed that the unfilled area extended uniformly across the specimen width, with the defect length varied systematically from 1 mm to 30 mm. Apart from this modification, all other model parameters were kept consistent with those described previously in Figure 3-13.

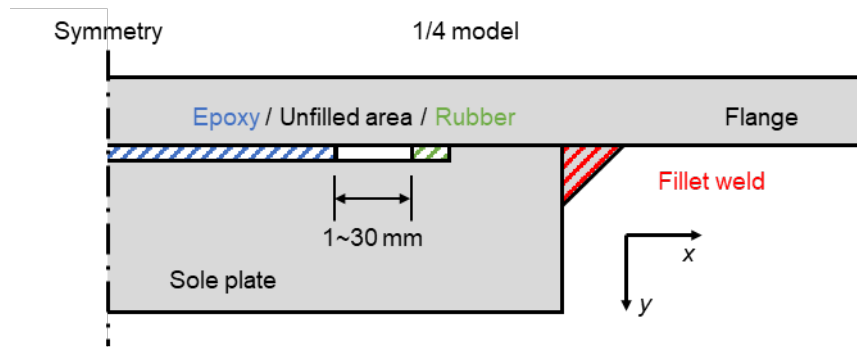


Figure 3-18 Analysis model with incomplete epoxy filling

Under a nominal stress amplitude of about 210 N/mm^2 , the ENS at the weld root was evaluated for different defect lengths, and the results are summarized in Figure 3-19. The analysis revealed that the ENS increased progressively with defect length for both WB0 and WB0.5 models. Interestingly, the sensitivity of stress growth to defect length was higher in the case of small defects; when the defect length was relatively short, the weld root stresses increased more steeply compared with the growth observed for larger defect lengths. In addition, a distinction between WB0 and WB0.5 models was observed when the defect size was small. For shorter unfilled regions, the ENS at the weld root in the WB0.5 model was approximately 16% higher than that in the WB0 model. However, as the defect length increased to 30 mm, the ENS results of the two models became closer, yielding only about 5% difference. This convergence suggests that the difference in weld

root stress between WB0 and WB0.5 models is governed primarily by the thickness of the epoxy layer rather than by variations in the gap between the flange and sole plate.

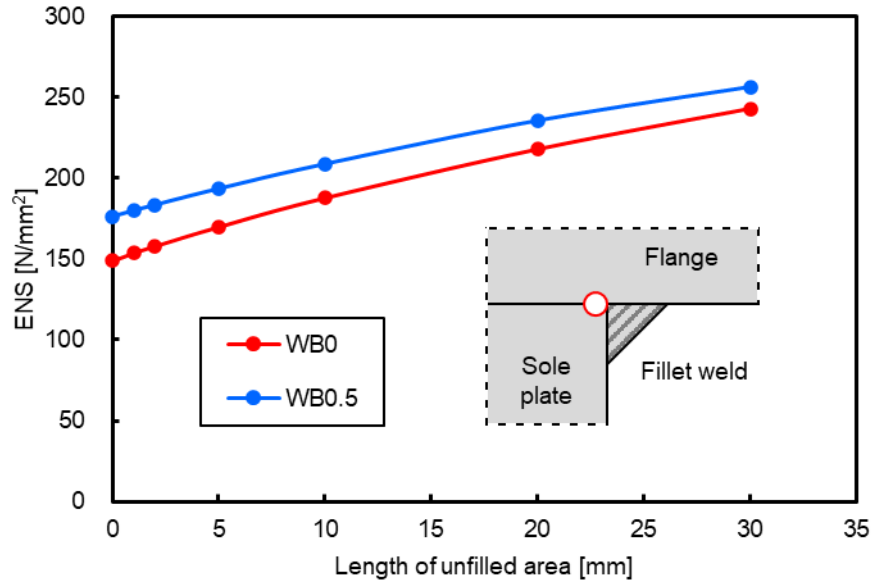


Figure 3-19 Variation of weld root ENS with incomplete filling distance

3.5 Summary

In this chapter, the influences of adhesive insertion were investigated through a combination of experiment and numerical simulation. The experimental program primarily examined welding quality and fatigue life. The simulation analyses provided detailed insights into stress distributions around the weld root and the effects of incomplete epoxy filling. The main findings can be summarized as follows.

(1) Welding heat input caused epoxy burning and subsequent gas generation, which led to welding defects. The experiments demonstrated that placing a 0.5 mm thick layer of heat-resistant rubber around the epoxy effectively minimized the occurrence of such defects. Specimens with 0.5 mm rubber (WB0.5) contained fewer and smaller welding defects than those without rubber (WB0) or with thicker rubber layers (WB1). Although the fatigue lives of WB0.5 specimens did not surpass those of WB0 specimens, the improvement in welding quality provides a promising basis for the potential engineering application of epoxy bonding-assisted welding.

(2) Both experimental observations and analytical simulations confirmed that epoxy insertion substantially reduced the fatigue stress at the weld root. Under four-point bending conditions, this reduction was approximately 60%. The epoxy was also effective in lowering normal stress, with reductions of about 55% for σ_x and 70% for σ_y . This highlights the effect of epoxy in mitigating stress concentrations, particularly in the y -direction.

(3) Because epoxy has a much lower elastic modulus than steel, a thicker epoxy layer creates a weaker constraining effect on the flange and sole plate. As a result, specimens with thicker epoxy layers showed less effective stress reduction at the weld root than those with thinner layers. Additionally, incomplete epoxy filling led to extra stress concentration at the weld root, increasing the severity of stress as the unfilled area lengthened. These findings indicate the importance of maintaining complete and uniform epoxy filling during fabrication to ensure optimal performance.

(4) The ENS method was shown to be more suitable than the nominal stress method for assessing the fatigue performance of fillet weld roots under bending and shear loads, as well as for assessing the improvements achieved by epoxy insertion. The ENS–N relationship exhibited a clearer concentration of experimental data compared with the S–N relationship based on nominal stress. For effective specimens, the correlation factor of the fitted ENS–N relationship reached 0.65, demonstrating the method’s effectiveness in fatigue strength evaluation.

CHAPTER 4

BONDING-ASSISTED WELDING BY SOFT METAL INSERTION

4.1 Introduction

In Chapter 3, the concept of adhesive insertion was examined as a method of bonding-assisted welding. Although the introduction of heat-resistant rubber reduced adhesive outflow to some extent, it did not eliminate the issue entirely. As a result, the adhesive was still susceptible to burning during welding, which not only weakened its bonding capability but also generated gases that could cause welding defects such as incomplete root penetration. These limitations highlighted the need to explore alternative bonding materials capable of withstanding welding heat input without degrading. In this chapter, the adhesive was replaced with a soft metal to implement bonding-assisted welding (soft metal insertion). Soft metals are characterized by their low hardness, high ductility, and excellent malleability, which enable them to deform easily and conform to irregular surfaces. Among various candidate materials, zinc was selected for investigation. Zinc offers several advantages. It is cost-effective compared to other soft metals like tin and copper. It has strong corrosion resistance and shows better high-temperature resistance and durability than epoxy. Additionally, zinc has a relatively low melting point of about 420 °C. This low melting point reduces thermal effects on the steel plates during heating. These characteristics make zinc a good alternative that can improve both welding quality and the long-term reliability of the joint. This chapter aims to evaluate the feasibility of bonding-assisted welding using zinc insertion and to examine its impact on the fatigue performance of fillet weld roots. To do this, a series of experiments and simulations were conducted.

4.2 Experiments

4.2.1 Specimens

The specimens were designed to replicate the structural configuration of actual bridge supports (Figure 3-1), with their dimensions proportionally scaled. Each specimen

had a flange plate measuring 300 mm in length and 9 mm in thickness, and a sole plate measuring 120 mm in length and 22 mm in thickness. These two plates were joined by fillet welding, and the overall specimen width was set to 40 mm. The fillet weld had a leg length of 4 mm, as shown in Figure 4-1.

Two main categories of specimens were prepared: as-welded specimens (W) and zinc-inserted specimens (Z). In the W category, the flange and sole plates were connected only by fillet welding, without any material insertion. In contrast, the Z category incorporated a zinc sheet with 0.5 mm thickness between the steel plates to establish additional bonding. To account for the influence of initial gaps between the flange plate and sole plate, the W specimens were subdivided into two types: W0, which had no gap, and W0.5, which had a 0.5 mm gap. For the Z specimens, the categorization was based on the distance from the zinc sheet to the fillet weld root. Specifically, three types were fabricated: Z0, with no distance from the zinc sheet to the weld root; Z5, with the sheet placed 5 mm away from the root; and Z10, with a distance of 10 mm from the root. The design basis for the three distances is detailed in Subsection 4.2.4.

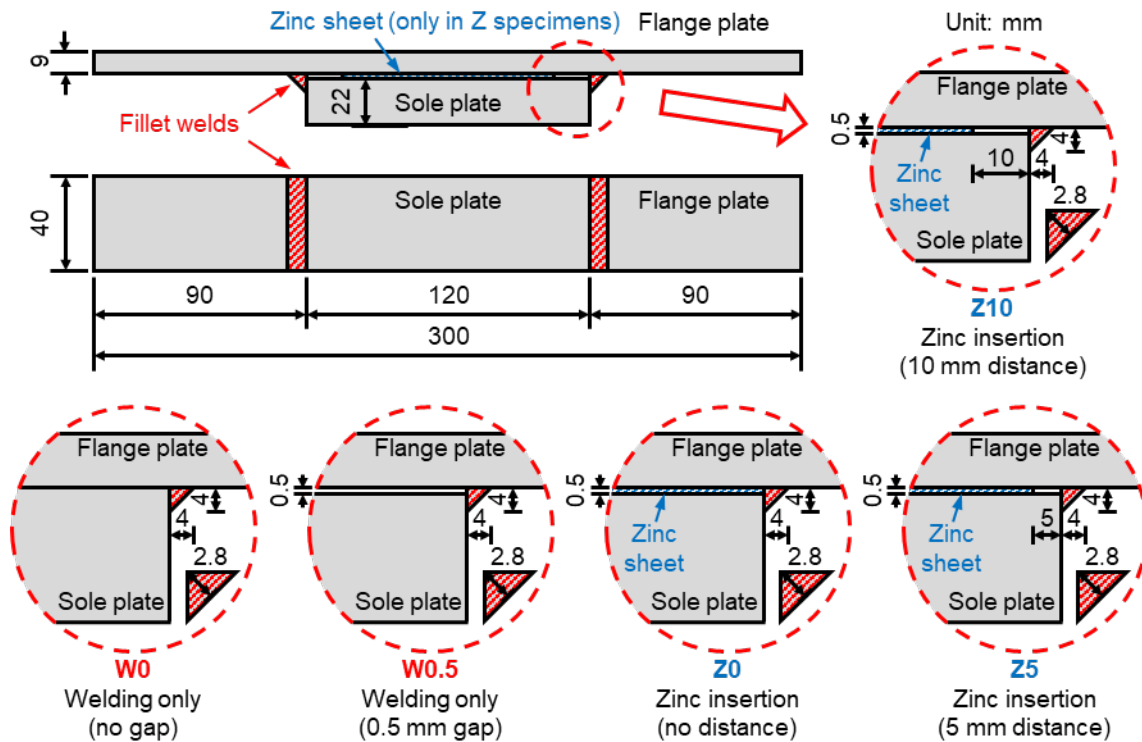


Figure 4-1 Details of specimens for soft metal insertion (unit: mm)

4.2.2 Fabrication

The fabrication process of the specimens is illustrated in Figure 4-2. All specimens were initially prepared from steel plates with a width of 200 mm, and prior to welding, the surfaces were treated by blasting to ensure proper cleanliness and surface roughness.

For the W0 specimens, the flange and sole plates were welded together after preparing the surfaces. For the W0.5 specimens, a steel sheet with 0.5 mm thickness was temporarily placed between the two plates before fillet welding. After the welding process, the steel sheet was removed, creating a 0.5 mm thick gap between the flange and sole plates.

For the Z specimens, a zinc sheet with 0.5 mm thickness was inserted between the two plates prior to welding. Unlike the W0.5 specimens, the zinc sheet was retained after welding, as shown in Figure 4-2 (b). Welding was carried out under controlled conditions with a current of 130 A, a voltage of 18 V, and a welding speed of 5.0 mm/s. However, during the fabrication of the Z0 specimen, the zinc sheet was too close to the weld root, causing the zinc to vaporize. The resulting vaporization adversely affected welding quality and posed safety concerns. Due to this issue, which will be discussed in detail in Subsection 4.3.1. Because of this, all Z0 specimens were excluded from the fatigue tests.

After welding, the specimens were heat-treated to enhance the zinc-steel bonding effect. The heat treatment was performed by a high-temperature furnace. The heating process started with heating gradually to 500 °C over 8 hours. Next, the temperature was maintained at this temperature for another 8 hours. The specimens were then cooled to about 20 °C slowly (Figure 4-2 (c)). The purpose of this treatment was to melt the zinc sheet, which has a melting point of approximately 420 °C, so that it could flow into and fill the interfacial gaps between the steel plates, thereby strengthening the metallurgical bond. According to previous research, creep in steel becomes significant only above 550 °C. Metallurgical transformations occur at higher temperatures [55, 99, 100]. Hence, this heat treatment process can be considered to have had no adverse impact on the steel.

Then, all specimens were cut into 40-mm wide pieces to prepare them for fatigue experiments (Figure 4-2 (d)). It is worth noting that cutting operations partially release residual stress. A previous study has reported that longitudinal residual stresses across the specimen length, as well as transverse stresses on the cut surface, are almost completely

relieved during cutting, leaving only a small level of residual transverse stress within the interior of the material [101]. Since the surface where fatigue cracks were to be observed exhibited negligible residual stress and the internal stresses were uniformly low across all specimens, the residual stress effects were considered insignificant and thus were not included in the subsequent fatigue analysis.

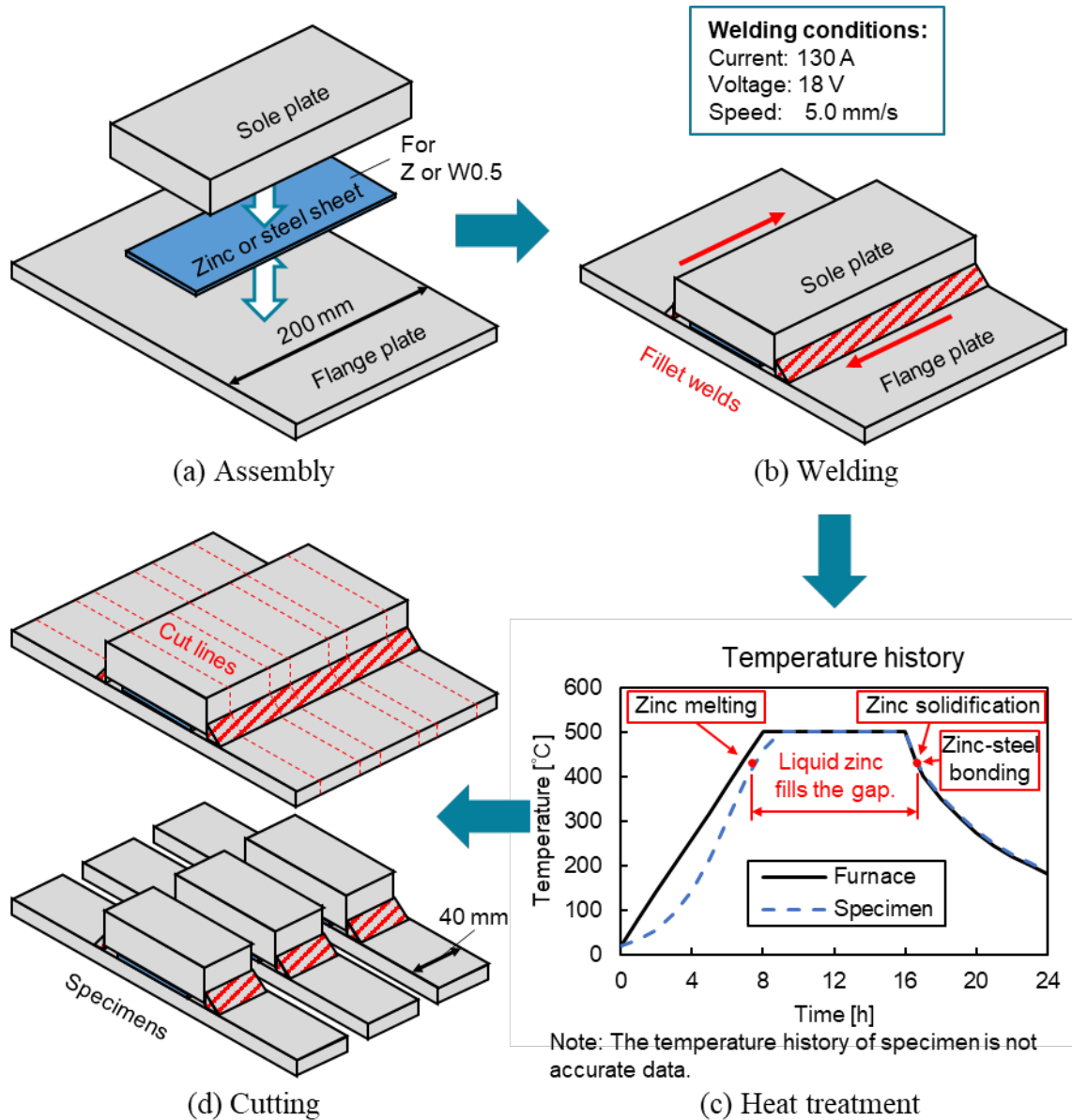


Figure 4-2 Fabrication process of the specimens

4.2.3 Materials

To examine the fatigue performance of weld roots in the vicinity of bridge supports, the structural steel SM400A [94], which is widely used in Japan, was chosen for the fabrication of the specimens. Two different plate thicknesses were employed. The flange plates were 9 mm in thickness, and the sole plates were 12 mm in thickness to simulate practical bridge support conditions. The welding consumable used was YGW12, as specified in JIS Z 3312, which is categorized as a 490 N/mm² class welding material [95]. The detailed chemical compositions of the steels used are presented in Table 4-1, and their corresponding mechanical properties are summarized in Table 4-2. In addition, zinc was selected as the bonding material for soft metal insertion. The choice of zinc was based on several advantages: (1) it is inexpensive compared to other soft metals, (2) it exhibits good corrosion resistance, and (3) its relatively low melting point of approximately 420 °C ensures that the heating process required for bonding has minimal thermal influence on the surrounding steel plates. The zinc sheets used in the experiments were fabricated from pure zinc, and their mechanical properties are also listed in Table 4-2.

Table 4-1 Chemical compositions of steels

Category	Chemical compositions (mass %)				
	C	Si	Mn	P	S
SM400A (9 mm)	0.08	0.25	0.82	0.018	0.003
SM400A (22 mm)	0.07	0.20	0.84	0.016	0.007
YGW12	0.09	0.44	0.94	0.012	0.012

Note: The data of YGW12 are nominal values.

Table 4-2 Mechanical properties of metals

Category	Elastic modulus [N/mm ²]	Yield strength [N/mm ²]	Tensile strength [N/mm ²]	Elongation [%]
SM400A (9 mm)	2.06×10^5	293	447	31
SM400A (22 mm)	2.06×10^5	285	439	34
YGW12	2.06×10^5	460	540	-
Zinc sheet	1.18×10^5	-	118	-

Note: All elastic moduli and the other data of YGW12 and zinc sheet are nominal values.

To confirm the bonding strength between the zinc and steel, dedicated specimens were prepared, as shown in Figure 4-3. Each specimen consisted of two steel blocks with a 0.5 mm-thick zinc sheet sandwiched between them, forming a bonding specimen of dimensions 40 mm × 40 mm × 44.5 mm. These bonding test specimens underwent the same heat-treatment process as the fatigue experimental specimens to replicate the zinc-steel bonding conditions. The bonding strength was evaluated using tensile loading tests, as illustrated in Figure 4-4.

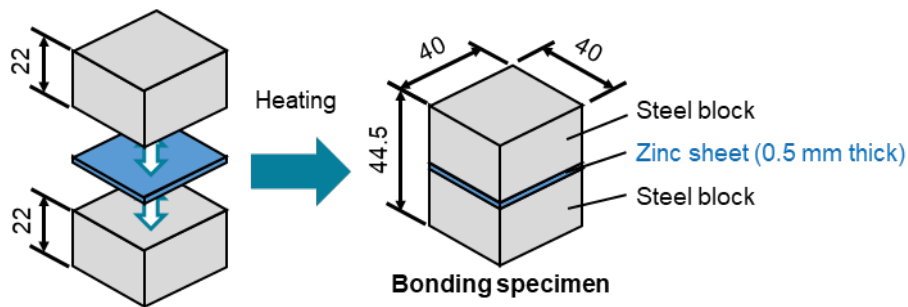


Figure 4-3 Details of the zinc-steel bonding specimen (unit: mm)

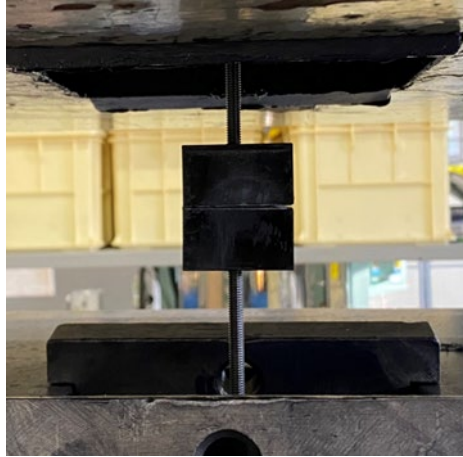


Figure 4-4 Axial tensile test setup

4.2.4 Safety distance

Using the same method as Subsection 3.2.4, the maximum temperature distribution of the steel plate during welding was estimated, as shown in Figure 4-5. Zinc has a boiling point of 907 °C. Using this temperature as the threshold, the safety distance is estimated to be at least 5 mm. Therefore, three groups of Z specimens were designed, each with a safety distance of 0 mm (no safety distance), 5 mm (exactly the safety distance threshold), and 10 mm (much greater than the safety distance threshold).

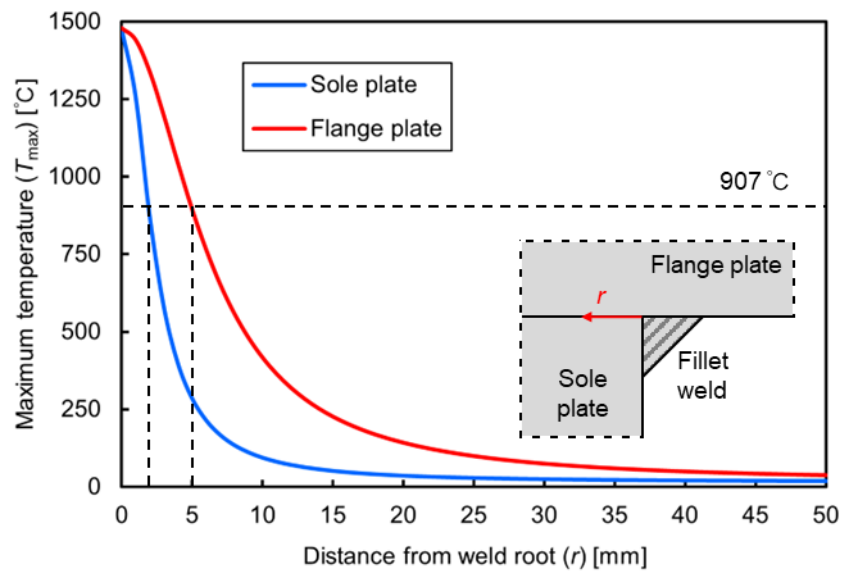


Figure 4-5 Maximum temperature distribution during welding (soft metal)

4.2.5 Setup and Measurements

The experimental setup is as illustrated in Figure 4-6. The fatigue test was conducted under a 4-point bending condition to replicate the loading condition typically experienced by fillet welds between the bottom flange and sole plate near steel bridge supports.

The fatigue experiments were conducted using a servo-hydraulic fatigue testing machine. The load ratio, defined as the ratio of minimum load to maximum load, was set at 0.1, and the loading frequency was maintained at 5 Hz. Twenty-five specimens were tested excluding the Z0 group. Thus, the test group comprised three W0, eight W0.5, six Z5, and eight Z10 specimens.

The initiation and propagation of fatigue cracks were monitored using magnetic particle testing. Figure 4-7 presents a detected fatigue crack. For consistency, the fatigue life was defined as the number of cycles applied until a fatigue crack exceeding 0.2 mm in length was detected at the weld root. The test was terminated if no cracks were found after two million load cycles.

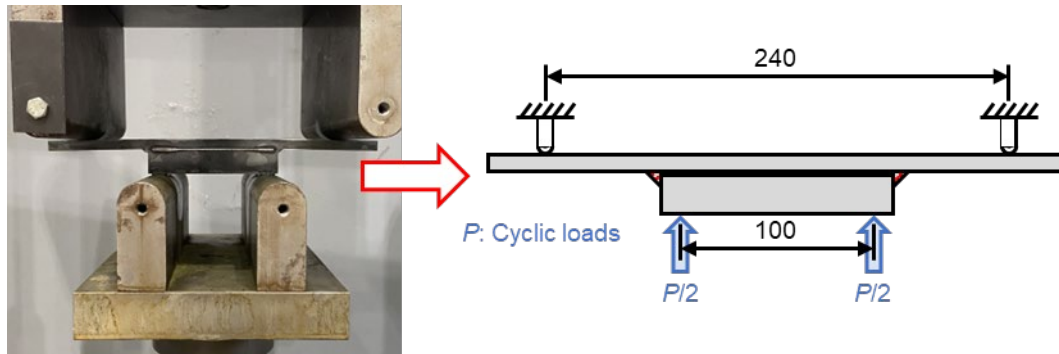


Figure 4-6 Fatigue experimental setup (unit: mm)

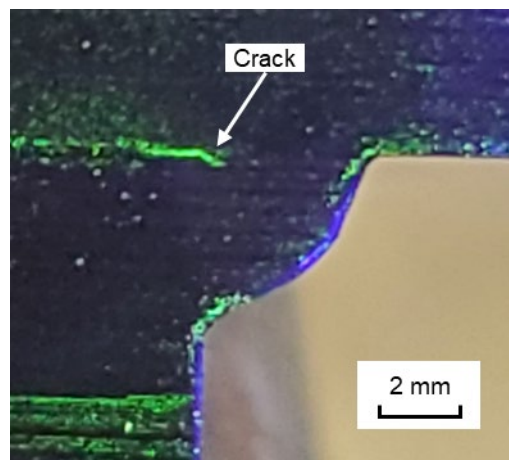
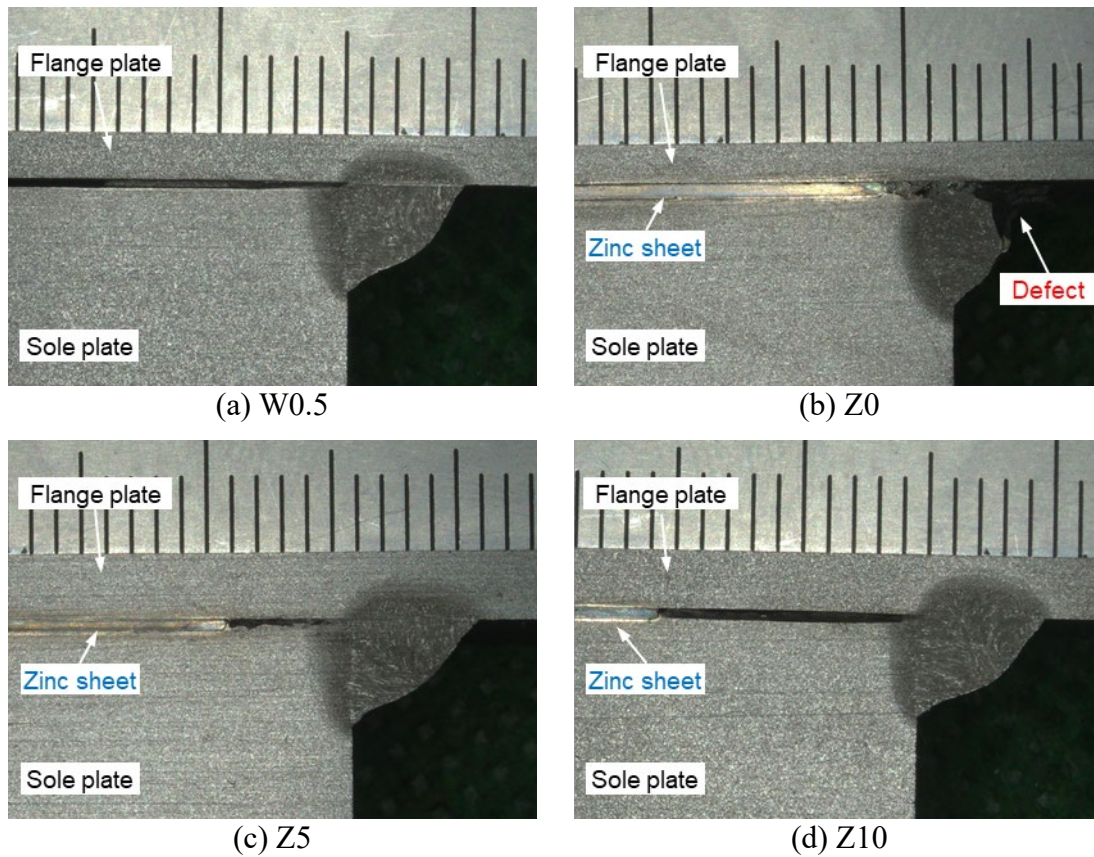


Figure 4-7 Root fatigue crack

4.3 Experimental Results

4.3.1 Welding Quality

Previous research on adhesive insertion highlighted persistent issues with weld stability, including incomplete root penetration and insufficient epoxy filling [93, 98]. In contrast, such defects were not observed when soft metal bonding-assisted welding was applied with zinc insertion. This improvement is largely due to the better controllability of the inserted metal sheet compared to liquid epoxy, ensuring a stable inserted volume. Nevertheless, the safety distance from the zinc sheet to the weld bead is important. As shown in Figure 4-8, welding failures frequently occurred in Z0 specimens, while both Z5 and Z10 specimens achieved weld quality comparable to that of the W0.5 specimens. The failure in Z0 specimens can be explained by the thermophysical properties of zinc. Specifically, zinc has a lower specific heat capacity and higher thermal conductivity than steel, as well as a boiling point far lower than the melting point of steel. Consequently, when zinc was placed too close to the weld, the heat input during arc welding caused rapid vaporization, leading to spattering, weld instability, and safety hazards. By contrast, the Z5 and Z10 specimens kept a sufficient safety distance between the zinc sheet and the weld, which effectively prevented zinc vaporization and ensured stable weld formation. These results demonstrate that it is essential to consider the mechanical bonding capability of the soft metal insertion. In addition, its thermal characteristics also need to be evaluated. To secure welding quality, a minimum distance from the soft metal insert to the weld root should be maintained. A safety distance of at least 5 mm is recommended for zinc insertion. Furthermore, a method similar to that presented in Subsections 3.2.4 and 4.2.4 can be applied to more accurately estimate the required safety distance.



Note: The minimum scale of the ruler is 1 mm.

Figure 4-8 Macrographs near the weld roots

4.3.2 *S-N* Relationship

Accurately calculating the nominal stress at the weld roots subjected to bending–shear coupling is difficult. A previous study evaluated fatigue strength of such weld roots using nominal stress but achieved limited accuracy [93]. Thus, similar to the approach adopted in Chapter 3, this chapter evaluates fatigue performance using the nominal stress at the top of the flange plate (Figure 4-9). Excluding specimens that endured more than two million cycles without failure, the other data points can be fitted by two distinct linear trends in the double logarithmic coordinate system. The blue line represents the trend fitted by a subset of Z specimens, while the red line represents the W specimens together with the remaining Z specimens. Most Z specimens demonstrated a notable improvement in fatigue strength, achieving a fatigue life equivalent to that of W specimens even under nominal stress amplitudes exceeding 1.7 times those of W specimens. However, some Z

specimens showed fatigue strength comparable to W specimens, suggesting variability in zinc–steel bonding strength. This issue will be further analyzed in Subsection 4.3.3. Moreover, W0.5 specimens showed a little higher fatigue strength than W0 specimens. This reduction is likely due to the sharper weld root geometry in W0 specimens, which promotes more serious stress concentration. This result will be further examined by numerical analysis later.

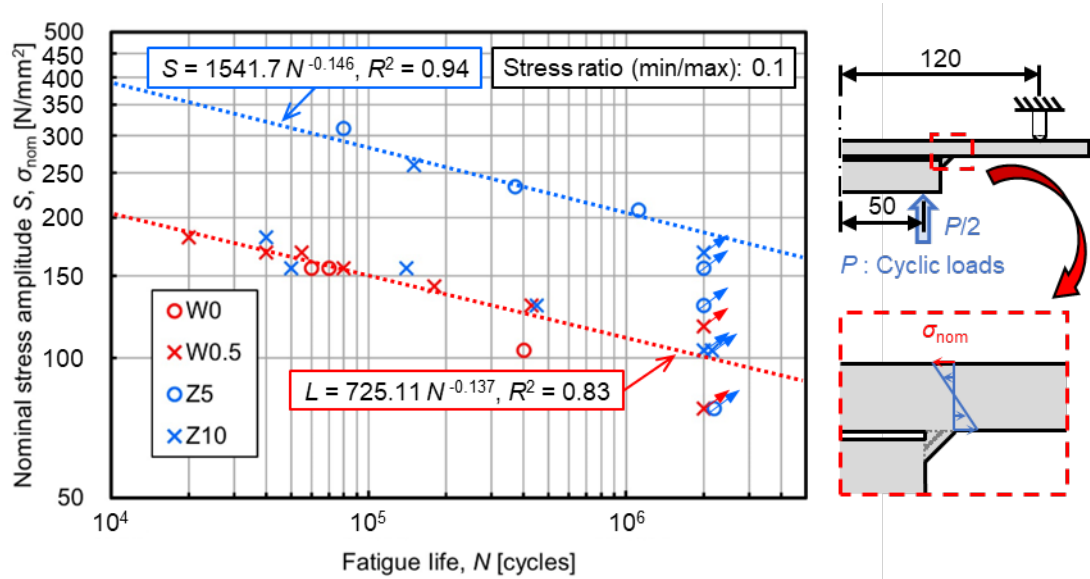


Figure 4-9 S – N relationship (soft metal insertion)

4.3.3 Zinc–steel Bonding Strength

Five specimens were tested to obtain zinc–steel bonding strength, and the results are summarized in Figure 4-10. In all cases, failure occurred within the zinc layer itself, leaving a thin zinc layer adhered to the interfaces of the steel blocks, as shown in Figure 4-11. This indicates that the bonding strength of zinc–steel interface was higher than the strength of the heated zinc layer. A large amount of fine internal defects generated during the liquefaction and subsequent solidification of the zinc, which markedly reduced its overall strength. The average strength measured was 0.92 N/mm^2 , but substantial variation occurred. This difference may be due to partial zinc outflow during heating. However, zinc outflow is not expected in actual bridges where the zinc sheet is sealed. In addition, this variability accounts for the variation in fatigue strength among the Z

specimens. Some with higher bonding strength showed enhanced resistance to weld root fatigue, as the bond provided a sustained reduction in local stress concentration. Conversely, specimens with weaker bonding likely experienced early bond failure, diminishing their ability to relieve weld root stress. Consequently, these specimens showed fatigue performance comparable to that of W specimens. It can be considered that the zinc–steel bond contributes to fatigue life extension through a self-sacrificial mechanism, a concept that will be examined in greater detail through numerical analysis.

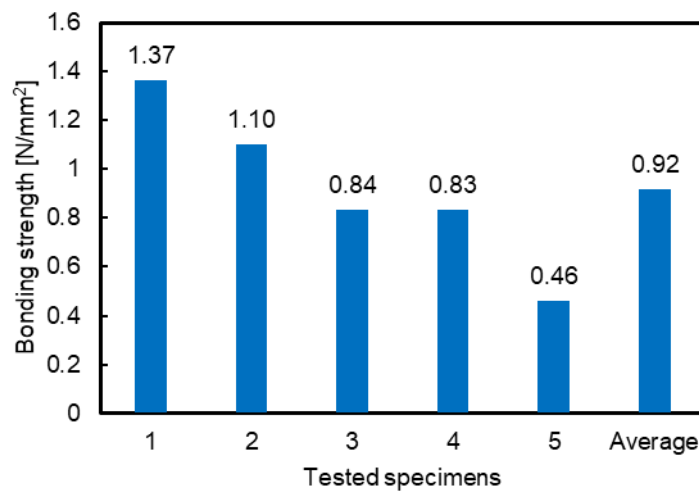


Figure 4-10 Zinc–steel bonding strength

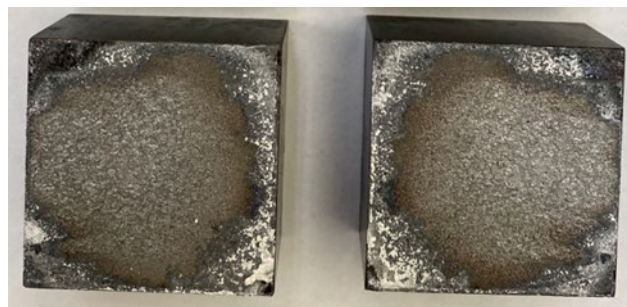
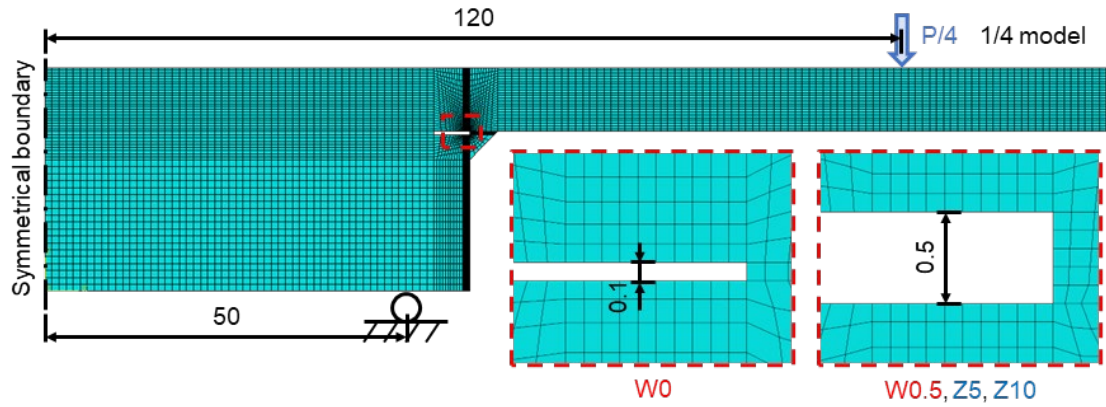


Figure 4-11 Interface of bonding failure

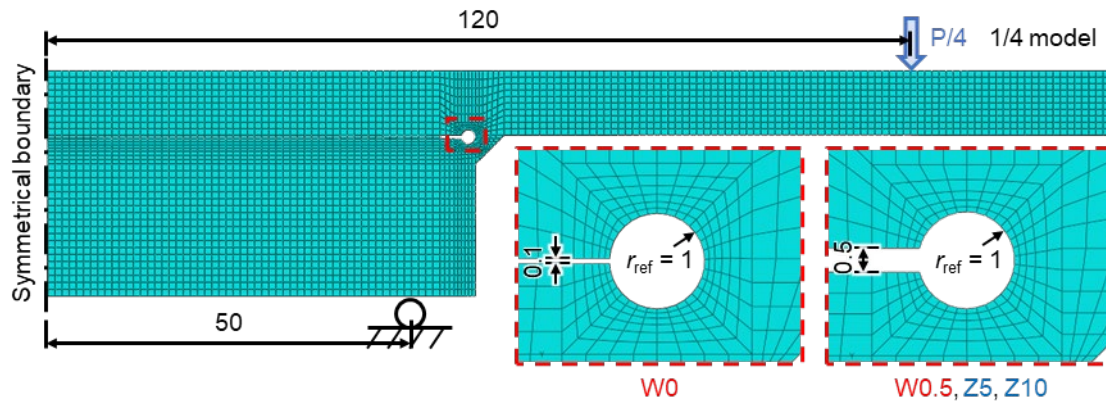
4.4 Finite Element Analysis

4.4.1 Finite Element Models

To evaluate fatigue stresses, three-dimensional elastic finite element models were constructed in ABAQUS CAE. Taking advantage of symmetry, one-quarter models were used to reduce computational cost. The geometric dimensions were consistent with those of the specimens shown in Figure 4-1, with idealized weld shapes, as illustrated in Figure 4-12 (a). In addition, the ENS models were developed with notches of a reference radius of 1 mm, as shown in Figure 4-12 (b).



(a) Ideal models



(b) ENS models

Figure 4-12 Fatigue analysis models (unit: mm)

The gap thickness between the bottom flange and sole plate was considered as 0.1 mm for W0 specimens and 0.5 mm for other groups of specimens. A refined mesh with a

mesh size of 0.1 mm was applied near the weld root, gradually transitioning to a global mesh size of 1 mm. For ENS models, the minimum element size is 0.2 mm, which is around the notch. All models employed 8-node hexahedral solid elements with reduced integration (C3D8R). The analysis was conducted under monotonic loading.

The steel plates and welds were considered as with identical properties. The elastic moduli were taken from Table 4-2, with Poisson's ratio set to 0.3. The bond layer was simulated using solid elements rigidly tied to the adjacent steel surfaces. Assuming ideal zinc-steel bonding which will never fail. The residual stresses were neglected, as they were partially released during specimen cutting.

4.4.2 Mechanism of Fatigue Improvement

The four specimen groups (W0, W0.5, Z5, and Z10) were assessed under a nominal stress amplitude of about 155 N/mm². The stress distributions of maximum principal stress obtained from the finite element models are present in Figure 4-13 and Figure 4-14. The weld root ENS for each model is summarized in Figure 4-15. All W specimens showed pronounced stress concentrations at the fillet weld root. In contrast, the Z specimens demonstrated a substantial reduction in weld root stress.

The results indicate that inserting zinc sheets significantly reduces the fatigue stress at the weld root. Specifically, the ENS in Z specimens are only about 10%–20% of that in W specimens. This demonstrates that zinc-steel bonding can effectively prevent weld root fatigue cracking. However, some Z specimens exhibiting a 70% improvement in fatigue strength, while the other Z specimens exhibited little improvement. This variation is likely due to differences in the strength and stability of the zinc-steel bond. Under cyclic loading, the zinc-steel bond may fail before fatigue cracks appear at the weld root, thereby extending the weld root's fatigue life through a self-sacrificial mechanism.

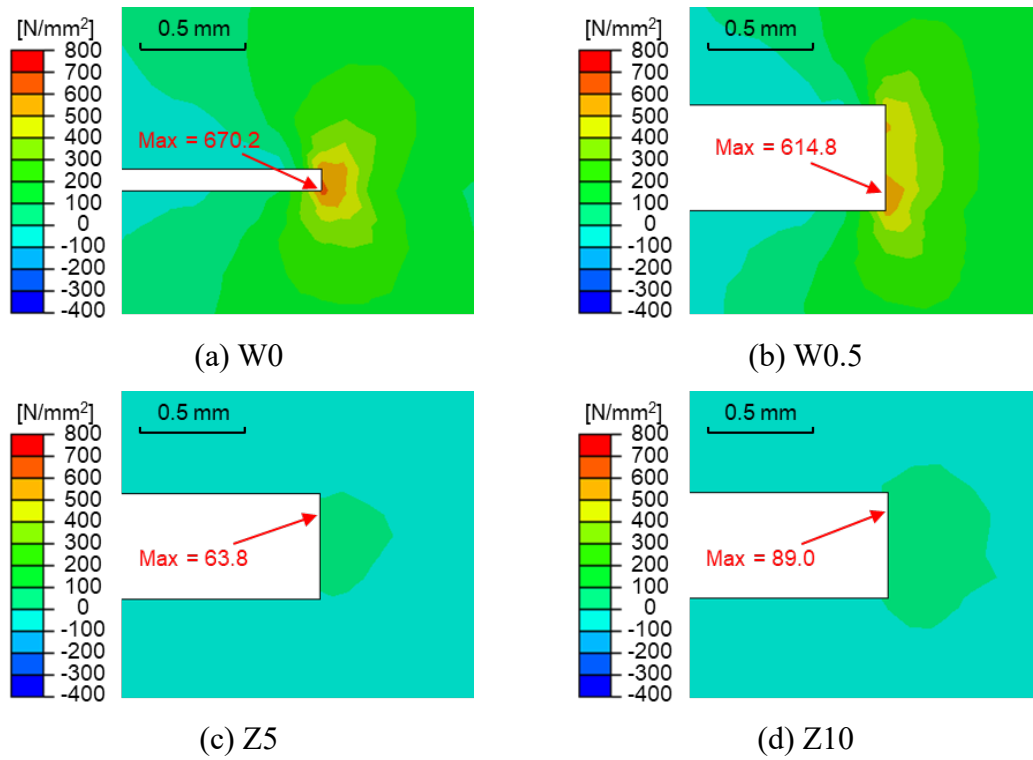


Figure 4-13 Maximum principal stress distributions of ideal models

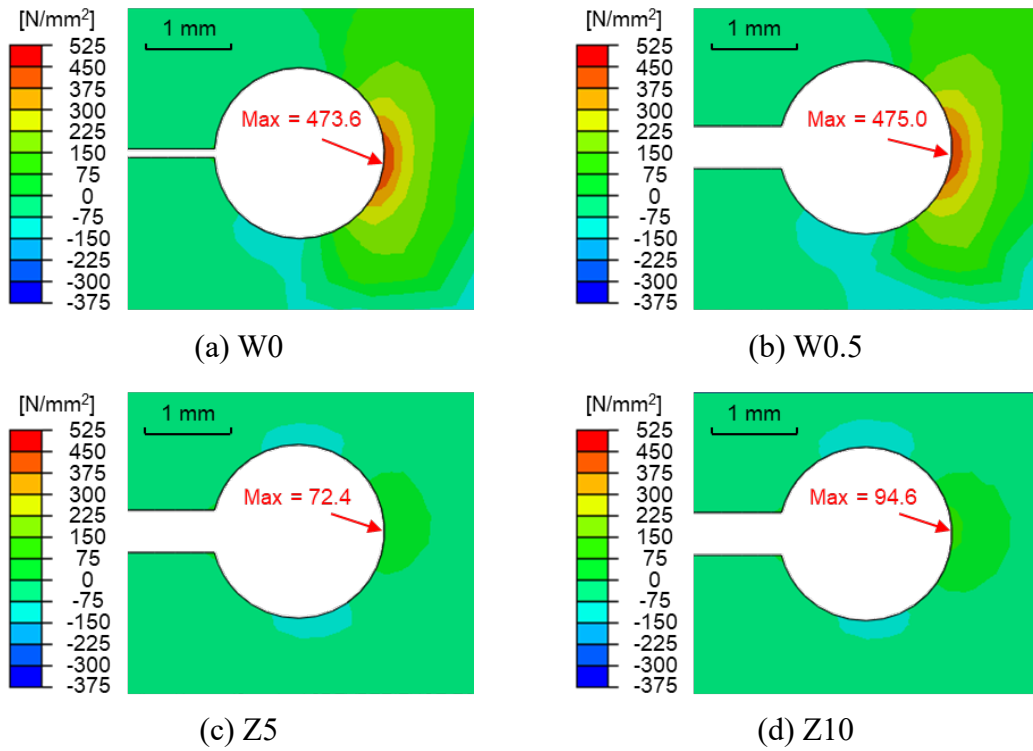


Figure 4-14 Maximum principal stress distributions of ENS models

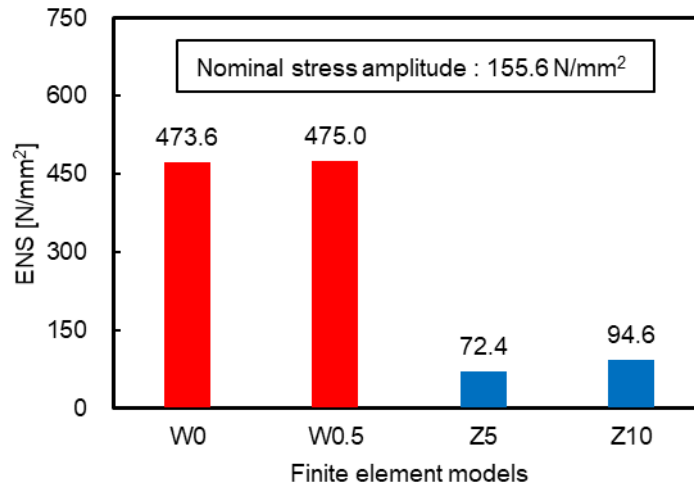


Figure 4-15 Comparison of weld root ENS (soft metal insertion)

The mechanism of weld root fatigue cracking can be divided into two stages for Z specimens with higher zinc-steel bonding strength. The two stages are before and after the zinc-steel bond fails. The fatigue stress at the weld root before the bond failure is only about 10%–20% of the stress after bond failure. Assuming the specimens obey the cumulative fatigue damage rule [37], the accumulated fatigue damage before bond failure is negligible. Based on the results in Figure 4-9, it can be inferred that this initial stage accounts for more than 95% of the total fatigue life of these specimens.

4.5 Summary

This chapter proposes a bonding-assisted welding method in which zinc is inserted between two steel plates to establish zinc-steel bonding, thereby enhancing the fatigue performance of the fillet weld root under bending-shear coupled loads. The effectiveness of this method was evaluated through both experimental and simulation analyses, leading to the following findings.

(1) In bonding-assisted welding, inserting zinc between the steel plates produces a more stable and higher-quality fillet weld compared to epoxy insertion, owing to better control of the inserted soft metal volume. Maintaining a sufficient safety distance (≥ 5 mm) between the zinc sheet and the weld root is essential to avoid welding defects.

(2) The formation of zinc-steel bonding improves the fatigue strength of the weld root under bending-shear coupled loads by approximately 70%. However, due to variability in bonding strength, some zinc-inserted specimens (Z) exhibited fatigue performance comparable to welding-only specimens (W).

(3) Finite element analysis confirms that zinc insertion substantially reduces the fatigue stress and deformation at the weld root. When the zinc-steel bonding is effective, the fatigue stress in Z specimens is reduced to approximately 10%–20% of that in W specimens, highlighting the potential of zinc-assisted bonding to enhance fillet weld fatigue performance.

CHAPTER 5

OPTIMIZATION OF BONDING-ASSISTED WELDING

5.1 Introduction

In Chapters 3 and 4, two ways of bonding-assisted welding techniques, adhesive insertion and soft metal insertion, were experimentally investigated. However, these experiments were performed on simplified specimens, which do not fully capture the structural complexity near actual bridge bearings. To address this limitation, this chapter adopts the multi-precision finite element models developed in Chapter 2 to study the implementation of bonding-assisted welding in actual bridges. The simulation of the bond layer follows the same method verified in Chapters 3 and 4, ensuring the accuracy and reliability of the analysis. Based on this approach, a series of numerical analyses was conducted to evaluate the effectiveness of bonding-assisted welding on actual bridges and to explore strategies for optimizing its application.

5.2 Finite Element Models

The finite element models were developed in ABAQUS CAE by integrating the global girder models with the local support models, as illustrated in Figure 2-3 and Figure 2-5. The local support models were categorized into two types: the conventional welding (W) model and the bonding-assisted welding (WB) model. The differences between these two models are shown in Figure 5-1.

The W models are consistent with the models in Chapter 2. They replicate the configurations of the real-world bridges while incorporating a circular notch at the fillet weld roots to obtain ENS. On the contrary, the WB models include a groove of appropriate thickness on the sole plate accommodating the bond layer and reducing the risk of welding defects [93].

The bond layer was simulated using solid elements, and the bonding was modeled with “tie” constraints, which couple the degrees of freedom of the slave surface to the master surface, as shown in Figure 5-2. The bonding materials are the same epoxy and

zinc as in the experiments in Chapters 3 and 4, respectively. Their material properties are shown in Table 5-1.

The effects of two key parameters were studied in detail through parameter analysis: the unfilled distance (d) from the weld root to the bond layer and the bond layer thickness (t_b), which are defined as shown in Figure 5-1.

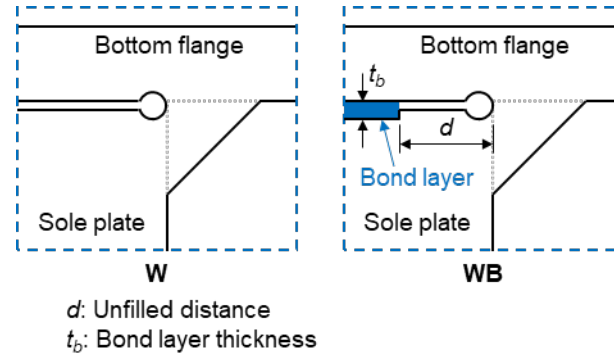


Figure 5-1 Differences between W and WB models

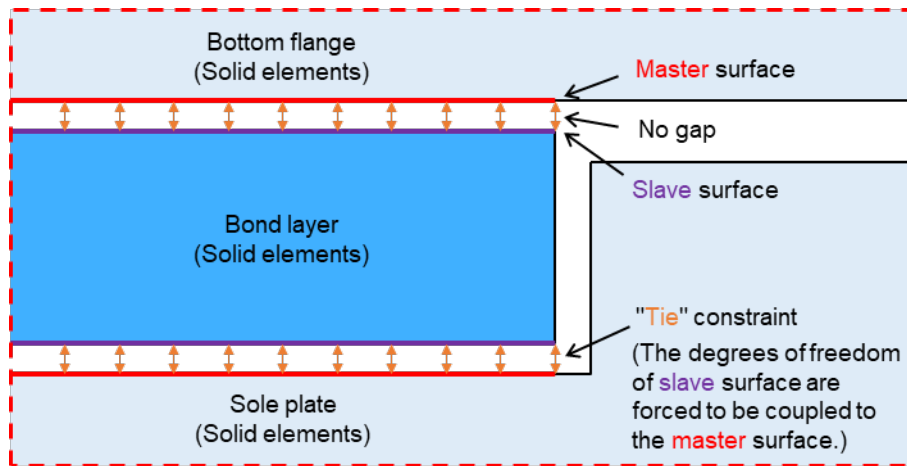


Figure 5-2 Simulation of bond layer

Table 5-1 Mechanical properties of bonding materials

Material	Elastic modulus [N/mm ²]	Poisson's ratio
Epoxy	3.8×10^3	0.35
Zinc	1.18×10^5	0.3

5.3 Fatigue Evaluation Results

5.3.1 Effect of Unfilled Distance

Taking Bridge C-2 as an example, the unfilled distance d ranging between 10 mm and 80 mm, while the bond layer thickness t_b was 0.50 mm consistently. The relationship between the weld root ENS and unfilled distance is shown in Figure 5-3. Fatigue life prediction was carried out using the FAT 225 design curve (Figure 2-8). In the results, WB-E and WB-Z denote epoxy insertion and zinc insertion, respectively.

The ENS at the weld root increased as the unfilled distance grew. To achieve a predicted fatigue life exceeding two million cycles, the ENS must remain below 225 N/mm². Accordingly, the allowable unfilled distances were less than 50 mm for WB-E and less than 60 mm for WB-Z. Because zinc has a higher elastic modulus than epoxy, it provides greater restraint stiffness, resulting in lower ENS values for WB-Z than WB-E. However, as discussed in Subsection 4.4.2, zinc–steel bonding enhances weld root fatigue life at the expense of its own strength, and bond failure may occur prior to weld root cracking. Thus, the actual fatigue life may be somewhat shorter than predicted. Nevertheless, the improvement compared to conventional welding is substantial, confirming zinc insertion as an effective fatigue improvement measure.

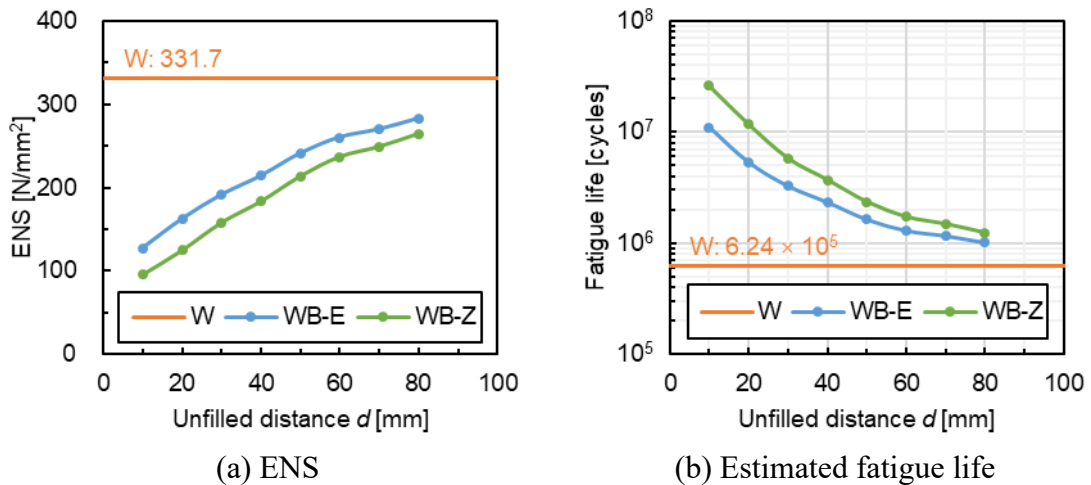


Figure 5-3 Effect of unfilled distance

Figure 5-4 shows the relationship between the highest maximum principal tensile stress $\sigma_{1,\max}$ within the bond layer and the unfilled distance. Both epoxy insertion and zinc

insertion exhibited similar trends: $\sigma_{1,\max}$ initially increased with d , reached a peak, and then gradually decreased and stabilized. To minimize stress in the bond layer and further improve weld root fatigue performance, the unfilled distance should be kept as small as possible. However, a minimum safety distance is necessary. Based on the simplified model experiments presented in Chapters 3 and 4, the safety distance should be at least 15 mm for epoxy insertion and at least 5 mm for zinc insertion.

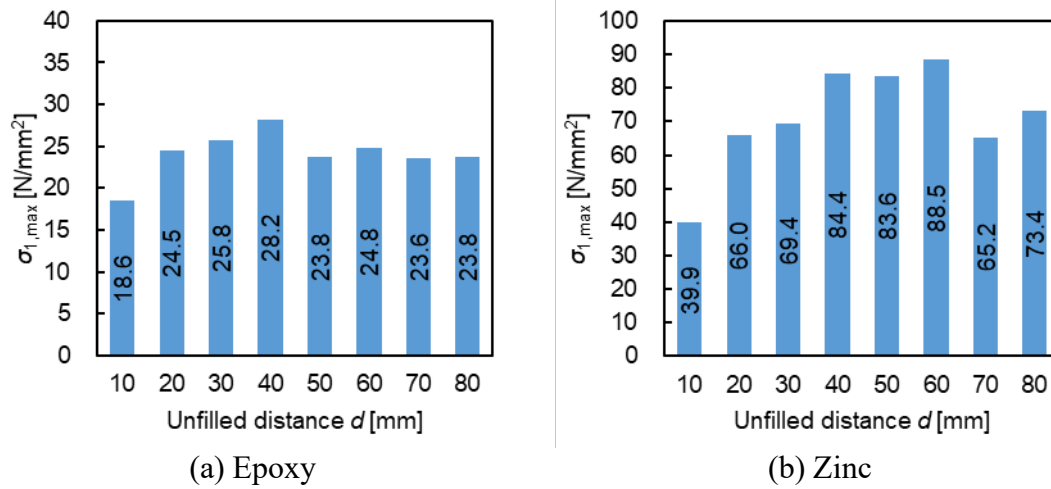


Figure 5-4 Maximum principal tensile stresses (unfilled distance)

5.3.2 Effect of Bond Layer Thickness

As noted in Subsection 3.4.3, the influence of epoxy thickness on weld root stress was previously examined. Building on this, the present analysis considered the effect of bond layer thickness t_b . The thickness varied from 0.25 mm to 1.50 mm, while the unfilled distance d of 20 mm was fixed. The analysis results are shown in Figure 5-5. For the WB-E model, the ENS at the weld root increased with increasing t_b , whereas the WB-Z model showed little sensitivity to t_b . This difference arises from the material properties: the elastic modulus of epoxy is much lower than that of steel, while the modulus of zinc is closer to that of steel. As a result, increasing epoxy thickness significantly reduces restraint stiffness, whereas zinc thickness has little effect. Nevertheless, even with a bond layer thickness of 1.50 mm, which exceeds common engineering practice, both WB models maintained predicted fatigue lives greater than two million cycles. Therefore, weld root fatigue life is not the governing factor for determining bond layer thickness.

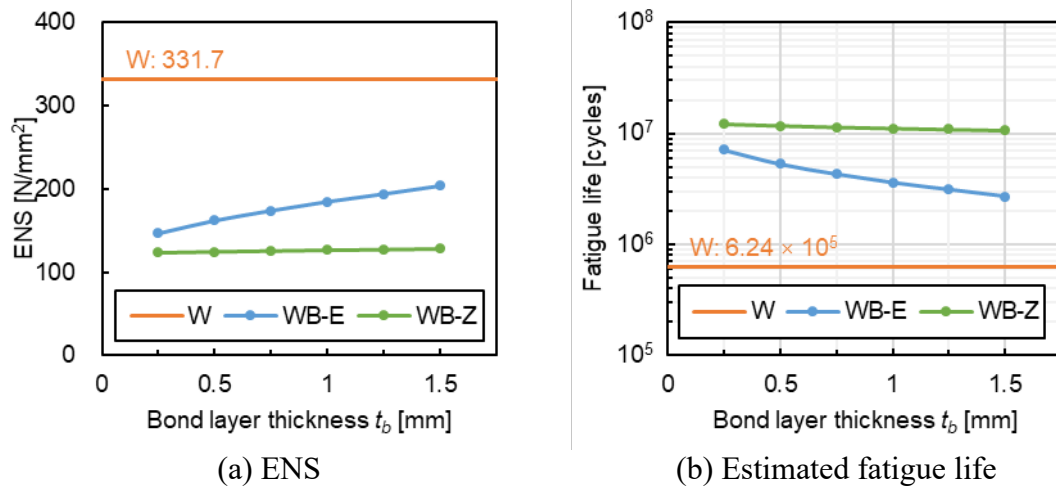


Figure 5-5 Effect of bond layer thickness

Figure 5-6 illustrates the relationship between the highest maximum principal tensile stress $\sigma_{1,\max}$ within the bond layer and bond layer thickness. Both epoxy and zinc models followed the same trend: $\sigma_{1,\max}$ decreased with increasing t_b . Thus, increasing the bond layer thickness can help reduce stress concentrations in the bond layer. For practical application, a thickness in the range of 0.50–1.00 mm is suggested. This range provides substantial enhancement in fatigue performance of weld root while lowering stress within the bond layer. In the case of epoxy, limiting stress is particularly important because long-term exposure may accelerate material degradation due to aging and fatigue [102]. For zinc insertion, maintaining lower stress can also extend the service life of zinc–steel bonding.

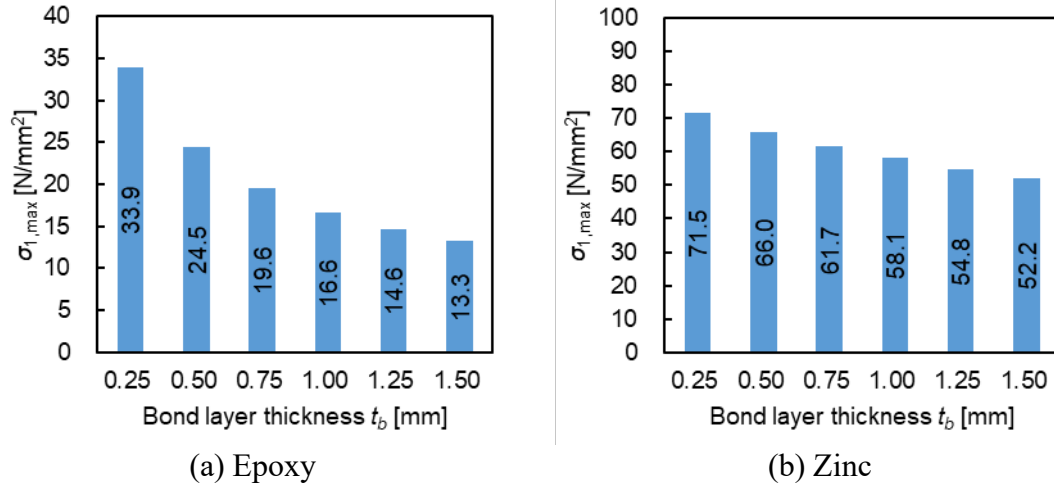


Figure 5-6 Maximum principal tensile stresses (bond layer thickness)

5.4 Stress Distribution

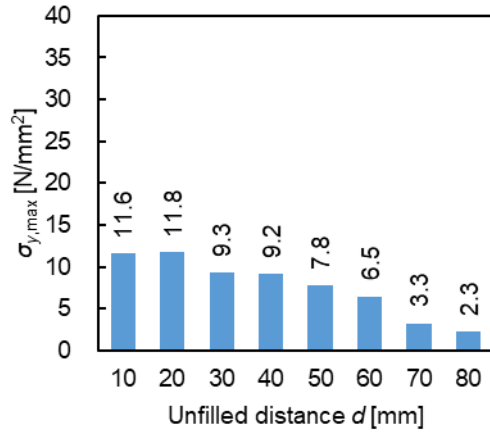
5.4.1 Adhesive Insertion

The preceding analyses revealed that the highest maximum principal tensile stress within the bond layer reached high levels in certain conditions, raising concerns about potential failure in practical applications. Moreover, the relationship between the maximum principal tensile stress $\sigma_{1,max}$ and the unfilled distance d , was found to be neither monotonic nor linear. These findings highlight the need for a more detailed investigation of the stress distribution in the epoxy bond layer.

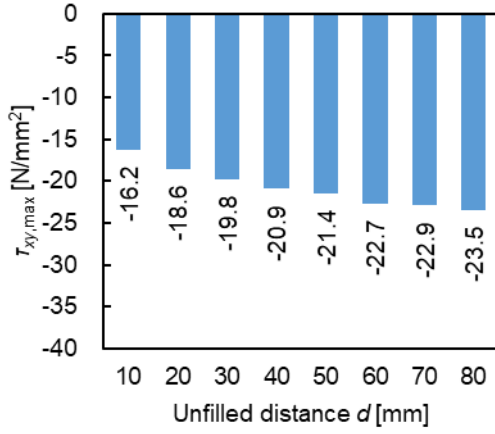
In actual engineering applications, epoxy strength is generally evaluated with respect to both axial tensile strength and shear strength. Accordingly, stress components were examined in this study. Figure 5-7 and Figure 5-8 show the relationships between stress components and unfilled distance d or bond layer thickness t_b , respectively. The analyzed stress components include the normal stress σ_y , the shear stresses τ_{xy} and τ_{yz} . The corresponding maximum values for them are $\sigma_{y,max}$, $\tau_{xy,max}$, and $\tau_{yz,max}$, respectively. In addition, Figure 5-9 to Figure 5-11 show the stress distributions within the bond layer for unfilled distance equals to 10 mm, 40 mm, and 80 mm, respectively, while t_b equals to 0.50 mm. Among them, d equals to 40 mm corresponds to the unfilled distance at which $\sigma_{1,max}$ reaches its peak.

In Figure 5-7, when $d < 20$ mm, the normal stress $\sigma_{y,\max}$ remains nearly constant at approximately 11.7 N/mm^2 . For $d \geq 20$ mm, $\sigma_{y,\max}$ decreases steadily with increasing d . In contrast, the shear stresses $\tau_{xy,\max}$ and $\tau_{yz,\max}$ increase progressively as d grows. The stress distribution results (Figure 5-9 through Figure 5-11) indicate that the maximum principal tensile stress consistently occurs near the edge of the epoxy layer, where both σ_y and τ_{yz} also reach their peak values. At $d = 10$ mm, the shear stress τ_{xy} at this edge is relatively low ($- 8.2 \text{ N/mm}^2$), but it rises sharply to $- 20.9 \text{ N/mm}^2$ at $d = 40$ mm, producing a substantial increase in the maximum principal tensile stress $\sigma_{1,\max}$. For $d \geq 40$ mm, σ_y at the edge decreases significantly, while τ_{xy} and τ_{yz} increase only slightly, resulting in a nearly stable $\sigma_{1,\max}$ of about 24 N/mm^2 . This phenomenon can also be interpreted as an aging process of epoxy under environmental causes. As the outer adhesive weakens, the effective unfilled distance increases, and the load carrying mechanism gradually shifts from restraining normal displacements to restraining shear displacements. Throughout this process, σ_y remains relatively low (below 12 N/mm^2), whereas τ_{xy} remains dominant, rising from $- 16.2 \text{ N/mm}^2$ to $- 23.5 \text{ N/mm}^2$. Consequently, the shear strength of epoxy–steel bonding strength emerges as a key factor governing the failure of bonding-assisted welding by epoxy insertion.

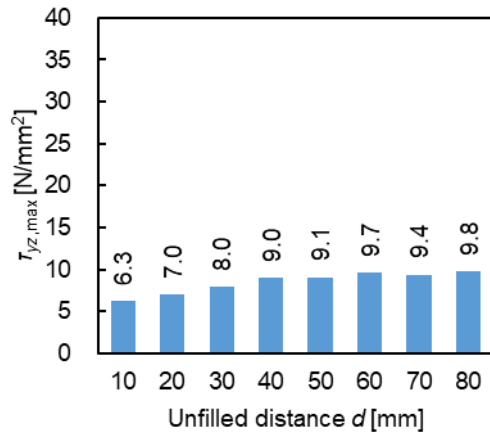
The effect of bond layer thickness is presented in Figure 5-8. All three stress components, $\sigma_{y,\max}$, $\tau_{xy,\max}$, and $\tau_{yz,\max}$, decrease as t_b increases. Even at the thinnest layer ($t_b = 0.25$ mm), $\sigma_{y,\max}$ and $\tau_{yz,\max}$ are relatively modest (15.6 N/mm^2 and 9.9 N/mm^2 , respectively), remaining below typical bonding strength of epoxy–steel joints. Nevertheless, $\tau_{xy,\max}$ reaches $- 24.4 \text{ N/mm}^2$, exceeding typical shear strength of epoxy. As a result, bond layer thickness $t_b \geq 0.50$ mm is suggested to avoid bond failure. Considering both fatigue performance improvement and construction feasibility, the optimal range of epoxy layer thickness lies between 0.50 and 1.00 mm, consistent with the findings in Subsection 5.3.2. Overall, the stress state within the epoxy bond layer depends strongly on both unfilled distance and thickness. Depending on the unfilled distance, the bond layer stress may be dominated either by normal stress or by shear stress. This underscores the importance of evaluating both tensile and shear strength of adhesive materials in design, since failure may be governed by either mode.



(a) Normal stress component σ_y

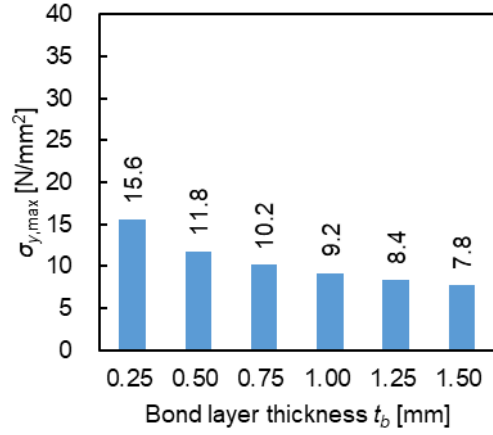


(b) Shear stress component τ_{xy}

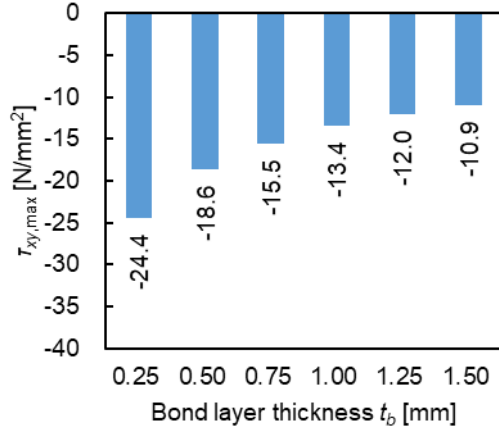


(c) Shear stress component τ_{yz}

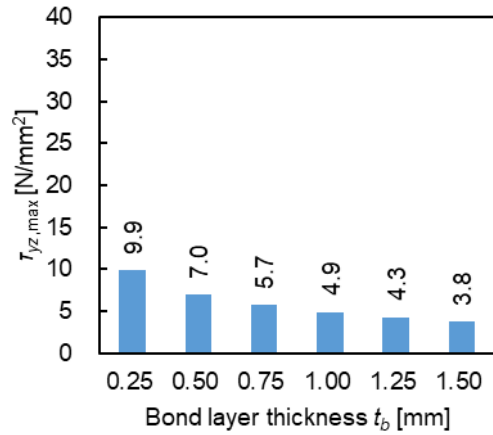
Figure 5-7 Variation of stress components with unfilled distance (epoxy)



(a) Normal stress component σ_y

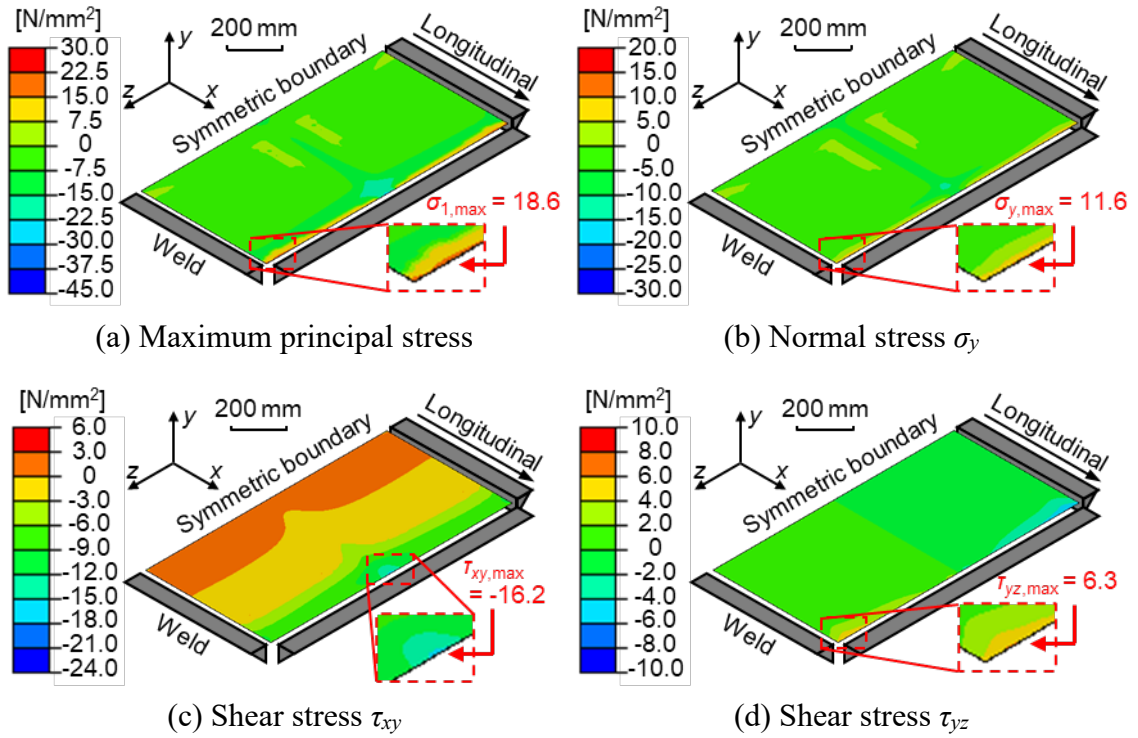
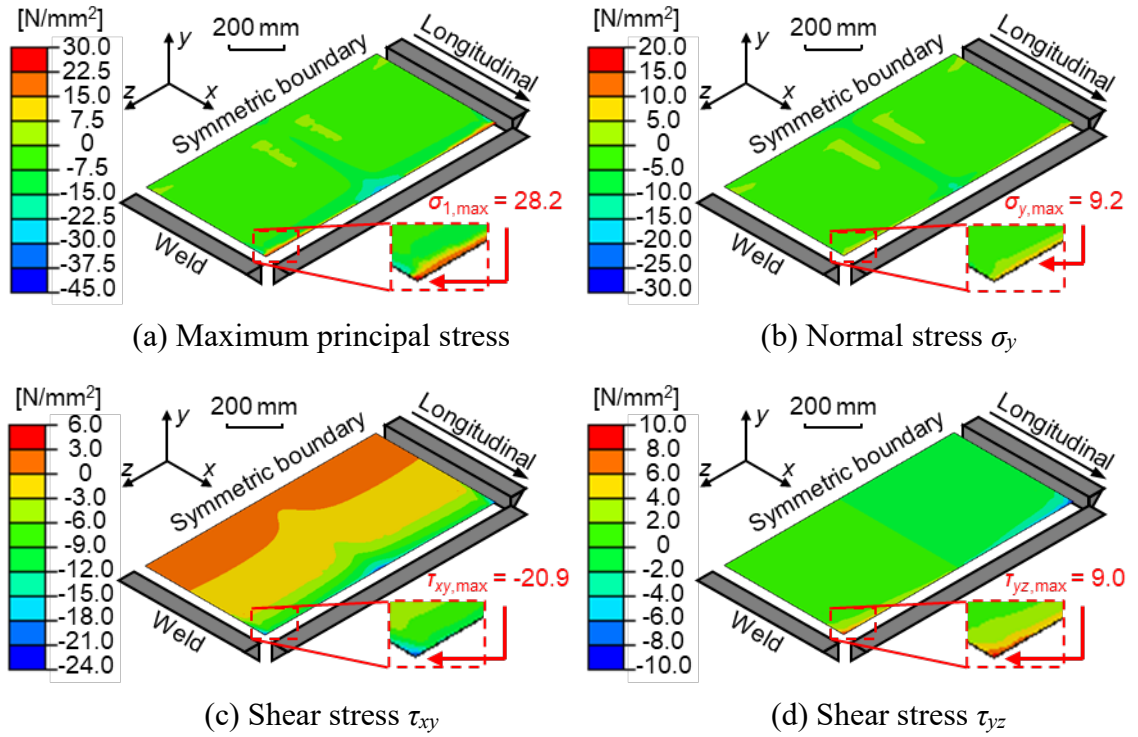


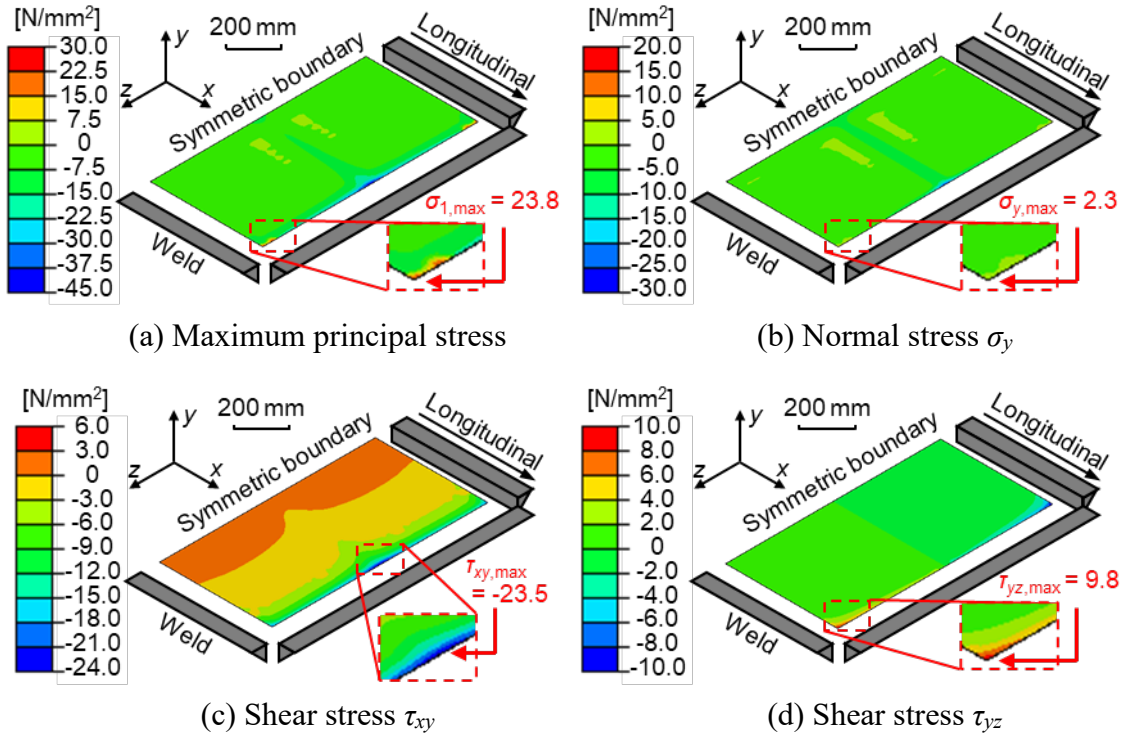
(b) Shear stress component τ_{xy}



(c) Shear stress component τ_{yz}

Figure 5-8 Variation of stress components with bond layer thickness (epoxy)


 Figure 5-9 Stress distribution of the epoxy layer ($d = 10$ mm, $t_b = 0.50$ mm)

 Figure 5-10 Stress distribution of the epoxy layer ($d = 40$ mm, $t_b = 0.50$ mm)


 Figure 5-11 Stress distribution of the epoxy layer ($d = 80$ mm, $t_b = 0.50$ mm)

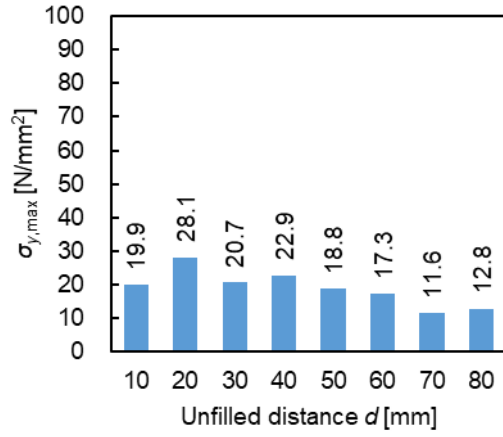
5.4.2 Soft Metal Insertion

Stress analyses were also performed for bonding-assisted welding using zinc insertion. The variations of stress components with unfilled distance d and bond layer thickness t_b are presented in Figure 5-12 and Figure 5-13, respectively. The stress distributions within the zinc layer for d equals to 10 mm, 60 mm, and 80 mm, and t_b equals to 0.50 mm are illustrated in Figure 5-14 through Figure 5-16. Among these, d equals to 60 mm corresponds to the unfilled distance at which the maximum principal tensile stress $\sigma_{1,\max}$ reaches its peak value.

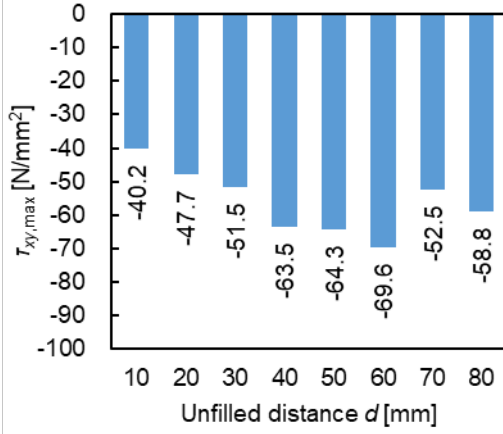
As shown in Figure 5-12, the variation trends of each stress component with the change of unfilled distance resemble those observed for epoxy insertion. Specifically, the stresses first increase with d and then decrease after reaching a peak. However, in the case of zinc insertion, the stress state is consistently dominated by the shear stress $\tau_{xy,\max}$, which attains its maximum at d equals to 60 mm. The shear stress $\tau_{yz,\max}$ follows a similar trend. This indicates that, for zinc insertion, the shear strength of zinc–steel bonding is a key

factor. The effect of bond layer thickness is shown in Figure 5-13. The overall patterns of variation are similar to those observed in epoxy insertion, with stresses decreasing as t_b increases. However, the magnitude of these changes is less pronounced for zinc insertion, indicating that bond layer thickness has a relatively minor influence on the stress state when zinc is used as the bond layer.

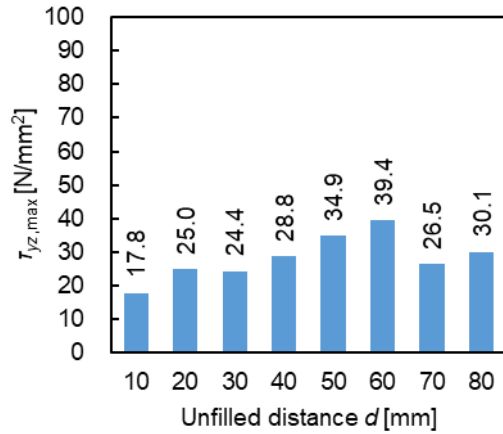
The stress distribution results are shown in Figure 5-14 to Figure 5-16. Similar to the epoxy case, the maximum stresses occur at the edge of the bond layer, where local stress concentrations develop. Nonetheless, because zinc has a much higher elastic modulus compared to epoxy, the stresses within the zinc layer are significantly greater than those observed in the epoxy layer under comparable conditions. While the zinc–steel bonding strength measured in this study did not reach such levels, previous studies have reported zinc–steel bonding strength exceeding 50 N/mm^2 [103]. Therefore, it is recommended that the unfilled distance in zinc insertion should not exceed 20 mm to avoid excessive stress development within the bond layer. Furthermore, a minimum safety distance of 5 mm between the bond layer and the weld root should be maintained to ensure both bonding stability and weld quality.



(a) Normal stress component σ_y

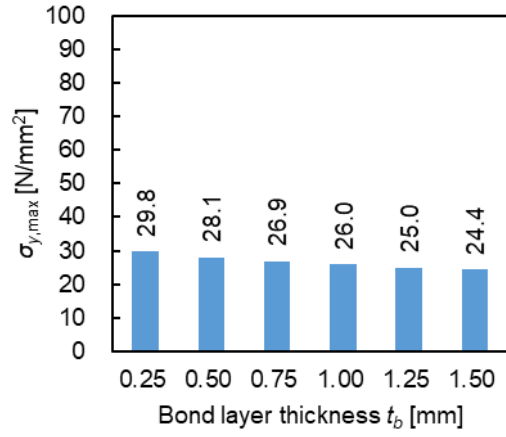


(b) Shear stress component τ_{xy}

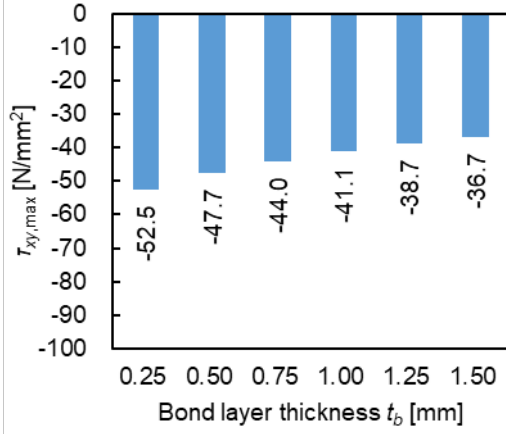


(c) Shear stress component τ_{yz}

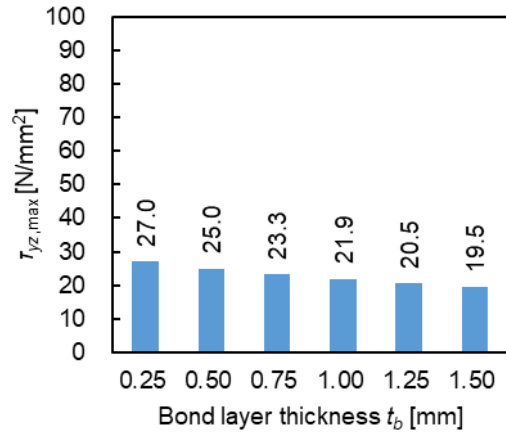
Figure 5-12 Variation of stress components with unfilled distance (zinc)



(a) Normal stress component σ_y

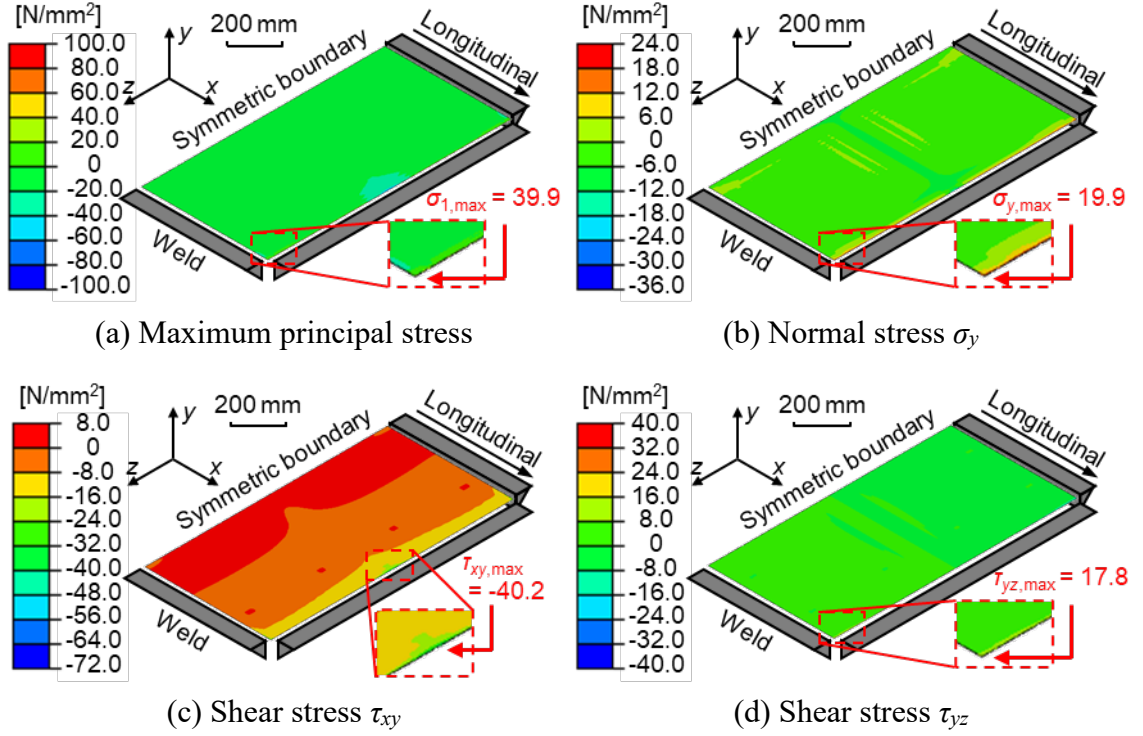
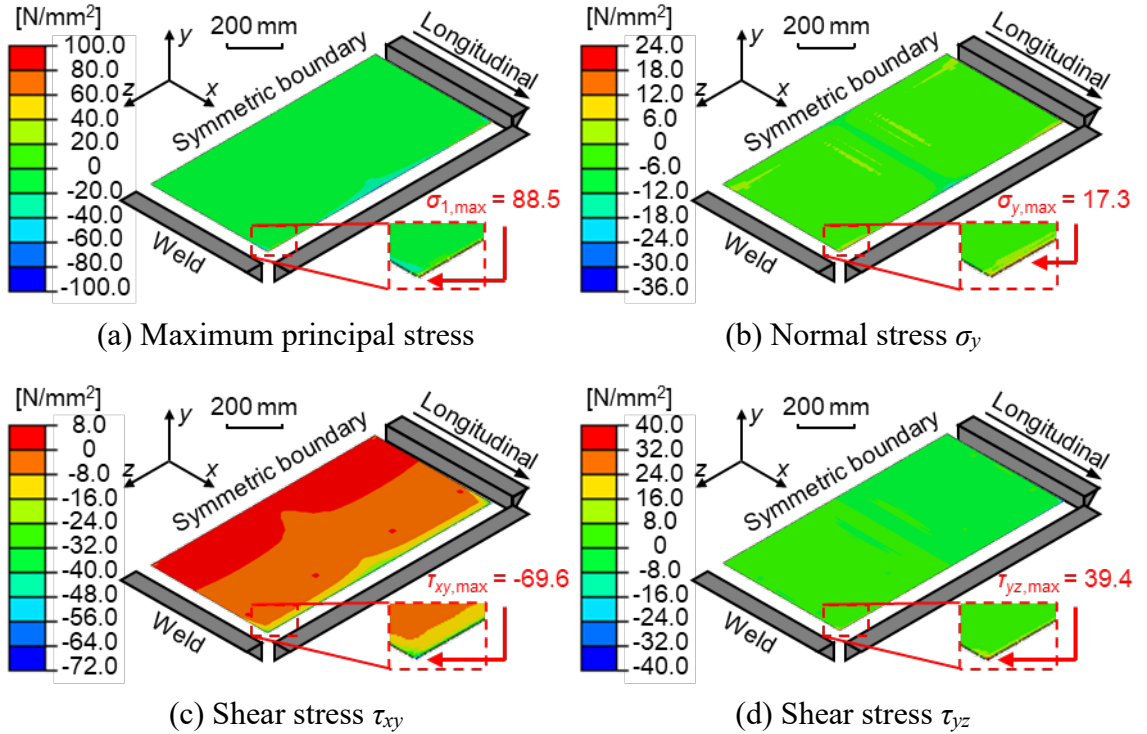


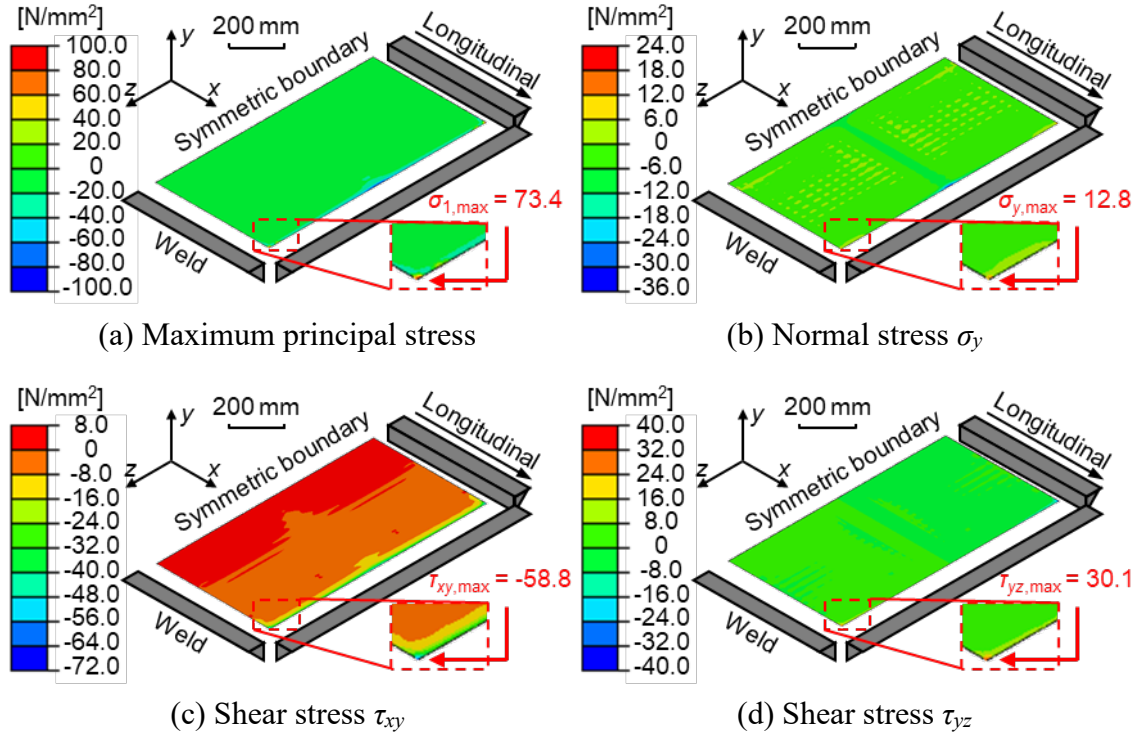
(b) Shear stress component τ_{xy}



(c) Shear stress component τ_{yz}

Figure 5-13 Variation of stress components with bond layer thickness (zinc)


 Figure 5-14 Stress distribution of the zinc layer ($d = 10$ mm, $t_b = 0.50$ mm)

 Figure 5-15 Stress distribution of the zinc layer ($d = 60$ mm, $t_b = 0.50$ mm)


 Figure 5-16 Stress distribution of the zinc layer ($d = 80$ mm, $t_b = 0.50$ mm)

5.5 Comparison and Discussions

5.5.1 Fatigue Improvement

Based on the results presented in the preceding sections, representative conditions were selected for the evaluation of bonding-assisted welding. For adhesive insertion, an unfilled distance of 20 mm and a bond layer thickness of 0.50 mm were adopted. The same conditions were also applied to soft metal insertion to allow a direct comparison of the two methods. The effectiveness of bonding-assisted welding was then assessed using the five representative bridges introduced in Chapter 2. The results are summarized in Figure 5-17. Across all bridges, the application of bonding-assisted welding resulted in a noticeable reduction in the weld root ENS, with reductions ranging from 14% to 62%. Among the two insertion methods, zinc insertion consistently exhibited a stronger stress-reducing effect compared with epoxy insertion. This can be attributed to higher elastic modulus of zinc, which provides greater restraint stiffness and thereby suppresses stress concentration at the weld root more effectively. The improvement in fatigue life was also remarkable. For the most improved bridge, C-2, the predicted fatigue life increased by

more than 8.5 times compared with conventional welding. In contrast, for the least improved bridge, C-5, severe stress concentrations occurred at the sole plate corners due to large longitudinal and transverse bending moments acting on the supports, which limited the improvement achievable through bonding-assisted welding. However, even for C-5, the predicted fatigue life increased by at least 1.5 times compared with conventional welding.

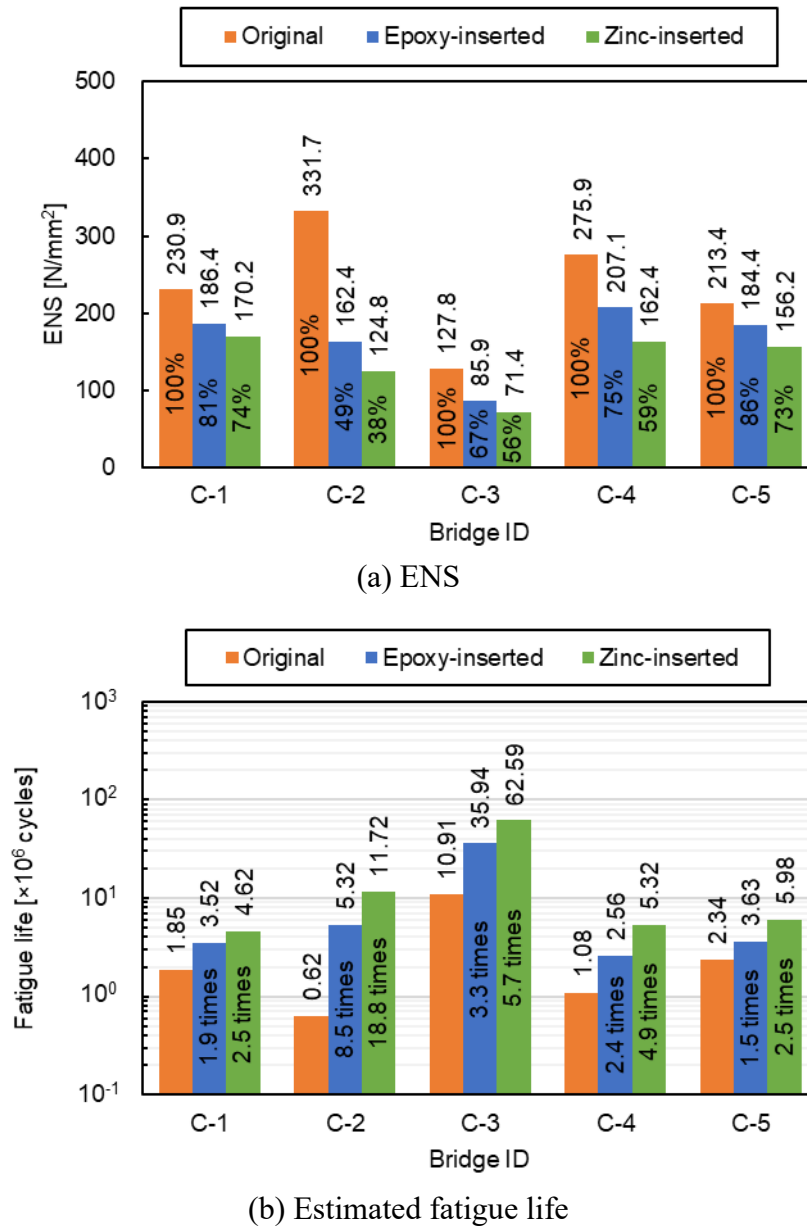


Figure 5-17 Comparison of effect with bonding-assisted welding

The finite element models assumed idealized zinc–steel bonding and did not account for bond failure, it is expected that the actual fatigue life of zinc-inserted models may be somewhat lower than the predicted values. Nonetheless, the results clearly demonstrate that both adhesive and soft metal insertions provide substantial improvements in weld root fatigue performance.

5.5.2 Application Feasibility

Epoxy insertion encountered challenges in controlling the volume of epoxy due to its liquid-like consistency and high viscosity before curing. Insufficient epoxy can lead to incomplete filling and limited fatigue improvement, while excess epoxy tends to overflow, making it difficult to maintain a safe distance from the weld and often resulting in welding defects (Figure 5-18). Even if heat-resistant rubber was added to prevent epoxy from overflowing, it cannot guarantee that there will be no welding defects. In contrast, zinc, as a solid metal, does not have the above problems. Welding defects can be avoided by calculating the safe distance in advance. Therefore, zinc insertion has higher controllability on welding quality.

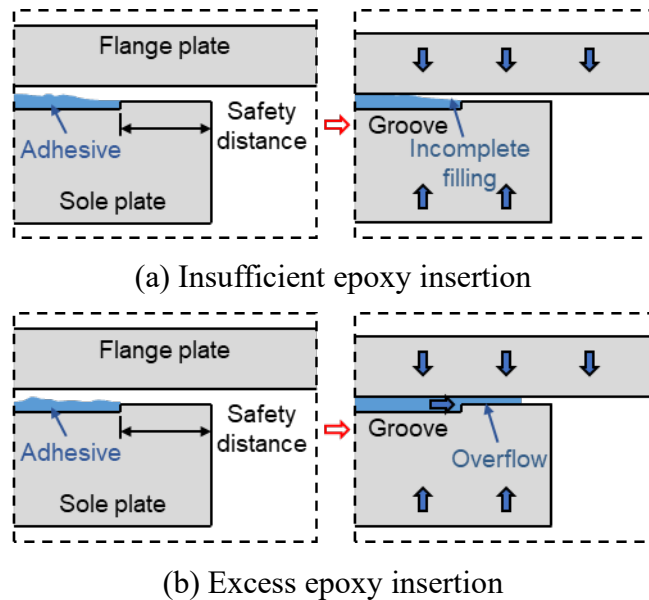


Figure 5-18 Defects of epoxy insertion

However, zinc insertion requires an additional heat treatment process. Specifically, the zinc sheet must be heated to at least above its melting point (420 °C) and maintained at that temperature for several hours. This requirement creates difficulties for large structural components and on-site construction. Consequently, zinc insertion is primarily applicable to workshops and relatively small structural components. In contrast, epoxy insertion does not require heating treatment, although it does require curing, and is therefore more suitable for on-site construction.

In addition to fabrication, durability is another important concern. Epoxy is prone to aging, and its typical lifespan in outdoor environments is approximately ten years [104]. In the application scenario considered in this study (bridge supports), the epoxy is sandwiched between two steel plates and sealed by welding on all sides. This configuration makes it difficult for the epoxy to be oxidized by air. Consequently, the lifespan of the epoxy in such conditions may be longer than expected. However, even in such an airtight environment, epoxy may gradually degrade in performance over time. Conversely, zinc exhibits superior long-term stability and durability under sealed conditions, significantly outperforming epoxy.

Nevertheless, zinc insertion also presents limitations. The primary problem is its lower bonding strength and stability compared with epoxy. Although the zinc–steel bonding strength measured in this study was below that of epoxy, other studies have reported values exceeding 50 N/mm² [103]. These findings suggest that zinc insertion holds greater long-term potential than epoxy insertion, if bonding strength can be further improved. A summary of the advantages and disadvantages of the two bonding-assisted welding methods is presented in Table 5-2, including a comparison with the tap bolts method (Subsection 1.3.3) [88].

In practice, epoxy insertion is more suitable for bridges experiencing high stress levels, where strong bonding strength is required to ensure fatigue resistance at critical locations. In contrast, zinc insertion is better suited to bridges subjected to relatively lower stress levels. This method ensures stable welding quality, extends service durability, and does not compromise the structure's inherent fatigue life. In addition, epoxy insertion is more suitable for on-site construction, while zinc insertion is more suitable for workshops and prefabrication because of the necessity of the heating process to melt the zinc sheet.

Moreover, given reports that zinc–steel bonding strength may eventually surpass that of epoxy [103], zinc insertion is expected to emerge as a more promising method for fatigue prevention in bridge applications.

Table 5-2 Comparison of the root crack prevention methods

Methods	Advantages	Disadvantages
Adhesive insertion (epoxy)	Strong bonding strength Stable bonding No additional heat treatment	Prone to welding defects Prone to bonding defects Short lifespan
Soft metal insertion (zinc)	Good welding quality Stable welding quality Long lifespan	Low bonding strength Unstable bonding Additional heat treatment required
Tap bolts [88]	Strong connection Long-last connection	Damage on original structure High machining accuracy required

5.6 Summary

In this chapter, the potential of bonding-assisted welding for application in actual bridges was examined through multi-scale finite element simulations. The optimization process was first carried out using a representative bridge, and the findings were subsequently generalized to four additional bridges to verify the broader applicability of the method. The key findings are as follows.

(1) The unfilled distance, defined as the distance between the bond layer and the weld root, plays a critical role in fatigue performance. A shorter distance provides more effective fatigue improvement and reduces stress within the bond layer. However, it must not be smaller than the minimum safety distance determined by material and process limitations. Based on the analyses, the recommended unfilled distance is 15–50 mm for epoxy insertion and 5–20 mm for zinc insertion.

(2) The thickness of the bond layer also influences performance. For epoxy insertion, increasing the thickness reduces the fatigue improvement effect on the weld root because of its relatively low stiffness. In contrast, zinc insertion is largely unaffected by thickness due to the closer stiffness match between zinc and steel. For both methods, however,

greater thickness helps decrease stress within the bond layer, thereby lowering the required bonding strength. Balancing these factors, a thickness of approximately 0.50 mm is recommended for practical applications.

(3) Bonding-assisted welding substantially reduces weld root fatigue stresses, leading to a significant extension of fatigue life. Numerical results for five representative bridges indicated reductions in weld root ENS ranging from 14% to 62%. Correspondingly, the predicted fatigue life increased by at least 1.5 times, and in many cases by more than 2.5 times, compared with conventional welding.

(4) Epoxy insertion is suitable for high-stress scenarios, utilizing the high bonding strength of epoxy to prevent bonding failure. In low-stress scenarios, zinc insertion is more suitable. Because zinc insertion produces better and more stable welding quality, it ensures that the original fatigue life of the structure is not adversely affected. In addition, epoxy insertion is more suitable for on-site construction, while zinc insertion is more suitable for workshops and prefabrication. Moreover, zinc has better durability than epoxy.

CHAPTER 6

CONCLUSIONS AND FUTURE RESEARCH

6.1 Conclusions

This dissertation carried out a series of experimental, theoretical, and numerical studies on a specific type of fatigue crack prone to initiation near steel bridge supports. This fatigue crack, referred to as Type D crack, originates at the root of the fillet welds between the sole plate and the bottom flange of the girders. Because the crack initiates internally and is surrounded by complex structural details, current detection and repair techniques are insufficient. To address this problem, this research proposed the bonding-assisted welding technology to improve fatigue resistance and suppress crack initiation at the source. The main conclusions are summarized below.

(1) Structural factors contributing to Type D cracks

Analysis of five actual bridges revealed that bridges with large girders or a high proportion of live loads are particularly susceptible to Type D cracks. Increasing fillet weld size or adopting bolt-weld hybrid connections can enhance weld root fatigue strength. However, these strategies face inherent limitations. Larger welds increase construction difficulty and cost. Bolts must be arranged close to the weld root to be effective, because the constraint range of bolts is limited. However, due to structural requirements, in many cases, the constraint range of the bolt cannot completely cover the weld root. As a result, structural adjustments alone cannot fully prevent Type D cracks. This limitation highlights the need for bonding-assisted welding technology, which supplements the bolt constraint range by restricting deformation around the weld root.

(2) Effectiveness of bonding-assisted welding

Both adhesive insertion and soft metal insertion demonstrated significant potential for preventing Type D cracks. Experiments with simplified specimens showed that adhesive insertion extended weld root fatigue life by more than three times, while soft metal insertion improved it by over ten times. Despite these promising results, each method has specific advantages and limitations, making them suitable for different application scenarios.

(3) Performance of adhesive insertion

Adhesive insertion provides strong and stable bonding, with a relatively simple construction process that does not require additional heat treatment. This makes it well-suited for bridges experiencing high stress levels and for on-site applications. However, adhesives have poor heat resistance and may burn during welding, leading to bonding and welding defects. These issues can be mitigated by placing heat-resistant rubber barriers of appropriate thickness and maintaining a safety distance of at least 15 mm between adhesive and welds. Furthermore, adhesive aging remains a concern, although the service environment is airtight. Based on parameter analyses of actual bridge bearings, an optimal configuration includes a safety distance of 15–50 mm and an adhesive thickness of approximately 0.5 mm.

(4) Performance of soft metal insertion

Soft metal insertion can avoid welding defects compared to adhesive insertion. By ensuring a safety distance of at least 5 mm, welds of comparable quality to conventional welding can be reliably achieved. Because the inserted metal is sealed within the joint, its durability is reliable. However, bonding requires melting the inserted metal sheet, which introduces practical constraints. For zinc, the process involves heating above 420 °C for several hours, limiting its feasibility to on-site construction. Thus, soft metal insertion is more suitable for workshop prefabrication or on-site construction of small components. Although this study observed lower and less stable zinc–steel bonding strength compared to adhesive–steel bonding, other studies reported high-strength zinc–steel bonds. This suggests that zinc insertion is a promising method. Based on bridge bearing analyses, the recommended parameters are a safety distance of 5–20 mm and a metal sheet thickness of 0.5 mm.

(5) Evaluation methods for fatigue performance

The ENS method is more suitable than the nominal stress method for evaluating weld root fatigue strength under bending–shear coupled loads, which corresponds to the conditions leading to Type D cracks. Additionally, the ENS method can evaluate the fatigue life improvement achieved by adhesive insertion. For soft metal insertion, however, its fatigue improvement mechanism involves a self-sacrificial process in which the zinc–steel bond fails before the weld root, making the elastic-based ENS method

insufficient for quantitative evaluation. Further experimental studies are therefore required to establish reliable evaluation methods for this case.

6.2 Ongoing and Future Research

This dissertation explored the potential of bonding-assisted welding technology for preventing weld root fatigue cracks; however, several limitations remain. To promote its application in engineering practice, the following areas require ongoing and future investigation.

(1) All tests and analyses in this study were conducted under the ideal assumption that epoxy does not undergo aging or deterioration. Since engineering structures are expected to perform over long service periods, future research will examine the long-term durability, degradation mechanisms, and potential performance losses of epoxy bonding-assisted welding.

(2) This study observed low bonding strength and high variability in zinc–steel bonding, although other studies have reported the possibility of achieving high-strength zinc–steel bonds. Further work is needed to establish reliable methods for producing stronger and more consistent bonding, which is critical for broader engineering applications.

(3) Most findings on practical applications were derived from simulation analyses. Although the modeling approach was validated using simplified experiments, large-scale testing under conditions closer to actual bridges is necessary to verify and refine these results for real-world use.

(4) The two bonding materials investigated, epoxy and zinc, each present distinct advantages and limitations, making them suitable for different application scenarios. Future research will explore new bonding materials, such as vitrimers and acrylic adhesives, that can simultaneously ensure high welding quality and strong, durable bonding performance.

(5) Due to the risk of bond failure, integrating sensors into the bond layer to monitor its bonding performance in real time will greatly improve the reliability of bonding-assisted welding technology.

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PUBLICATIONS

Journal Papers:

- [1] **Jiahao Mao**, Mikihiro Hirohata*, Yifei Xu, Kyong-Ho Chang. Fatigue Performance of Bonding-assisted Fillet Weld Roots by Inserting Adhesive Material, *Fatigue & Fracture of Engineering Materials & Structures*, Vol. 47, No. 10, pp. 3646–3657, 07, 2024. (Peer-reviewed)
- [2] **Jiahao Mao**, Feng Jiang, Mikihiro Hirohata*, Yanyue Qin. Numerical Analysis on Fatigue Performance in Fillet Weld Roots of Steel Bridge Bearings. *Civil Engineering Journal*. Vol. 11, No. 11, pp. 4800-4820, 11, 2025. (Peer-reviewed)
- [3] **Jiahao Mao**, Mikihiro Hirohata*, Feng Jiang, Daichi Hamasaka. Improving Fatigue Performance of Fillet Weld Roots Using Soft Metal Insertion. *Journal of Constructional Steel Research*, Vol. 236, Part A, pp. 110021, 01, 2026. (Peer-reviewed)

Conferences:

- [4] Yifei Xu, Mikihiro Hirohata*, **Jiahao Mao**. An Investigation on Fatigue Crack Prevention for Weld Root by Assistive Adhesive Bonding, 13th – Japanese–German Bridge Symposium, 08, 2023.
- [5] **Jiahao Mao**, Mikihiro Hirohata*, Yifei Xu. Verification of Prevention Effect for Fatigue Crack from Weld Root by Assistive Adhesive Bonding, National Meeting of JWS, Vol. 2024s, pp. 144–145, 06, 2024.
- [6] **Jiahao Mao**, Mikihiro Hirohata*, Daichi Hamasaka. Experimental Study on Fatigue Life Improvement of Fillet Weld Root by Soft Metal Insertion, National Meeting of JWS, Vol. 2024f, pp. 292–293, 12, 2024.
- [7] **Jiahao Mao**, Mikihiro Hirohata*. Numerical Analysis of Bonding-Assisted Technology for Preventing Weld Root Fatigue Cracks: A Case Study of a Real Bridge, IABSE Symposium Tokyo 2025, pp. 2466–2473, 05, 2025. (Peer-reviewed)

Patent:

- [8] Mikihiro Hirohata, **Jiahao Mao**, et al. An Improvement Method for Fatigue Performance of Weld Root on Sole Plates. (In Japanese, Under review)

ACKNOWLEDGEMENTS

In September 2022, I arrived in Japan from China and began my journey at the University of Osaka as a research student. A year later, in October 2023, I officially entered the doctoral program. Now, as I approach the completion of my doctorate, I reflect on more than three years of learning, growth, and discovery.

I owe my deepest gratitude to Professor Mikihiro Hirohata, my academic advisor, for his invaluable guidance, insightful advice, and constant encouragement throughout my doctoral studies. His expertise and patience have been instrumental in shaping my academic development. I am also sincerely grateful to Professor Toshiro Kamada and Professor Toru Inui for their constructive feedback, thoughtful discussions, and continuous support, all of which greatly enhanced the quality of this dissertation. I also wish to thank MM Bridge Co., Ltd. for supporting this research, especially Dr. Toshimitsu Suzuki and Mr. Hideaki Konishi.

My heartfelt thanks extend to all current and former members of the Structural Engineering Laboratory for fostering a stimulating and supportive research environment. I am grateful for their technical assistance, thoughtful suggestions, and many engaging discussions that helped refine my ideas. I am especially thankful for their kindness and help when I first arrived in Japan, which allowed me to quickly adapt to both the laboratory and life in a new country.

I am profoundly indebted to my family for their unconditional love, patience, and unwavering support throughout my academic journey. Their encouragement has been my greatest source of strength. Finally, I wish to express my heartfelt appreciation to all my friends who have supported me, both academically and personally, during these memorable years.

This dissertation would not have been possible without the guidance, assistance, and encouragement of all those mentioned above. To each of them, I extend my sincerest gratitude.

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October 2025