Electroslag Welding Applications for Steel Building Construction in Japan: A State-of-the-Art Review

Yukihiro Harada, Jun Iyama, Yuka Matsumoto, Kazuaki Suzuki, and Koji Oki

ABSTRACT

Electroslag welding (ESW) is advantageous for the improvement of the efficiency of welding thick steel plates, and the application of ESW has been gradually spreading to civil structures such as steel bridges and steel building structures. This paper presents a review of state-of-the-art ESW application for steel building structures in Japan. A brief history of the development of ESW technology, which involves the installation of interior diaphragm plates in welded built-up box columns for high-rise steel buildings, is first introduced, followed by a recent recommendation for the design and fabrication of ESW joints to prevent brittle fracture under earthquake loading.

Keywords: electroslag welding, built-up box column, interior diaphragm, brittle fracture, seismic application.

INTRODUCTION

Electroslag welding (ESW) was originally developed in the Soviet Union in 1951, initially as an efficient vertical welding method. It was developed to complete the joining process by simultaneously melting and solidifying the solid electrode wire and base metal in the molten slag by enclosing a large weld pool with a water-cooled copper sliding plate (Yamaguchi, 1997; Chambers and Manning, 2016), as shown in Figure 1. ESW could significantly improve the efficiency of welding thick steel plates; thus, this welding method was first implemented as a welding technology in shipbuilding in response to a demand for improved productivity and economic efficiency.

Yukihiro Harada, Professor, Department of Architecture, Faculty of Engineering, Chiba University, Chiba, Japan. E-mail: yharada@faculty.chiba-u.jp (corresponding)

Jun Iyama, Associate Professor, Department of Architecture, Faculty of Engineering, University of Tokyo, Tokyo, Japan. E-mail: iyama@arch1.t.u-tokyo.ac.jp

Yuka Matsumoto, Professor, Department of Architecture and Building Science, College of Urban Sciences, Yokohama National University, Yokohama, Kanagawa, Japan. E-mail: matsumoto-yuka-kg@ynu.ac.jp

Kazuaki Suzuki, Architectural Committee, Japan Iron and Steel Federation, Nihonbashi Kayabacho, Tokyo, Japan (Nippon Steel Corporation). E-mail: suzuki.953.kazuaki@jp.nipponsteel.com

Koji Oki, Architectural Committee, Japan Iron and Steel Federation, Nihonbashi Kayabacho, Tokyo, Japan (JFE Steel Corporation). E-mail: k-oki@jfe-steel.co.jp

Paper No. 2022-10

The ESW method has various advantages over general arc welding. First, it can efficiently weld a wide range of plate thicknesses in a single pass, resulting in less heat distortion than that of arc welding with multiple passes. Second, because arc heat is not used as a heat source, spatter generation is reduced, and shielding gas supply is not required, resulting in superior welding workability. Third, it has a wide margin of accuracy for groove processing and can be welded with a square groove without gas cutting; therefore, the bevel preparation is simple. However, owing to the high welding heat input, the ESW has a disadvantage that the crystal grains of the weld metal and heat affected

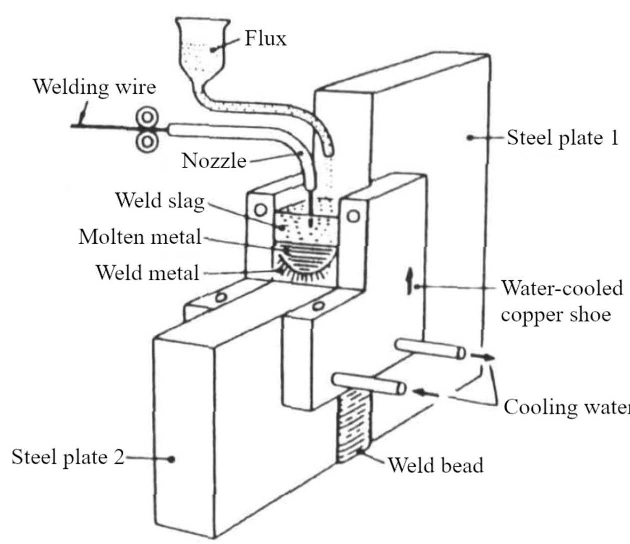


Fig. 1. Schematic of the earliest ESW (adapted from Yamaguchi, 1997).

Table 1. Timeline of ESW Development for Building Structures in Japan	
1951	ESW was developed in the Soviet Union
1965	First ESW application to shipbuilding at Kawasaki Heavy Industry in Japan
1968	Kasumigaseki Building completed; the first full-fledged application of ESW to steel building structures in Japan
1970–1971	Two-sided ESW for weld box column was developed
1971	Four-sided ESW for weld box column was developed
1971	“Specification of Consumable ESW” published by Architectural Institute of Japan
1990s	Simplified nonconsumable ESW was developed
1995	Hyogo-ken Nanbu Earthquake
2016	“Guidebook for Prevention of the Occurrence of Brittle Fractures of Inner Diaphragm Plate Electroslag Welds” published by Japanese Society of Steel Construction

zone (HAZ) tend to be coarsened, making it difficult to obtain high toughness, which may be a serious problem in steel structures, particularly in seismic applications.

In the United States, ESW applications have spread to civil steel structures due to their high welding efficiency. The application of ESW for steel structures began in steel bridges (Chambers and Manning, 2016). However, problems with ESW joints were found in several steel bridges in the 1970s, and thus ESW has not been widely used in steel bridges. The situation is similar for steel building structures. There have been limited applications of ESW for steel building structures in the United States (e.g., Agic and Hampton, 1968). Recently, the ESW has been successfully applied to weld gusset plates to embedded panels and box columns to connect seismic buckling braces at the Wilshire Hotel in downtown Los Angeles, California (Lee et al., 2014).

In contrast, in Japan, using ESW to install interior diaphragm plates in welded built-up box columns for the construction of high-rise steel buildings is a common practice. This paper aims to present state-of-the-art research and development on ESW applications in steel building structures in Japan. A history of ESW technology development is first presented in Table 1, and then the essence of a design recommendation on ESW welding of interior diaphragm plates, which was recently published based on a series of research coordinated by the Japan Iron and Steel Federation, is introduced. Welding thicker steel plates by using technologies like ESW is expected to increase, particularly in earthquake-prone countries, as more tall buildings are being constructed. Furthermore, experience and knowledge gained on ESW in Japan over the years is expected to benefit for the future development of high-rise building structures worldwide.

DEVELOPMENT OF ESW FOR STEEL BUILDING STRUCTURES IN JAPAN

The Beginning of ESW Application to Building Steel

In 1964, a simplified ESW method with a consumable nozzle and fixed cooling plates was developed and implemented by several Japanese companies, as shown in Figure 2 (Ishiguro et al., 1975), which was applied to the outer panel of a ship for the first time (Yamaguchi, 1997). The consumable nozzle melts and gets consumed as the welding progresses; thus, the nozzle lift mechanism that is used with the conventional consumable ESW, as shown in Figure 1, is unnecessary. The disadvantage of this method is that the weld length is limited to the length of the consumable nozzle.

The simplified consumable ESW was first used in 1965 for the construction of steel building beam-to-column connections; the Kintetsu Terminal Building in Nagoya was one of the earliest applications (Sakurai and Takahara, 1966). Thereafter, the efficiency of welding thick steel plates for building applications became a major concern. At that time, concerns were raised regarding the decrease in the toughness of heat affected steel materials around ESW joints due to high heat input (Yamaguchi, 1997).

In the Kasumigaseki Building in Tokyo, which was completed in 1968 with the height of 147 m and is recognized as the first high-rise building in Japan, ESW was adopted in earnest to build the steel frameworks. Thick I-beams were used for both columns and beams. The beam-to-column connection was a split-tee type, and a consumable nozzle type ESW was adopted to efficiently weld the beam flange and the top plate of the tee, as shown in Figure 3 (Ueno and Ariyasu, 1968; Matsumoto, 2000).

In the 1960s, wide-flange (H-section) shapes were widely used for columns in high-rise steel buildings because wide-flange steel was less expensive than a box section. However, structural designers considered the box section to be structurally more efficient than the wide-flange shape. During that era, two fabrication procedures for built-up box columns were used. Figure 4(a) shows one approach in which each diaphragm plate had to be extended beyond one face of the column; the flange plate of the column at that face also had to be made discontinuous first and to be welded later. Figure 4(b) shows the second approach, in which a beam-to-column connection assembly was inserted between two column segments. Both approaches were labor intensive and expensive—and therefore not widely adopted.

This situation changed after ESW was introduced to weld interior diaphragms in built-up box columns in the 1960s, which significantly reduced the fabrication cost, especially for columns with thicker plates. With this method, face-plates of the column could be made continuous without the need to cut and re-weld. Since then, shop-welded built-up box columns with ESW have been widely used for high-rise steel building construction in Japan.

In the earliest application of ESW, three sides of a box column were first assembled in a U-shape as shown in

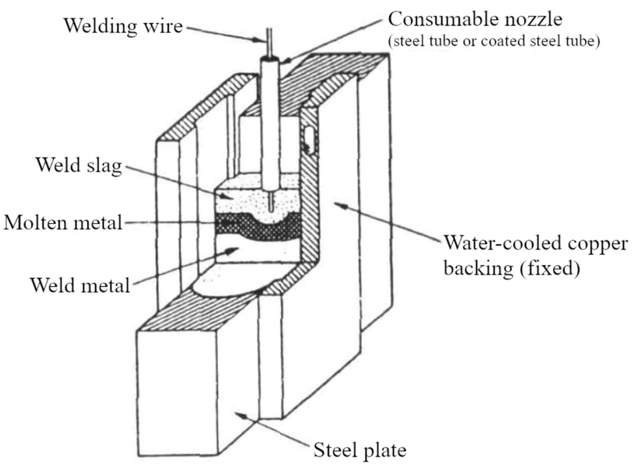


Fig. 2. Schematic of simplified consumable ESW (adapted from Yamaguchi, 1997).

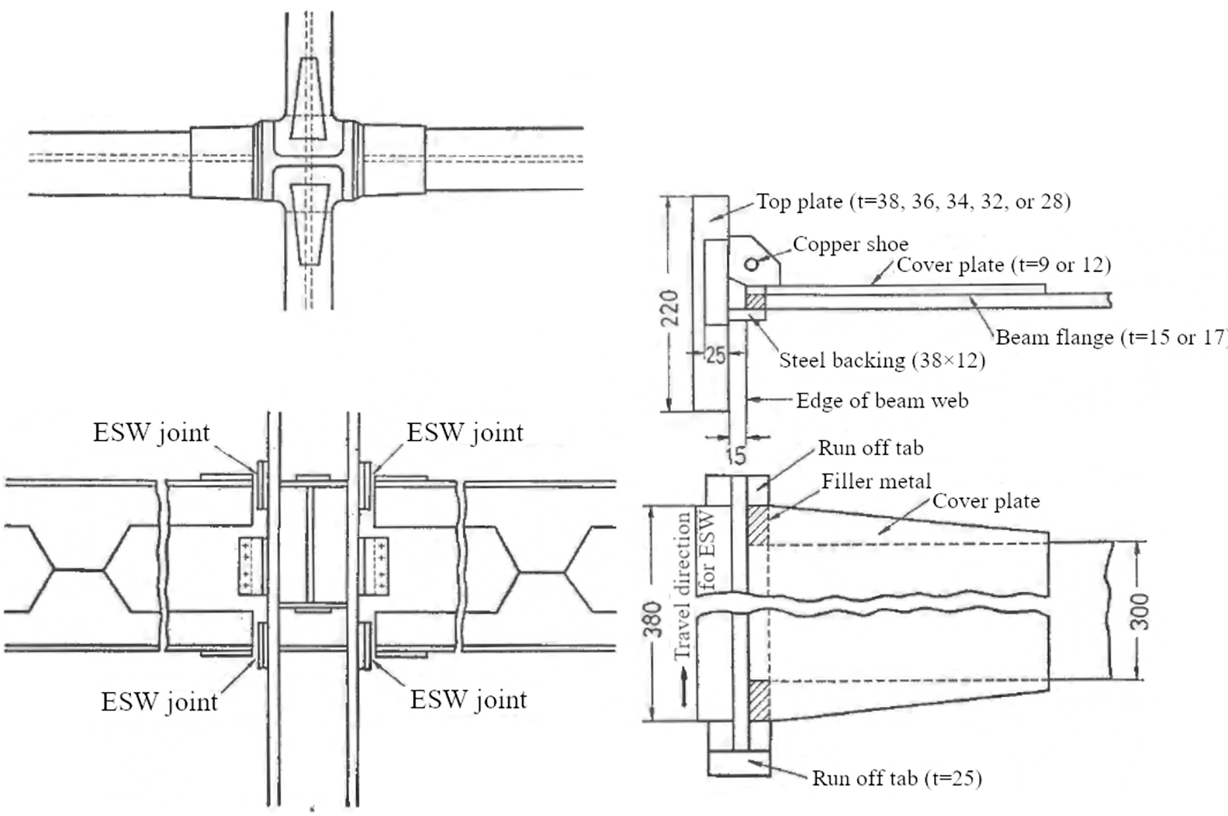


Fig. 3. ESW applications in Kasumigaseki Building (unit: mm) (adapted from Ueno and Ariyasu, 1968).

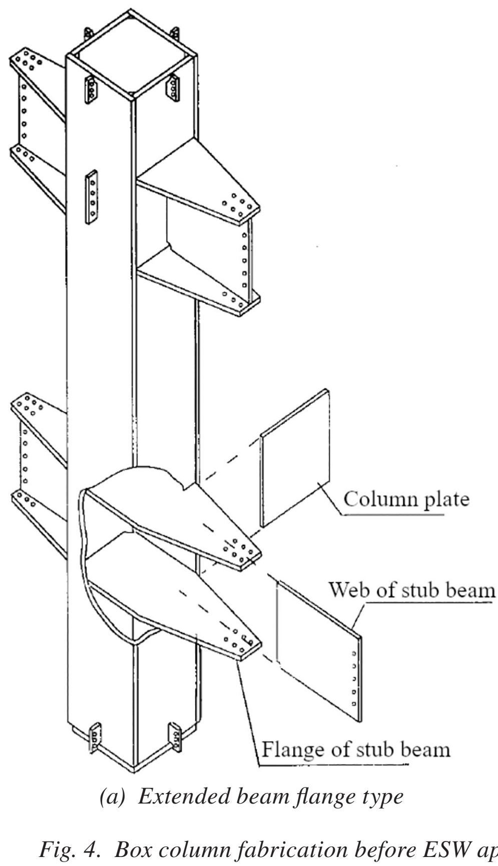
Figure 5(a). After that, three sides of the interior diaphragm plate were manually welded to the column plates. In this method, the column plates distorted by the weld heat, and the distorted column faceplates had to be straightened. After enclosing the box column with the remaining face-plate, the fourth side of the interior diaphragm plate, which was enclosed within the box column, was welded with the consumable ESW as shown in Figures 5(b) and (c) (Japan Welding Engineering Society, 2004). Since welding with ESW introduced a large amount of heat input, a considerable amount of effort was required again to straighten the distorted face plates.

A two-sided ESW method was developed in 1970 to solve these problems. In this fabrication method, the column was assembled in a U-shape, and only two sides of the diaphragm plate were manually welded with semi-automatic CO_2-gas shielded metal arc welding. With the enclosing plate in position, then ESW was applied to complete the remaining two welds of the diaphragm plate (Japan Welding Engineering Society, 2004). After the two-sided ESW method was introduced for several years, a four-sided ESW method was developed in 1971, as shown in Figure 6. In response to this trend in ESW applications to steel building structures, the Architectural Institute of Japan (AIJ) published the welding standard for consumable ESW in 1971 (AIJ, 1971). In the 1980s when the labor cost became more expensive, fabricators were motivated to automate steel fabrication and preferred the four-sided ESW method. Since then, the four-sided ESW method has been a standard practice for welding interior diaphragm plates in built-up box columns.

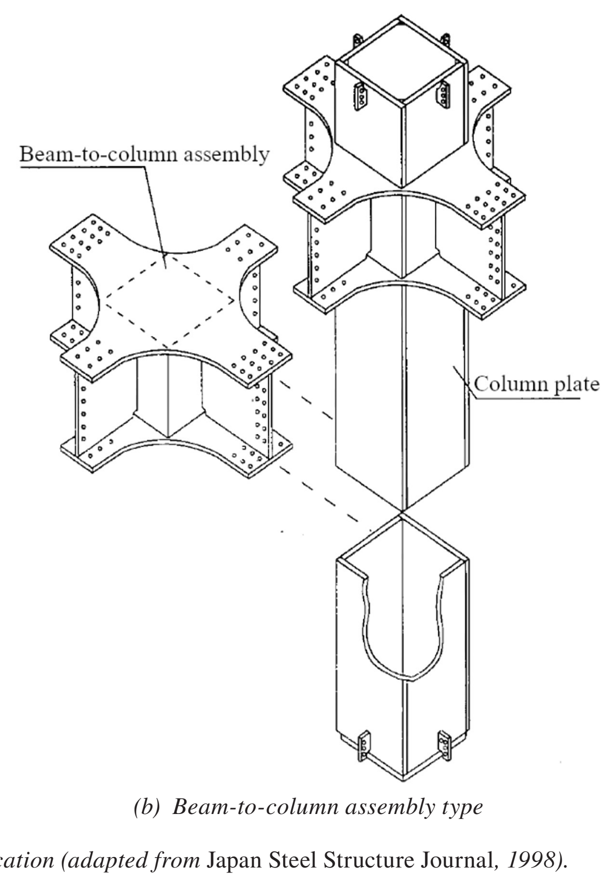
Development of Simplified Nonconsumable ESW

Because the consumable ESW had been widely applied in fabricating built-up box columns for high-rise construction, the disadvantage of the consumable ESW had become recognized. The high heat input [several hundred thousand Joules per centimeter (several hundred thousand Joules per in.)] due to the low travel speed (less than several centimeters per minute) produced a HAZ with mechanical properties significantly deteriorated (Wada et al., 1990).

One of the solutions to avoid these defects was to develop a highly efficient welding method with a reduced heat input. A simplified nonconsumable ESW method with an elevating nozzle was developed by Nippon Steel Corporation (Nippon Steel Welding & Engineering, 2019) to address the problem in 1980s, as shown in Figure 7. The welding process is called by its trade name (“simplified electroslag



(a) Extended beam flange type



(b) Beam-to-column assembly type

Fig. 4. Box column fabrication before ESW application (adapted from Japan Steel Structure Journal*, 1998).*

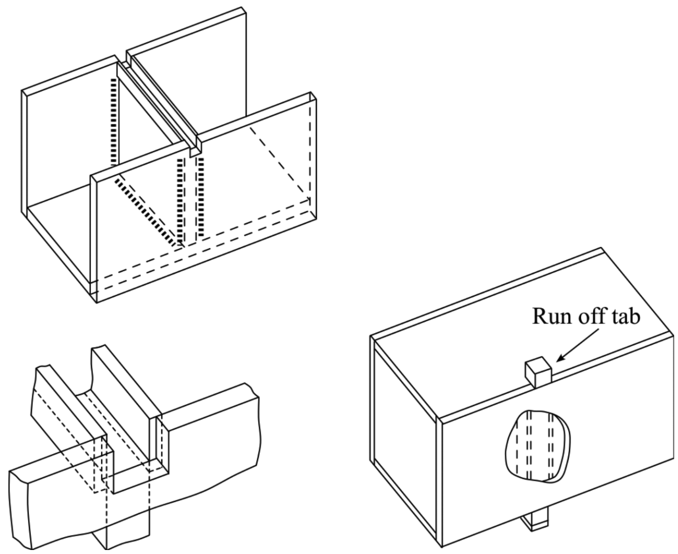


Fig. 5. First application of ESW for interior diaphragms of box columns (adapted from Japan Welding Engineering Society, 2004).

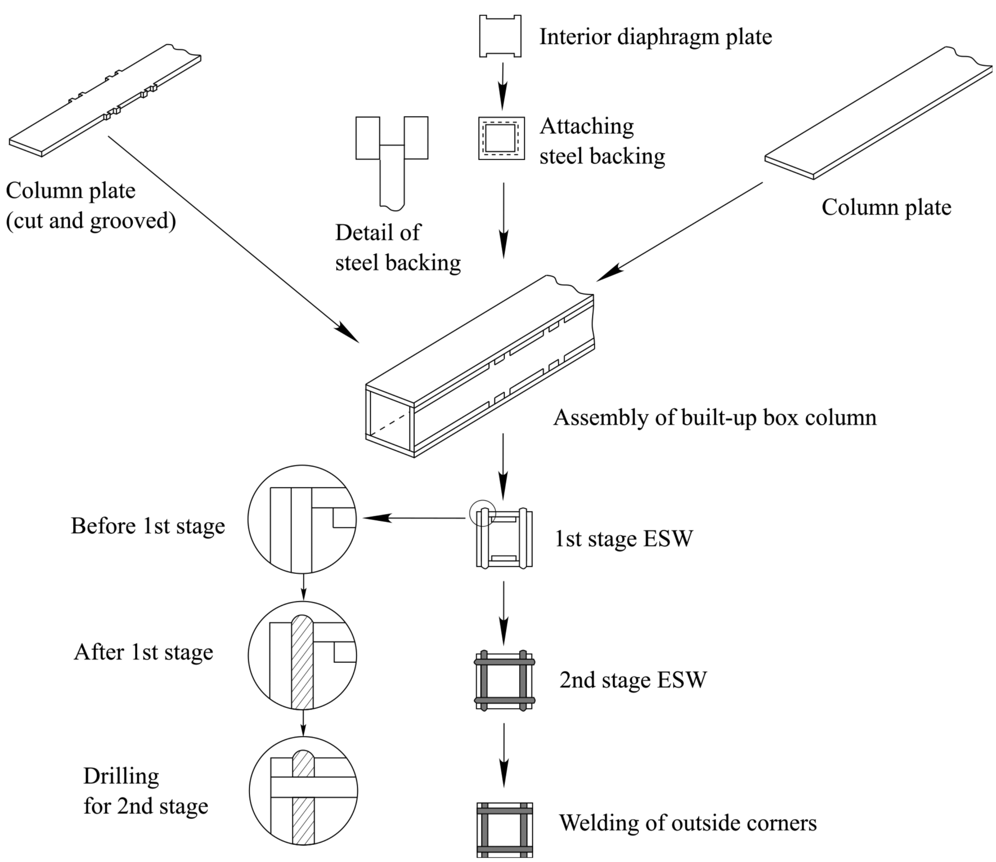


Fig. 6. Four-sided ESW method in current practices (adapted from Japan Welding Engineering Society, 2004).

welding process with nonconsumable elevating tip") and has been applied to almost all diaphragm welding of box column in Japan.

The welding wire in this process is fed into the welding slag in a groove through a nonconsumable nozzle at an electrode extension that depends on the welding current. A high travel speed is possible with a thin weld wire of 1.6 mm and a longer electrode extension. The welding speed was approximately doubled, and the heat input was reduced by about a half. The simplified nonconsumable ESW is also economical because it does not require consumable nozzles and the cooling plates were omitted. The heat source can be increased to obtain sufficient melting by oscillating the nozzle in the direction of the diaphragm thickness. With the simplified ESW, almost the same welding conditions can be used regardless of the diaphragm thickness, making the operation more effortless. The nozzle moves upward automatically as the welding progresses, allowing almost unsupervised welding after welding initiation. Therefore, multiple ESW units can be operated simultaneously by a single person.

RECOMMENDATIONS FOR BRITTLE FRACTURE PREVENTION OF INTERIOR DIAPHRAGM ELECTROSLAG WELDS

Background of the Publication of the Recommendation

The Hyogo-ken Nanbu Earthquake in 1995 caused significant damage to steel structures, and this event was recognized as the first one in which many brittle fracture damages were observed in beam-to-column connections in steel building structures in Japan. The typical fracture involved a crack initiating from the beam flange weld in through-diaphragm-type beam-to-column connections, as shown in Figure 8(a). No damage was reported on the ESW joint in welded box columns, as shown in Figure 8(b).

A series of dynamic loading tests on full-scale beam-to-column subassemblages was conducted after the earthquake, and fracture at the ESW part in the interior diaphragm of built-up box columns was observed (Akiyama et al., 2002); this failure mode was similar to the fracture observed in the through-diaphragm-type beam-to-column connections. In the beam-to-built-up box column connection, a brittle

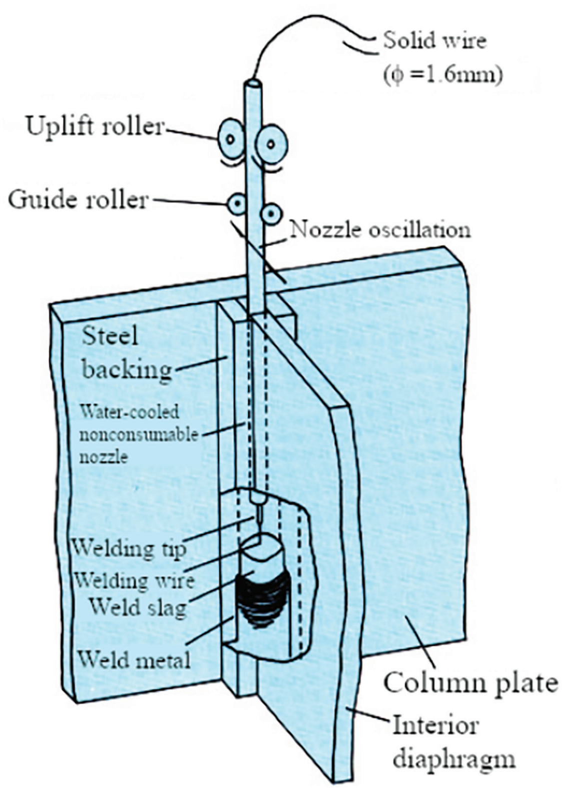
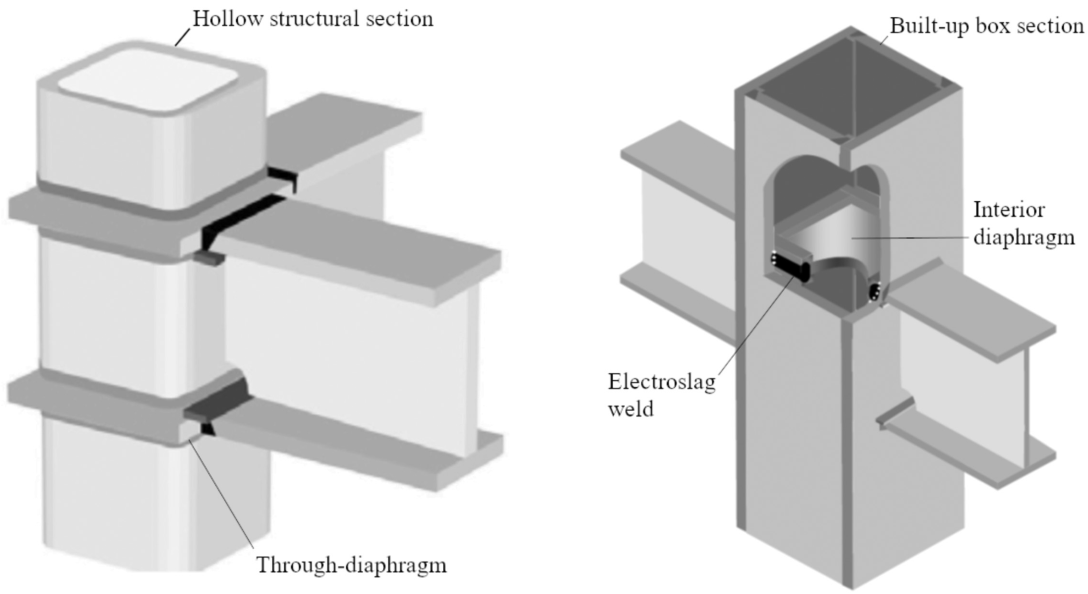


Fig. 7. Simplified nonconsumable ESW (adapted from Nippon Steel Welding & Engineering, 2019).

fracture initiating from the slit tip—that is, the tip of the slit, which is the gap between the column face plate and a steel backing—in the ESW part was observed, as shown in Figures 9 and 10. This was the first reported brittle fracture case of an ESW joint for steel building construction in Japan. This failure mode drew a strong interest from the engineers and researchers involved in high-rise steel building construction because there had been concerns about the decrease in the toughness of heat affected steel materials around the ESW due to high heat input presented earlier.

A series of coordinated research on fracture prevention of the ESW joint was then conducted in Japan starting in the early 2000s (Shimanuki et al., 2003). For the column



(a) Through-diaphragm-type beam-to-column joint *(b) Interior diaphragm type*

Fig. 8. Types of beam-box column joints (adapted from JSSC, 2016b).

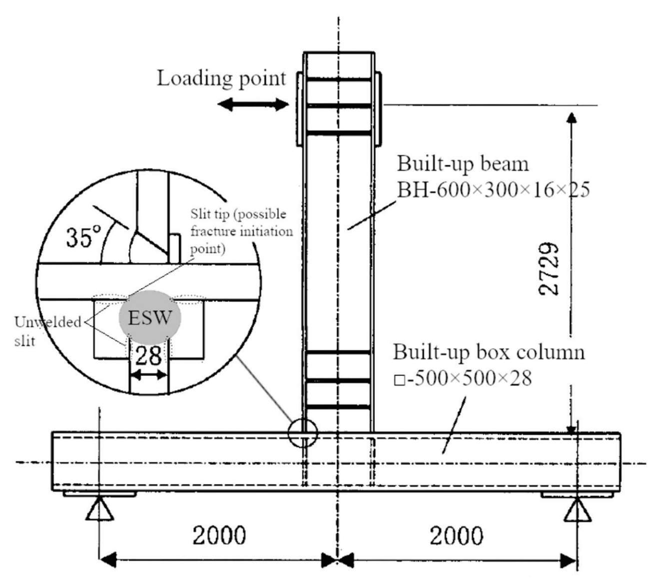


Fig. 9. Example of subassemblage specimen with fractured ESW at the interior diaphragm (unit: mm) (adapted from Akiyama et al., 2002).

plates, the studies focused on a commonly used 490 N/mm^2 (71 ksi) class steel (by ultimate strength) in the SN grade steel, which is specified as rolled steels for building structure in Japanese Industrial Standards. Both loading tests and numerical simulations were conducted to determine the failure mechanism in the ESW joint and identify conditions under which brittle fracture would occur. Two types of loading tests were conducted to reproduce the fracture of the ESW joint: biaxial tensile tests of ESW joints, as shown in Figure 11, and the beam-to-column subassemblage tests with built-up box columns incorporating interior diaphragms welded by ESW. This coordinated research obtained considerable technical data and findings on the toughness of ESW welds required to avoid fracture. In 2016, the Japanese Society of Steel Construction (JSSC) published a recommendation entitled *Guidebook for Prevention of the Occurrence of Brittle Fractures of Inner Diaphragm Plate Electroslag Welds* (JSSC, 2016a), hereafter referred to as the *Guidebook*, to share the findings and provide recommendations for the three parties involved in steel building construction: structural designers, steel fabricators, and construction engineers.

Fundamental Idea of Fracture Prevention of ESW Joint at the Interior Diaphragm

Shimokawa et al. (2019) summarized the experimental results of a series of loading tests. Figure 12 shows an example of the observations from the testing of three beam-to-built-up box column subassemblage specimens. In the example, if the Charpy V-notch (CVN) value, E_v, at the HAZ, fusion zone (FL), or weld metal (WM) of the ESW joint was lower, brittle fracture would occur, and both the

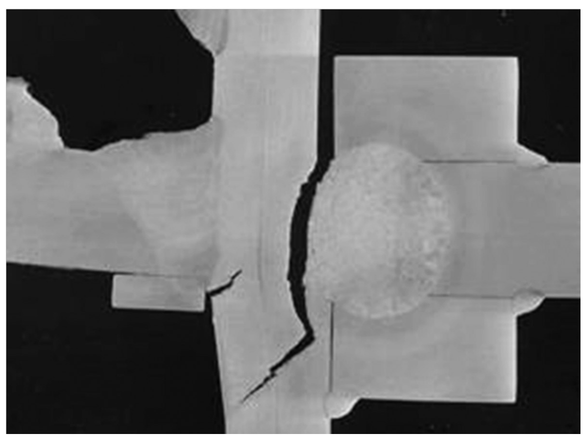


Fig. 10. Example of ESW joint fracture in beam-to-box column subassemlage loading tests (JSSC, 2016a).

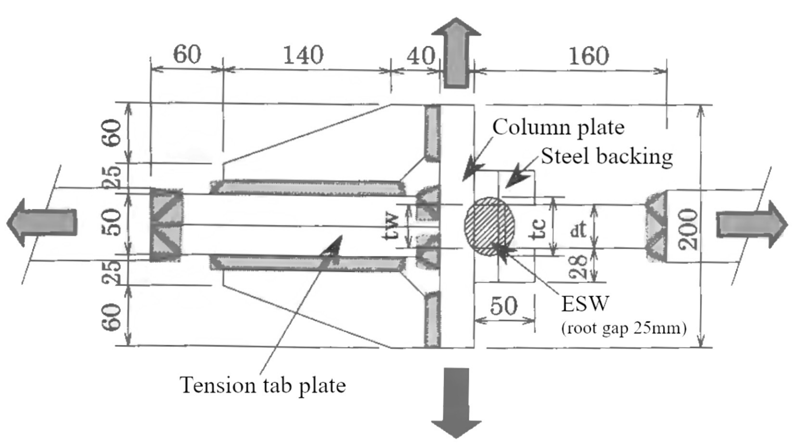
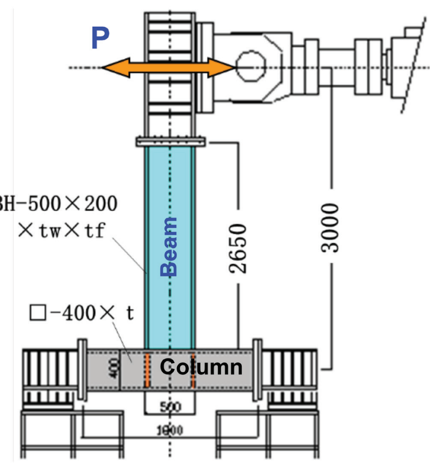


Fig. 11. Biaxial tensile test of ESW joint (unit: mm) (adapted from JSSC, 2016a).

ultimate strength and ductility were smaller (the sampling positions of CVN specimens for the HAZ, FL, and WM correspond to "HAZ1," "Bond," and "Depo1," respectively, in the later section, Weld Testing Method for ESW at the Interior Diaphragm). In Specimens H-2 [the smallest E_v = 14 J (11 ft-lb) at the fusion zone] and H-3 [the smallest E_v = 14 J (11 ft-lb) at the weld metal], a crack initiated at the slit tip and propagated from the interface between the ESW joint and the HAZ, and then propagated to the column plate due to the tensile force acting perpendicular to the HAZ at the slit tip, as shown in Figures 12(c) and (d). In contrast, Specimen H-1 (the smallest E_v = 69 J (51 ft-lb) at the weld metal) showed a ductile fracture at the interior diaphragm, not at the ESW joint, as shown in Figure 12(b). Based on the results from a series of experiments, the following findings were confirmed:

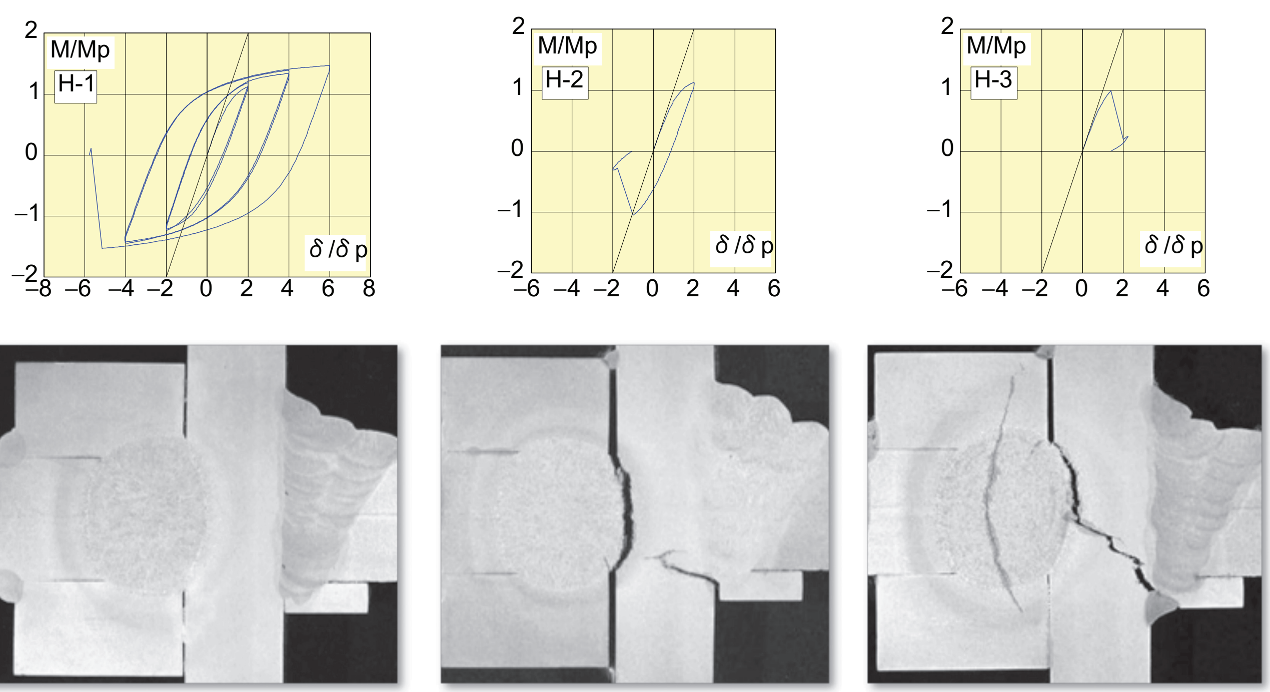


No.	Column		Beam section	Test temp.
	Section	Steel		
H-1	□-400×28	High toughness SN490C	SN490B BH-500×200×22×40	−20°C
H-2				−40°C
H-3				−40°C

No.	Column	Test temp.	HAZ		F.L.		W.M.	
			E_v (J)	B (%)	E_v (J)	B (%)	E_v (J)	B (%)
H-1	High toughness SN490C	−20°C	74	72	75	77	69	73
H-2		−40°C	29	91	14	97	44	82
H-3		−40°C	19	94	18	93	14	92

(B: percent brittle fracture)

(a) Outline of the subassemblage tests



(b) Specimen H-1 *(c) Specimen H-2* *(d) Specimen H-3*

Fig. 12. Example results of beam-to-built-up box column subassemblage loading tests (adapted from Shimokawa et al., 2019).

1. Brittle fracture was caused by the crack initiation and propagation from the slit tip at the ESW joint.
2. Brittle fracture did not occur in specimens with CVN toughness at the designated zone where brittle fracture could initiate exceeding 47 J at 0°C (35 ft-lb at 32°F).
3. A positive correlation existed between the nominal tensile stress acting on the ESW joint and the toughness at the zone when the brittle fracture would occur.

Based on the findings of the loading test results, it is essential to limit the applied tensile stress acting on the interior diaphragm to prevent brittle fracture of ESW joint. Figure 13 shows a schematic of the tensile forces acting on the ESW joint. The tensile force acting on the interior diaphragm, P_d, is determined from horizontal equilibrium with the tensile forces P_f and P_w acting on the beam flange and web, respectively. The driving force P_{req} to tear the slit tip in the HAZ of the ESW joint is dominated mainly by P_d, combined with the tensile forces P_c acting on the column plate. The required driving force P_{req} to initiate brittle fracture at the slit tip is a function of the toughness near the slit tip. Thus, limiting P_d (or the nominal tensile stress σ_d at the critical cross-section of the ESW joint) to a sufficiently small value is essential to prevent brittle fracture. An evaluation procedure to determine whether fracture will occur at the ESW joint is provided in the *Guidebook*, as shown in Figure 14. The fundamental idea is that the nominal tensile stress, σ_d, at the ESW joint should be limited to a threshold value, which is dependent on the CVN toughness of the ESW joint. The procedure to determine the CVN toughness is also provided in the *Guidebook*.

The nominal tensile stress limit for the interior diaphragm was established from the biaxial tensile test results and the subassemblage loading test results. Figure 15 shows the relationship between the ultimate strength of the ESW joints and the CVN toughness (at the zone where the brittle fracture initiated in the ESW joint). Each data point in the figure corresponds to a single test in which the brittle fracture occurred at the ESW joint. In this figure, the ultimate strengths of the joints P_u were normalized by the nominal yield strengths of the ESW joint, P_{dy}. The nominal yield strength P_{dy} is defined as the product of the yield stress of the interior diaphragm and the effective cross-sectional area of the ESW joint, considering the penetration of ESW and the spread of the tensile stress as shown in Figure 16(a); the penetration depth notably affects the strength and deformation capacity of the subassemblage with ESW joints (Umeda et al., 2022). For the results of beam-to-column subassemblage tests, P_u was computed from Figure 16(b) as follows:

$$P_u = \frac{M_{cf}}{H_b - 2_{bf}t} \tag{1}$$

and thus the corresponding nominal tensile stress is

$$\sigma_d = \frac{P_u}{t_w(B_b + 2t_s)} = \frac{M_{cf}}{(H_b - 2t_{bf})t_w(B_b + 2t_s)} \tag{2}$$

where M_{cf} is the design maximum bending moment at the beam-end, H_b is the height of the beam, B_b is the width of the beam flange, t_{bf} is the thickness of the beam flange, t_w is the width of the ESW joint at the interior surface of the column plate surface, and t_s is the thickness of the column plate.

In Figure 15, it can be seen that the fracture resistance increases as the CVN value increases, although the scatter of fracture resistance is considerable. From these test results, a required CVN toughness to prevent the fracture at the ESW joint can be derived. In the figure, the lower bound of the E_v–P_u/P_{dy} relationship is roughly represented by a bilinear curve consisting of two parts: an increasing part for smaller E_v and the plateau at $P_u/P_{dy} = 1$ for larger E_v. The boundary between the two parts is $E_v = 47$ J (35 ft-lb), which means that the ESW joint will be expected to fully exert its yield strength if E_v exceeds 47 J (35 ft-lb). In contrast, if E_v is smaller than 47 J (35 ft-lb), the ESW joint will not bear tensile strength corresponding to the yield tensile stress; thus, the maximum nominal tensile stress will decrease for smaller E_v. The bilinear relationship therefore gives a required CVN toughness to prevent the fracture corresponding to P_u/P_{dy}.

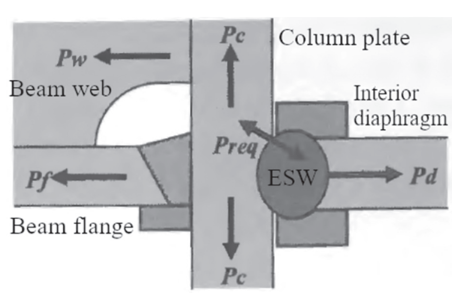


Fig. 13. Tensile forces acting at the ESW joint (adapted from JSSC, 2016a).

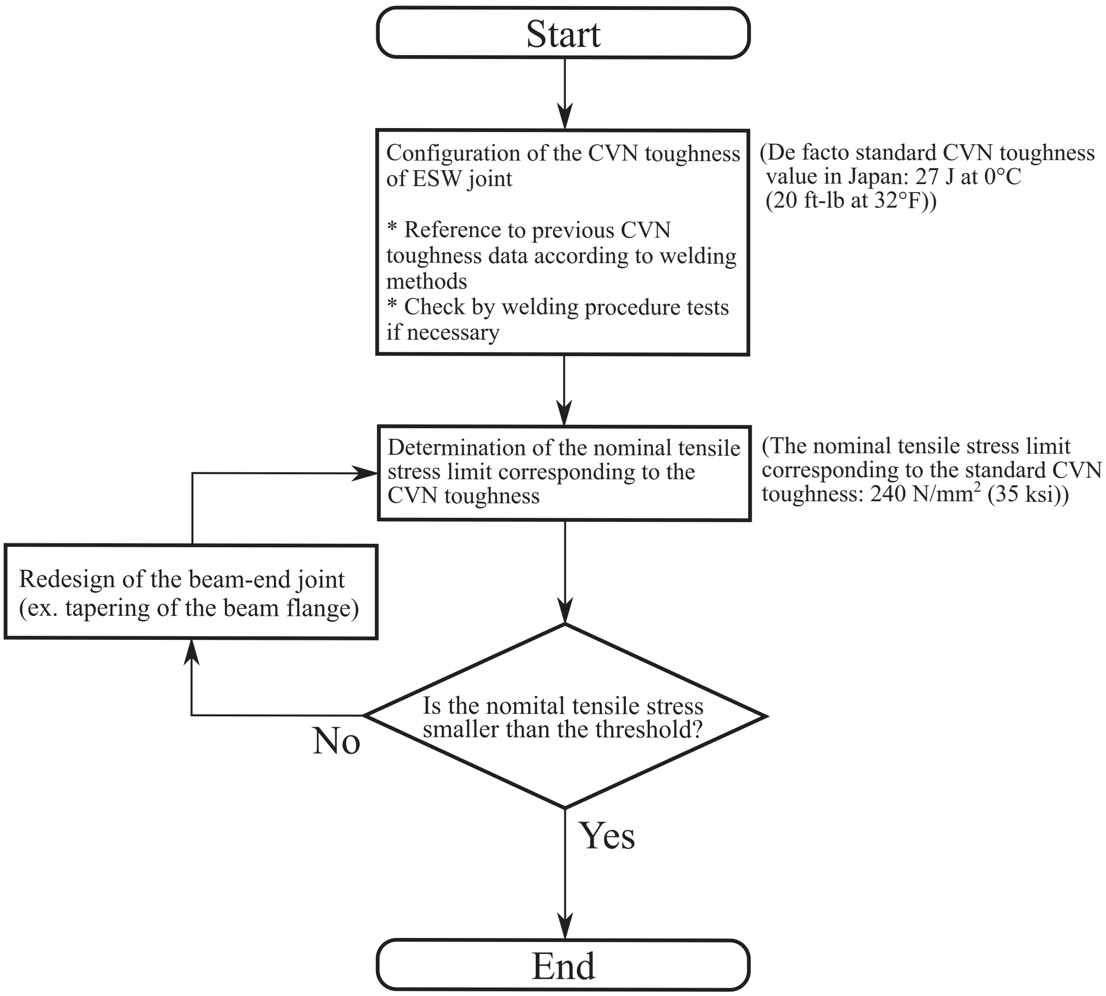


Fig. 14. Procedure of fracture prevention design of ESW joints at interior diaphragms (adapted from JSSC, 2016a).

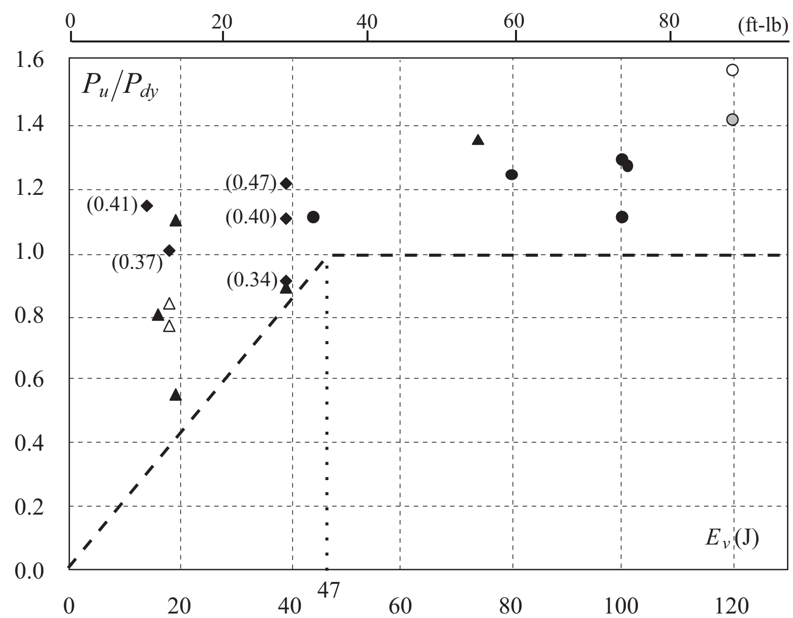
In the *Guidebook*, the limiting values for the nominal tensile stress are listed in Table 2 as a simpler representation of the bilinear curve in Figure 15. As shown in the table, the toughness level of 15 J (11 ft-lb) is specified as the minimum requirement, whereas the next level of 27 J (20 ft-lb) is considered the de facto standard toughness level in Japan. The highest level of 47 J (35 ft-lb) is specified as the required toughness for the interior diaphragm to attain its yield stress without experiencing brittle fracture at the ESW joint. These criteria are also included in a recent manual for standard mechanical tests of welds in steel building construction in Japan (JSSC, 2016b).

Note that the test results in Figure 15 also include the information on the tensile stress of the column plate as a tensile stress ratio, which is defined by the ratio of the nominal tensile stress due to axial force and bending moment on the column member to the yield stress. A higher axial tensile stress in the column plate would influence the occurrence of brittle fracture of the ESW joint, but the criteria in Table 2 are independent from the tensile stress ratio. To explicitly consider the effect of the tensile stress, the detailed assessment procedure to be presented in the next section can be used.

Detailed Assessment Scheme of the Possibility of ESW Fracture

The *Guidebook* also introduced a detailed assessment method to evaluate the risk of brittle fracture at the ESW joint. Because the ESW joint fracture is characterized by the fracture starting from the slit tip between the steel backing and column plate, it is necessary to consider the effect of local stress concentration—that is, to evaluate whether the fracture can be prevented by the fracture mechanics approach. In this detailed method, the concept is to limit the local tensile stress acting on the slit tip to be sufficiently smaller, depending on the toughness of the ESW joint. Such local approach would be necessary if the tensile stress acting on the column plate is high and a detailed check of the risk of fracture of ESW joints is required. The application of the procedure requires a deal of effort; refer to the *Guidebook* and the references in the *Guidebook* for more information. This method is versatile as it can be applied to steel other than the 490 N/mm^2 (71 ksi) class steel.

In the detailed assessment, the risk of fracture of the ESW joint is assessed using original indices γ_{req} and γ_{lim} related to the maximum principal stress at the slit tip between the interior diaphragm and the column plate. As imagined from

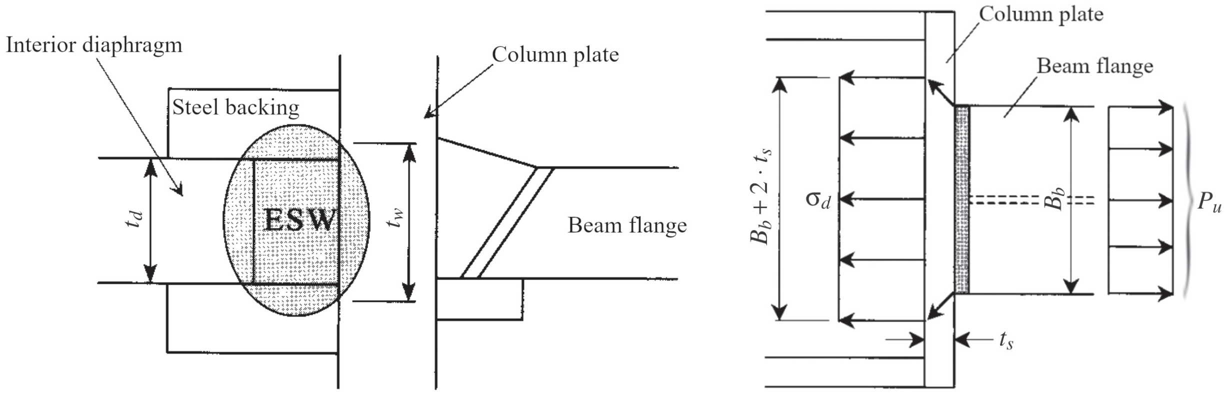


Location of crack initiation	Biaxial tensile test Tensile stress ratio 0	0.3	0.6	Subassemblage test
Deposited metal	○	● (gray)	●	-
HAZ	△	-	▲	(#)◆

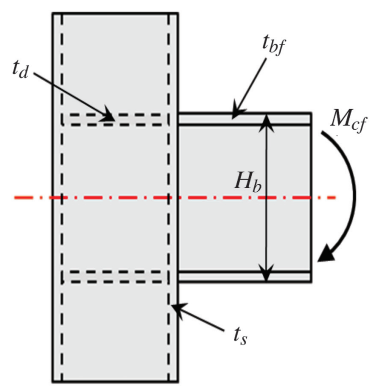
(#) : The value in parentheses shows the tensile stress ratio corresponding to the nominal tensile stress due to bending moment at the experimental ultimate strength

Fig. 15. Maximum tensile resistance versus Charpy impact energy (adapted from JSSC, 2016a).

Table 2. Criteria for the Upper Limits of Tensile Stress Acting on the Interior Diaphragm (adapted from JSSC, 2016a)			
Toughness level of the electroslag welds	15 J (11 ft-lb) or higher	27J (20 ft-lb) or higher	47 J (35 ft-lb) or higher
Criteria for the upper limits of acting stress	160 N/mm^2 (24 ksi) or less (0.5*F*)	240 N/mm^2 (35 ksi) or less (0.75*F*)	325 N/mm^2 (48 ksi) or less (1.0*F*)
F: Nominal yield stress for 490 N/mm^2 (71 ksi) class steel [= 325 N/mm^2 (48 ksi)]			



(a) Definition of effective cross-sectional area (adapted from JSSC, 2016a)



(b) Definition of beam-to-column subassembly dimensions

Fig. 16. Maximum nominal tensile stress in the interior diaphragm.

Figure 13, the existence of a high tensile stress in the column plate increases the magnitude of the principal stress at the slit tip, and thus the fracture resistance decreases. In addition, the higher tensile stress in the column plate would change the direction of the principal stress to make it aligned orthogonally to the direction of the slit. Thus, the driving force is more likely to initiate the opening of the crack at the slit tip.

Figure 17 shows the detailed assessment method. The required joint strength index γ_{req} is compared with the critical joint strength index γ_{lim}. If the condition $\gamma_{req} < \gamma_{lim}$ is satisfied, then the ESW joint is not expected to fracture. Otherwise, the joint detail or the design toughness of the ESW should be reconfigured; the assessment will be repeated until the condition is satisfied.

To determine the required joint strength index γ_{req}, the nominal tensile stress σ_d acting on the interior diaphragm was first calculated by Equation 2 presented in the previous section. The principal stress σ_{mreq} in the HAZ (at the slit tip in the ESW joint) corresponding to the calculated σ_d is then estimated through the σ_d–σ_m conversion curves, which were predetermined by finite element analyses, as shown in Figure 18. The σ_d–σ_m curves depend on the tensile stress ratio of the column plate, as shown in Figure 19. In the *Guidebook*, the σ_d/σ_u–σ_m/σ_u curves are given for the tensile stress ratio lower than or equal to 0.6. (In the Commentary in the *Guidebook*, the σ_d/σ_u–σ_m/σ_u curve for higher tensile stress ratio larger than 0.6 is also given, which was obtained by extrapolating the FEA results for lower stress ratios. However, the curve is not supported by experimental data.)

To determine limiting joint strength index γ_{lim}, the CVN value at a prescribed temperature T required in the design is first set. Kayamori et al. (2007) derived the correlation equation between the CVN value at HAZ of ESW joints and

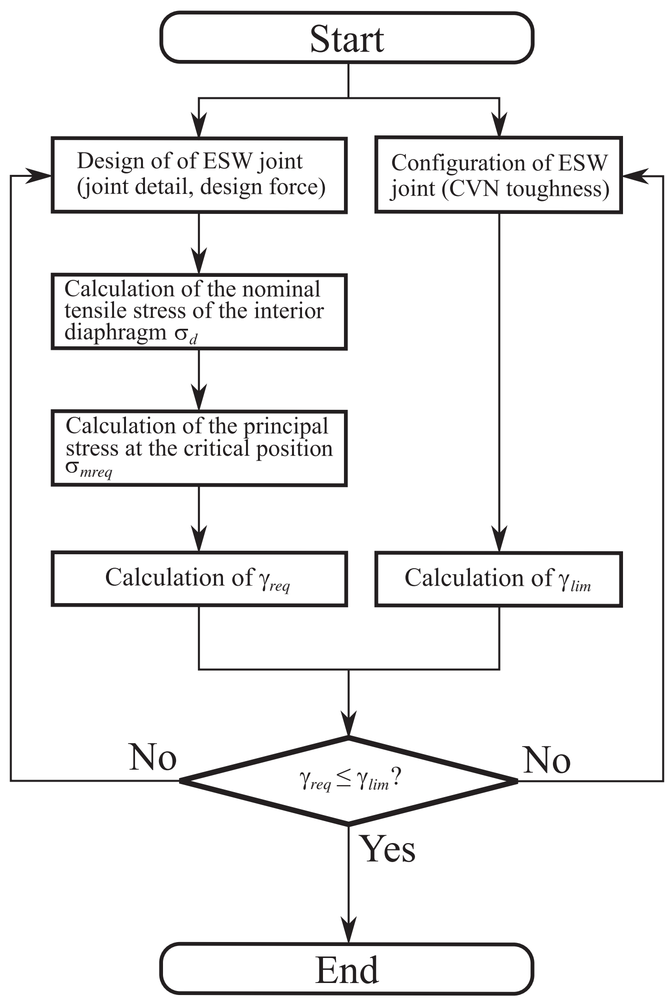


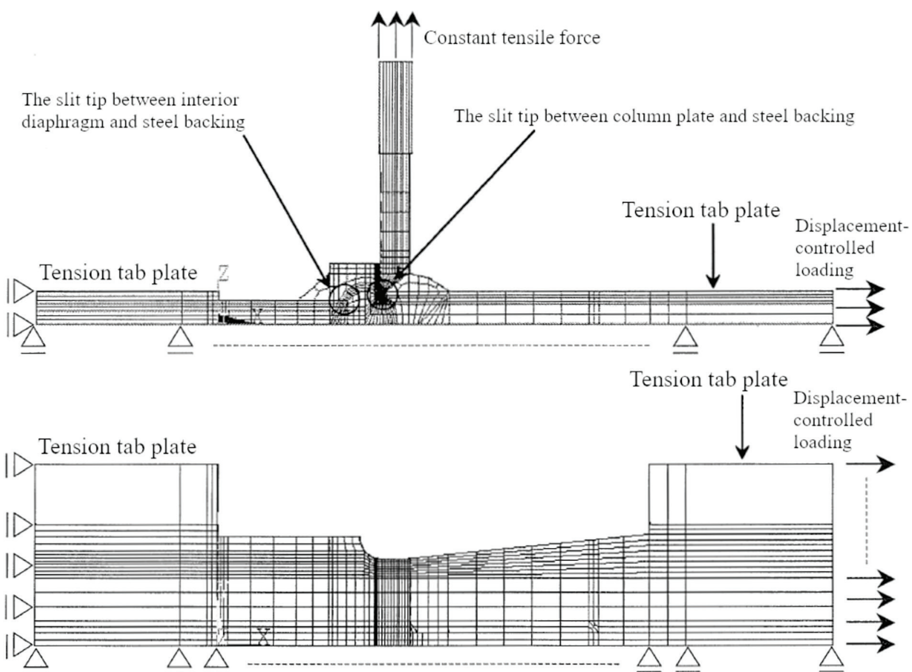
Fig. 17. Procedure for detailed assessment of fracture design of ESW joints (adapted from JSSC, 2016a).

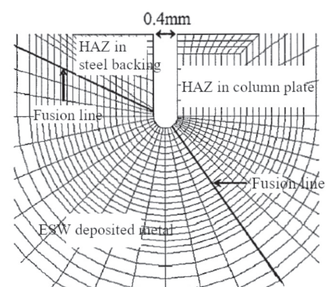
the principal stress limit in the ESW joints by integrating loading test results, FEA results, and fracture mechanics-based investigation. The limiting joint strength index γ_{lim} corresponding to CVN is then calculated using the principal stress limit.

Weld Testing Method for ESW at the Interior Diaphragm

To examine the risk of brittle fracture of an ESW joint, it is necessary to evaluate the toughness of the ESW joint. Because the starting point of the fracture of an ESW joint is at the slit tip between the ESW joint and the steel backing and the point is close to the fusion zone (the boundary between the steel material and the weld metal), the *Guidebook* provides a clear procedure to sample CVN specimens in order to determine the toughness of the ESW joint; such unified procedure did not exist prior to the publication of the *Guidebook*.

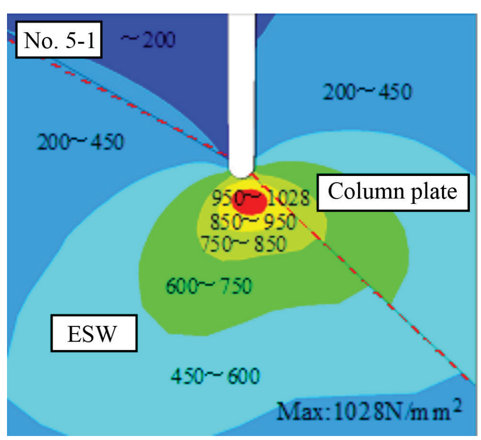
For each ESW joint, V-notch specimens are sampled from three positions: the bottom of the V-notch is on the bond zone (Bond), 1 mm (0.039 in) toward the weld metal from the bond (Depo1), or 1 mm (0.039 in) toward the base metal from the bond (HAZ1) as shown in Figure 20. The reason the three positions are chosen is to evaluate the toughness





(Finite element mesh near the slit tip between column plate and steel backing)

(a) Finite element model



(b) Example of principal stress distribution near the slit tip between column plate and steel backing

Fig. 18. Principal stress distribution near the slit tip (adapted from Song et al., 2009).

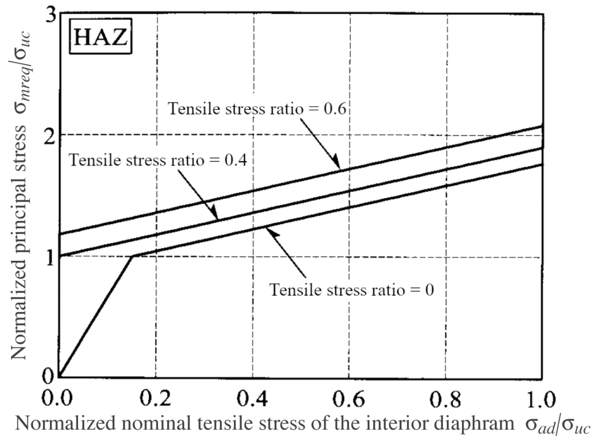
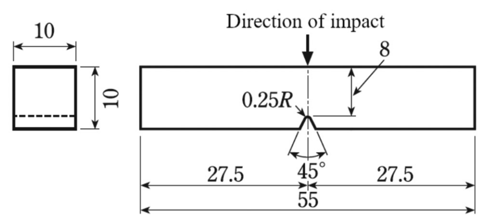
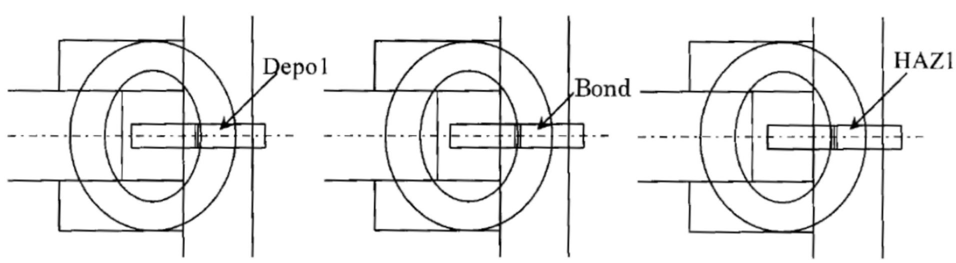


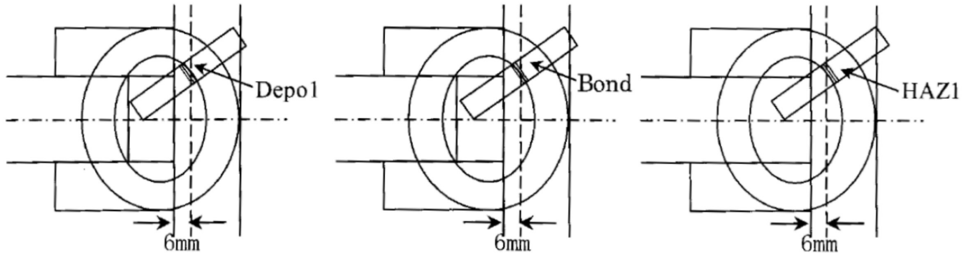
Fig. 19. Normalized principal stress versus normalized nominal tensile stress in the HAZ of ESW joint (adapted from JSSC, 2016a).



(a) CVN specimen (unit: mm) (adapted from JSSC, 2009)



(b) Type A sampling (adapted from JSSC, 2016a and 2016b)
[An annular area represents the HAZ, as shown in Figs. 10 and 12(b)-(d)]



(c) Type B sampling (adapted from JSSC, 2016a and 2016b)

Fig. 20. Collection positions of Charpy V-notch test specimens.

of the material surrounding the slit tip. Figure 20(a) shows the specimen shape, which is the same defined as in ISO 148-1 (ISO, 2016). Three V-notch specimens at each notch position are required for a total of nine specimens. At each position, the average CVN value is calculated from three specimens, and the lowest average CVN value from Depo1, Bond, and HAZ1 represents the toughness of the ESW joint.

The Charpy V-notch specimens are collected such that the centerline of the interior diaphragm aligns with the centerline of the V-notch specimen, as shown in Figure 20(b). This is called Type A sampling in the *Guidebook*. This sampling method is relatively easy to position and to cut out the V-notch specimen. Therefore, this type of sampling has been widely adopted. However, it has been indicated that the measured Charpy energy decreases significantly, and the toughness of the ESW joint cannot be adequately evaluated if the ESW penetration into the column plate is significant. When this occurs, the bottom of the V-notch of the specimen is near the mid-thickness of the column plate. It is well-known that the toughness of steel is less in the through-thickness direction than in rolling or transverse direction, and the centerline segregation of steel plates may further reduce the toughness.

In addition to conventional Type A sampling, various V-notch specimen sampling methods were investigated to avoid inaccuracy in the toughness. To address the potential limitation of Type A sampling, Type B sampling method was introduced in the *Guidebook*, as shown in Figure 20(c). In Type B sampling, the longest edge of the V-notch specimen is not parallel to the centerlines of the interior diaphragm, unlike in Type A sampling. As shown in Figure 20(c), the specimen is oriented at the position where the V-notch is at or near the intersection between the line of the ESW joint and a line 6 mm (0.24 in) away from the back surface of the column plate. The choice of the sampling type depends on the extent to which the fusion line of the ESW joint is close to the mid-thickness of the column plate. In a recent manual for the mechanical test of weldment in steel building structures (JSSC, 2016b), however, Type B sampling is adopted as the standard sampling method for Charpy impact test specimens from the ESW at the interior diaphragm.

CURRENT AND FUTURE DEVELOPMENT

The *Guidebook* is intended for the 490 N/mm^2 (71 ksi) class of steel column plates, and the research challenge is to expand the application to high-performance steel. For example, a series of experimental studies on the ESW of SA440C steel, whose tensile stress is higher than 590 N/mm^2 (86 ksi), was conducted (Iyama et al., 2019), that validates the assessment methods in the *Guidebook* with a slight modification for higher-strength steels. Studies on the 780 N/mm^2 class steel are currently under way. In addition, the use of ESW on high-strength structural steel with a thermo-mechanically controlled process (TMCP) was studied (Fujisawa et al., 2019); softening of the HAZ in the ESW joint of TMCP steel is inevitable, and this study demonstrated that the ESW joints that had enough penetration—that is, t_w was large enough, as shown in Figure 16(a)—did not show brittle fracture if the HAZ was softened.

ACKNOWLEDGMENTS

This paper was prepared as the work of the International Publicity Committee on Welding Technology for Building Structures in the Japanese Society of Steel Construction under the financial support of the Japan Iron and Steel Federation. The authors are grateful to the two organizations and other members of the committee. Professor Chia-Ming Uang at the University of California, San Diego, and Professor Atsushi Sato at Nagoya Institute of Technology gave many valuable comments to the draft of this paper; the authors would like to express our deepest gratitude to them.

REFERENCES

Agic, T. and Hampton, J.A. (1968), "Electroslag Welding with Consumable Guide on the Bank of America World Headquarters Building," *Welding Journal*, December, pp. 939–946.

AIJ (1971), *Welding Work Standards V Consumable Electroslag Welding*, Architectural Institute of Japan. (in Japanese)

Akiyama, H., Yamada, S., Matsumoto, Y., Takeuchi, T., and Sugimoto H. (2002), "Full Scale Shaking Table Test on Ultimate Seismic Resistance of Advanced Type of Beam-to-Column Connections," *Journal of Structural and Construction Engineering (Transactions of AIJ)*, Vol. 67, No. 551, pp. 141–148. (in Japanese)

Chambers, J.J. and Manning, B.R. (2016), "Electroslag Welding: From Shop to Field," *STRUCTURE magazine*, February, pp. 20–23.

Fujisawa, S., Umeda, T., Shimokawa, H., Ishii, T., Harada, Y., Morita, K., Kobayashi, M., and Song, Y.H. (2019), "Highly Efficient Welding Process for Built-up Box-Section Column Tensile Tests of Electro-Slag Welded Joints of Thermo Mechanical Control Process (TMCP) Structural Steel," presented at 12th Pacific Structural Steel Conference, Tokyo.

Ishiguro, T., et al. (1975), *Welding in Steel Building Structures*, Rikoh Tosho. (in Japanese)

ISO (2016), Metallic Materials—Charpy Pendulum Impact Test—Part 1: Test Method, ISO 148-1:2016, International Standards Organization, Geneva, Switzerland.

Iyama, J., Matsumoto, Y., Ishii, T., Shimokawa, H., Nikaido, M., and Yamada, S. (2019), "Fracture Strength of Electroslag Welding Joint with High-Performance Steel," *Journal of Constructional Steel Research*, Vol. 153, pp. 495–508.

Japan Steel Structure Journal (1998), *Development of Steel Structure Technology in Japanese Architecture—A Brief History of 50 Years after World War II*, Chapter 18, Production Processing Technology. (in Japanese)

Japan Welding Engineering Society (2004), Q09-02-01: "Built-up Box Sections Are Often Used for Steel Columns of Building Structures. How are interior diaphragms welded in through-column type beam-to-column joints?" Japan Welding Engineering Society, http://www-it.jwes.or.jp/qa/details.jsp?pg_no=0090020010 (accessed July 28, 2022). (in Japanese)

JSSC (2009), *Introduction to Design of Steel Structures,* 4th Ed., Japanese Society of Steel Construction. (in Japanese)

JSSC (2016a), *Guidebook for Prevention of the Occurrence of Brittle Fractures of Inner Diaphragm Plate Electroslag Welds*, Japanese Society of Steel Construction. (in Japanese)

JSSC (2016b), *Manual for Standard Mechanical Tests of Weld Zone in Steel Building Structures*, Japanese Society of Steel Construction. (in Japanese)

Kayamori, Y., Suzuki, T., Shimokawa, H., Tabuchi, M., Nakagome, T., and Morita, K. (2007), "Study on Fracture Properties of Electroslag Welds: Part 12, Correlation between Charpy Absorption Energy and Maximum Principal Stress," *Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan. C-1, Structures III*, pp. 789–790. (in Japanese)

Lee, P., Garai, R, Ozkula, G., Uang, C.M., and Sarkisian, M. (2014), "Issues on Using Welded Built-up Box Columns in Steel Special Moment Frames," presented at the Tenth U.S. National Conference on Earthquake Engineering, Anchorage, Alaska, July 21–25.

Matsumoto, M. (2000), "Steel Structures," *Journal of the Japan Welding Society*, Vol. 69, No. 8, pp. 628–632. (in Japanese)

Nippon Steel Welding & Engineering (2019), *Simplified Electroslag Welding Process with Non-Consumable Elevating Tip (SESNET method)*, Nippon Steel Welding & Engineering Co. Ltd., https://www.weld.nipponsteel.com/techinfo/weldqa/detail.php?id=27U3U9W (accessed July 28, 2022). (in Japanese)

Sakurai, R. and Takahara, S. (1966), "Electroslag Welding Applied to Steel Building Structures," *Manufacturing & Technology*, Vol. 18, No. 4, pp. 3–16. (in Japanese)

Shimanuki, H., Suzuki, T., and Azuma, K. (2003), "An Investigation on Brittle Fracture of Electro-Slag Welded Joint between Interior Diaphragm and Box Column, Part 1. Material Properties of Welded Joint," *Summaries of Technical Papers of Annual Meeting Architectural Institute of Japan. C-1, Structures III*, pp. 877–878. (in Japanese)

Shimokawa, H., Song, Y.H., Ishii, T., Suzuki, T., Harada, Y., and Morita, K. (2019), "Study on Fracture Behavior of Electroslag Welding Joint," presented at the 12th Pacific Structural Steel Conference, Tokyo, Japan.

Song, Y.H., Shimokawa, H., Ishii, T., Suzuki, T., Kayamori, Y., Harada, Y., and Morita, K. (2009), "Study on Fracture Behavior of Electroslag Welded Joints Using Cruciform Model Joints," *Kou kouzou rombunshuu*, Vol. 16, No. 64, pp. 49–63. (in Japanese)

Ueno, M. and Ariyasu, H. eds. (1968), *Consumable Nozzle Electroslag Welding*, Morikita Publishing. (in Japanese)

Umeda, T., Ishii, T., Fujisawa, S., and Shimokawa, H. (2022), "Effect of Penetration Depth of Electro-Slag Welding on Ductility of Beam-to-Column Connection," *Journal of Constructional Steel Research*, Vol. 189, Article No. 107081.

Wada, T., Namba, M., and Tanaka, T. (1990), "Welding Consumables for Building and Civil Engineering," *Journal of the Japan Welding Society*, Vol. 59, No. 7, pp. 514–521. (in Japanese)

Yamaguchi, M. (1997), "Welding Consumables and Procedures for Electroslag Welding," *Journal of the Japan Welding Society*, Vol. 66, No. 4, pp. 279–283. (in Japanese)