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Ultimate strength of austenitic stainless steel fillet-welded connections with weld metal fracture

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ABSTRACT

This study investigates ultimate behaviors such as ultimate strength and weld metal fracture mechanism of austenitic stainless steel (304 type) fillet-welded connections with TIG (tungsten inert gas) welding through experimental method. Main variables of test specimens are weld length and welding direction against applied force. Specimens are fabricated with transverse fillet weld (TFW series), longitudinal fillet weld (LFW series) and full fillet weld FW series against loading direction. Specimens of TFW series, LFW series and FW series failed by tensile fracture, shear fracture and block shear fracture in the weld metal of connections. Test ultimate strengths were compared with predictions based on the design specifications, AISC (American institute of steel construction) and ASCE (American Society of Civil Engineers) and existing researcher's equations. Current design equations underestimated excessively the strength of welded connections with weld metal fracture. Consequently, modified design formula is suggested considering actual fracture path and the effect of stress triaxiality on welded connections and its validation was verified in this paper.

1. Introduction

Since stainless steel has various excellent characteristics such as corrosion resistance, durability, aesthetic appeal and fire resistance etc. compared with carbon steel. The application of stainless steel as structural material in building construction has been steadily increasing thanks to characteristics stated above. The annual consumption of stainless steel has increased at a growth rate of 5% over the last 20 years, including the industrial field, building and infrastructure. Especially, the rate of growth of stainless steel used in building and infrastructure construction has been even faster, not least due to rapid development in China. Stainless steel has traditionally been used for non-structural or secondary parts such as facades and roofing since the 1920s. Nowadays, stainless steel is used in a very wide range of structural elements, from back-up structures and castings for glazing systems to load-bearing members in building and arches in bridges [1]. Stainless steels can be divided into five main groups; austenitic, ferritic, martensitic, duplex and precipitation hardening. Austenitic stainless steels among them, especially 304 and 316 types are the most widely utilized in building structures. The austenitic grades have excellent corrosion resistance, as well as good formability, toughness and weldability. Their high impact strength at low temperatures is often exploited in the applications with extremely low temperatures. The austenitic grades are non-magnetic in the fully austenitic condition as occurs in well-annealed alloys. They are generally readily welded, but care is required in the selection of consumables and practices for more highly alloyed grades. Austenitic stainless steels are often described as non-magnetic, but may become slightly magnetic when machined or worked. Austenitic structure (FCC, face centered cubic structure) is very tough and ductile down to absolute zero. In addition, austenitic structures do not lose strength at elevated temperatures as rapidly as ferritic structure (BCC, body centered cubic structure). Many studies have been performed extensively on the structural behaviors of welded connections for these austenitic types due to low welding workability and superior corrosion resistance in the industrial fields. Topkaya and Oosterhof et al. proposed the design equations for block shear fracture in the base metal of welded connections fabricated with carbon steel not weld metal fracture and finite element analyses were conducted to predict the ultimate block shear strength of the welded specimens and the respective failure strength of the shear and tension planes [2,3]. Topkaya and Oosterhof et al. proposed unified equations such as Eq. (1) and Eq. (2), respectively that provided more consistent block shear capacity predictions than Eq. (3) and Eq. (4) of AISC specification for bolted connections through experimental and numerical procedures. The proposed equations are as follows,

$$R_n = \frac{F_u}{\sqrt{3}}A_{gv} + 1.25F_uA_{gt} \tag{1}$$

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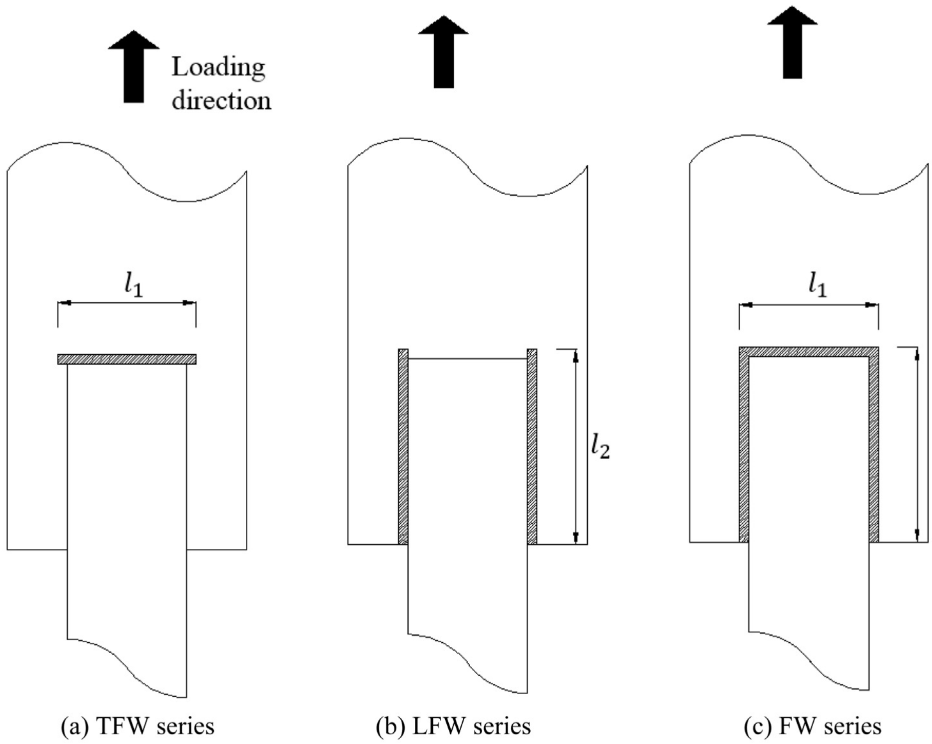

Fig. 1. Specimen series according to the welding direction against loading direction. (a) TFW series. (b) LFW series. (c) FW series.

$$R_n = \frac{F_y + F_u}{2\sqrt{3}} A_{gv} + U_t F_u A_{nt} \tag{2}$$

Where, R_n is the block shear strength in welded connection, F_y is the yield stress of materials, F_u is the tensile strength of materials, A_{gv} is gross area subject to shear, A_{gt} is gross area subject to tension, A_{nt} is net area subject to tension, U_t is equivalent tensile stress factor; taken as 1.25 in Eq. (2) for the case of concentrically loaded welded lap plate connections.

The block shear fracture behavior of welded connection is not well understood. Moreover, although it has been shown to govern failure in experimental tests for certain connection geometries, this case is not addressed explicitly in current design specifications. Topkaya and Oosterhof et al. have shown that the direct application of AISC equations to welded connections consistently underestimates ultimate strength observed in physical tests [3]. Therefore, a better understanding of block shear fracture in welded connection is required to achieve efficient and reliable designs of welded connections,.

Also, Fen Wei et al. performed an experimental study of block shear fracture behaviors of coped beams with a welded clip angle connection [4]. Main parameters were the web block aspect ratio (height to width ratio of the web block connected to the clip angle) and the connection rotational stiffness. It is known from test results that the block shear fracture strength for most test specimens got higher with increasing web block aspect ratio and increasing connection rotational stiffness. Test strengths were compared with those calculated by current design specifications. As a result, since the design equations did not consider the influence of the web block aspect ratio and the connection rotational stiffness, they provided conservative predictions of the block shear strengths of the test specimens.

Representatively, current block shear fracture equations in the AISC specification for bolted connections consider a combination of net tension plane fracture ($F_u A_{nt}$) and either yielding of the gross shear plane ($0.6F_y A_{gv}$) or fracture of the net shear plane ($0.6F_u A_{nv}$), with the capacity being taken as the lesser of the following two Eq. [5],

$$R_n = 0.6F_y A_{gv} + U_{bs} F_u A_{nt} \tag{3}$$

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \tag{4}$$

Where, R_n is the block shear strength in bolted connection, F_y is the yield stress of materials, F_u is the tensile strength, A_{gv} is gross area subject to shear, A_{nv} is net area subject to shear, A_{nt} is net area subject to tension, U_{bs} is reduction coefficient; taken as 1.0 for uniform tensile stress and 0.5 for non-uniform tensile stress.

The effects of welding on G450 sheet steel (which is included in AS 1397 and is a cold-reduced sheet steel with in-line galvanizing. The steel has 450 MPa yield stress and 480 MPa tensile strength) in the heat affected zone with respect to the connection strength has been studied by L.P. Teh et al. [6]. Specimens of transverse and longitudinal fillet, and flare-bevel welded connections in G450 thin-walled steels were tested to fail by weld metal in welded connection. It was known from the comparison of test results and design strength predictions the current design rules such as AISI S100-12(North American Specification for Cold-Formed Steel Structural Members) and *AS/NZS 4600* (the Australian/New Zealand Standard for Cold-Formed Steel Structures) were not adequate for certain connection configurations in G450 steel. Reduced resistance factors were proposed for some connections.

However, unfortunately, most of previous studies and proposed equations stated above were focused on the tensile fracture, shear fracture and block shear fracture of base metal and weld metal in carbon welded connections. AISC specification for structural steel buildings with hot-rolled carbon steels [5] and ASCE specification for the design of cold-formed stainless steel structural members [7] provide the design fillet-weld strengths of tensile fracture and shear fracture according to loading directions against welding axis not including block shear strength of weld connections.

In this paper, since latest design specifications for hot rolled and cold-formed steel structural members consider the fracture strengths of both base metal and weld metal in welded connections, experimental research has been carried out to investigate the ultimate behaviors of cold-formed austenitic stainless steel (STS304, corresponds to ASTM 304 type) fillet-welded connections by TIG (tungsten inert gas) welding with ultimate fracture in weld metal not base metal. Main variables of test specimens are the weld lengths (in the transverse direction of