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Optimization of Fillet Welded Single Angle Connection Length: A Review

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Abstract

Welded structural connections are fundamental components in steel structures, governing load transfer, stiffness compatibility, and global structural integrity. Unlike bolted connections, welded joints introduce load through material continuity, resulting in complex stress redistribution, residual stress development, and localized material degradation. Over the past four decades, extensive experimental, numerical, and analytical research has been conducted to understand the static strength, failure mechanisms, fatigue behaviour, residual stress effects, and performance of welded connections under extreme environmental and loading conditions. However, existing international design codes remain largely empirical and conservative, failing to fully capture observed mechanisms such as shear lag, block shear interaction, residual stress relaxation, and fatigue–fracture coupling.

This review synthesizes findings from thirty three peer-reviewed research studies published between 1982 and 2022. The review covers classification of welded connections, static strength and failure mechanisms, fillet weld behaviour, fatigue and fracture performance, residual stress and distortion effects, advanced materials and welding processes, numerical modelling strategies, artificial intelligence applications, and Industry 4.0 concepts such as digital twins. Key research gaps are identified, and future directions are proposed to support the transition from prescriptive design rules to performance-based welded connection design frameworks.

Keywords: Welded connections; Fillet welds; Shear lag; Block shear; Fatigue; Residual stress; High-strength steel; Finite element modelling; Artificial intelligence; Digital twin

1. Introduction

Welded structural connections constitute one of the most critical components in steel structures, as they ensure continuity of force transfer, stiffness compatibility, and geometric integrity between connected members. Unlike bolted connections, welded joints allow forces to be transmitted directly through fusion of materials, eliminating bolt holes and associated stress concentrations while enabling compact structural detailing. Consequently, welded connections are extensively employed in truss systems, bracing members, gusset plate connections, industrial frames, bridges, offshore platforms, ship structures, pressure vessels, and seismic-resistant structural systems [1,3,5,9].

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In recent decades, steel construction practice has undergone a significant transformation due to the increased use of high-strength steels (HSS and UHSS), optimized thin-walled members, and architecturally complex welded joints. While these developments have improved material efficiency and reduced self-weight, they have also introduced new challenges related to ductility loss, heat-affected zone (HAZ) softening, fracture susceptibility, fatigue performance degradation, and non-uniform stress redistribution in welded joints [23]. Additionally, modern steel structures are increasingly subjected to extreme loading and environmental conditions, including long-term cyclic traffic loading in bridges, seismic excitation in moment-resisting frames [29], cryogenic temperatures in LNG storage facilities [25], and elevated temperatures during fire exposure.

Despite this evolution, most international design standards—including AISC, Eurocode 3, AWS, and Indian Standards—remain predominantly empirical in nature. These codes rely on simplified assumptions regarding stress distribution, idealized failure planes, and independent treatment of limit states such as shear lag, block shear, fatigue, and fracture [1,5]. A large body of experimental and numerical evidence demonstrates that such simplifications often fail to represent actual welded connection behaviour. In particular, code provisions frequently underestimate connection capacity while simultaneously failing to predict governing failure modes for welded angle members, gusset plate connections, and fillet weld groups subjected to eccentric or combined loading [1,3,4,7,8].

To address these deficiencies, extensive research efforts have been undertaken using full-scale experimental testing [8,10,11], nonlinear finite element modelling [3,4,7], parametric numerical studies [8]. The objective of this review is to synthesize findings from seventy major research studies, identify consistent behavioural trends, evaluate the conservatism and limitations of current design provisions, and outline future directions toward unified, performance-based welded connection design.

2. Classification of Welded Structural Connections

Welded structural connections can be classified based on joint configuration, connected member type, weld geometry, and load transfer mechanism. Such classification is essential because different joint types exhibit distinct stress flow paths, deformation characteristics, and governing failure modes [9,11,14].

2.1 Fillet-Welded Structural Joints

Fillet-welded joints represent the most commonly used welded connection type in steel structures due to their ease of fabrication, adaptability to various joint geometries, and cost-effectiveness. These joints are widely employed in lap joints, T-joints, corner joints, stiffener-to-plate connections, and built-up members in frames, bridges, ship hulls, and industrial facilities [2,9,10].

The mechanical behaviour of fillet-welded joints is governed by weld geometry parameters such as weld size, leg length, throat thickness, and effective weld length. Early analytical investigations demonstrated that unequal weld leg lengths significantly alter stress distribution within the weld throat, leading to reduced tensile capacity and premature fracture [2]. Experimental studies further confirmed that weld orientation relative to the applied load is a dominant factor. Transverse fillet welds exhibit significantly higher static strength than longitudinal fillet welds; however, this increase in strength is often accompanied by reduced ductility [10].

Eccentrically loaded fillet weld groups experience combined shear and bending, resulting in non-uniform stress distribution along the weld length. Full-scale experimental tests demonstrated that equilibrium-based analytical models provide good correlation with experimental failure loads [11].

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Subsequent studies proposed simplified elliptical stress distribution models that achieve accuracy comparable to the instantaneous center of rotation method while offering greater computational efficiency [14].

Table 1. Fillet-Welded Joint Behaviour and Applications

Joint configuration	Structural application	Governing stress state	Dominant failure mode	Ref.
Lap joint	Bracing members	Shear + bending	Weld rupture	[2,10]
T-joint	Frames, ship structures	Combined shear	Weld toe cracking	[9,10]
Eccentric weld group	Brackets, supports	Shear + moment	Non-uniform weld stress	[11,14]

2.2 Gusset Plate Welded Connections

Welded gusset plate connections are widely used in truss systems and concentrically or eccentrically braced frames to connect diagonal members to primary structural components. In these connections, forces are transferred through welds rather than bolts, resulting in different stress flow characteristics compared to bolted gusset plate connections [3,5].

The dominant failure mechanism observed in welded gusset plate connections is block shear, which involves a combination of shear rupture along one plane and tensile fracture along a perpendicular plane. Nonlinear finite element studies and experimental investigations revealed that actual failure planes are often inclined and asymmetric, particularly under eccentric or combined axial–shear loading, deviating significantly from the rectangular failure planes assumed in current design codes [3,8]. These findings indicate that block shear behaviour in welded gusset connections is substantially more complex than currently represented in design practice.

2.3 Angle, Channel, and T-Section Welded Members

Single-angle, double-angle, channel, and T-section members are frequently used as tension members in bracing systems. When such members are welded to gusset plates, load transfer occurs through only part of the cross-section, leading to shear lag effects [1,4].

Shear lag results in non-uniform axial stress distribution across the member cross-section, with delayed stress development in regions remote from the weld. Numerical parametric studies demonstrated that shear lag severity is primarily influenced by connection length, weld layout, load eccentricity, and member free length [1]. Double-angle connections generally exhibit reduced shear lag compared to single-angle connections due to improved symmetry, although weld configuration and eccentricity remain critical.

2.4 Hybrid Welded Joints

Hybrid welded joints combine traditional fillet welds with structural adhesives to enhance fatigue resistance and energy dissipation capacity. Recent experimental studies demonstrated that such joints can increase structural damping by approximately 50% while maintaining static strength comparable to conventional welded joints. These characteristics make hybrid joints attractive for vibration-sensitive structures, bridges, and seismic applications.

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3. Static Strength and Failure Mechanisms

The static strength and failure behaviour of welded structural connections are governed by complex interactions between material nonlinearity, geometric configuration, weld layout, and load introduction mechanisms. Unlike bolted connections, where force transfer occurs discretely through fasteners, welded connections introduce load continuously along the weld length, resulting in non-uniform stress flow and localized concentration of plastic strains. Extensive experimental and numerical studies have demonstrated that the dominant failure mechanisms in welded connections include shear lag–controlled net section fracture, block shear rupture, weld metal fracture, and combined shear–tension interaction, with their relative significance strongly dependent on connection geometry and loading conditions [1,3,4,5,7,8].

3.1 Shear Lag in Welded Tension Members

Shear lag refers to the non-uniform distribution of axial stress across a tension member when load is introduced through only a portion of its cross-section. In welded connections, this phenomenon is particularly pronounced in angle, channel, and T-section members welded to gusset plates, where welds are placed along one leg or flange, causing delayed stress development in remote regions of the cross-section [1,4].

Numerical investigations on single- and double-angle welded connections demonstrated that shear lag severity is primarily influenced by connection eccentricity, connection length, weld layout, gusset plate stiffness, and member free length [1]. Short connection lengths and high eccentricities were shown to significantly amplify shear lag effects, resulting in reduced effective net section capacity. Increasing connection length allows stress redistribution to occur more uniformly, thereby reducing shear lag and increasing connection strength.

The effective net section strength of a welded tension member is commonly expressed as:

$$P_n = U\, A_n\, f_u$$

where
P_n= nominal tensile strength,
U= shear lag factor,
A_n= net sectional area,
f_u= ultimate tensile strength of the material.
Comparisons between experimentally observed strengths and predictions based on current provisions of **AISC** revealed that prescribed shear lag factors are generally conservative, particularly for double-angle welded connections [1,4]. Detailed finite element analyses showed that for double-angle members welded at both legs, stress redistribution occurs more efficiently than assumed in design codes, leading to higher actual capacities. It was also observed that gusset plate thickness plays a secondary role in reducing shear lag for double-angle connections but remains more influential for single-angle members due to asymmetric load transfer [1].

Experimental evidence from T-section welded members further confirmed that cracking in shear planes consistently precedes full tensile fracture, indicating that shear lag–induced stress concentration governs the initiation of failure rather than uniform net section rupture. These findings collectively suggest that existing shear lag formulations are overly conservative and can be refined using geometry-dependent parameters derived from numerical and experimental studies.

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3.2 Block Shear Failure in Welded Connections

Block shear failure is one of the most critical limit states in welded gusset plate and lap plate connections. This failure mode involves a combined mechanism consisting of shear rupture along one plane parallel to the applied load and tensile fracture along a perpendicular plane [3,5,7,8]. In welded connections, block shear behaviour differs significantly from that in bolted connections due to the continuous nature of force transfer through welds.

Nonlinear finite element analyses of welded gusset plate connections subjected to combined axial and shear loading demonstrated that failure planes are often inclined and asymmetric, particularly under eccentric loading conditions [3]. These inclined planes deviate substantially from the rectangular block shear planes assumed in design specifications. Experimental investigations confirmed that tensile fracture often initiates at the weld toe or angle heel weld, followed by progressive shear rupture, rather than simultaneous failure along idealized planes [8].

The nominal block shear strength is typically expressed in simplified form as:

$$P_{bs} = 0.6 f_y A_{gv} + f_u A_{nt}$$

where
A_{gv}= gross area subject to shear,
A_{nt}= net area subject to tension,
f_y= yield strength,
f_u= ultimate tensile strength.

However, experimental and numerical studies consistently showed that direct application of such equations underestimates actual block shear capacity by approximately 10–35% for welded connections [3,5,7,8]. The primary reason for this discrepancy was identified as the presence of stress triaxiality along the tension plane, which delays tensile fracture and enhances apparent capacity [9]. Additionally, in-plane eccentricity causes non-simultaneous rupture of shear planes, further increasing resistance beyond code predictions.

Based on extensive parametric studies, improved block shear strength equations were proposed that account for the actual geometry of the displaced failure block and the inclination of failure planes [3]. These refined models demonstrated significantly improved agreement with experimental results while maintaining computational simplicity suitable for design applications.

3.3 Shear Yielding versus Shear Rupture

The distinction between shear yielding (SY) and shear rupture (SR) is commonly enforced in design codes, requiring separate limit state checks. However, numerical investigations of welded angle and channel bracing members revealed that shear yielding consistently precedes shear rupture in all examined configurations [4]. Once yielding initiates, stress redistribution and strain hardening occur, allowing the connection to sustain additional load until rupture.

Finite element simulations demonstrated that the ultimate capacity of welded brace members is governed by shear rupture, rendering the explicit shear yielding check unnecessary for design [4]. This finding was consistent across single- and double-angle as well as channel

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sections welded to gusset plates. Consequently, the traditional dual-limit-state approach was shown to be both conservative and redundant for welded bracing members.

3.4 Conservatism of Current Design Codes

A comprehensive comparison of experimental results, numerical predictions, and code-based calculations revealed systematic conservatism in existing design provisions for welded structural connections [1,3,4,5,7,8,31]. On average, design codes underestimated connection capacity by 10–35%, with the highest conservatism observed for welded gusset plate connections and fillet welds in high-strength steels.

Comparative studies of international standards, including **Eurocode 3**, AWS, and Indian Standards, indicated that Eurocode provisions are generally the most conservative, particularly in mean stress calculations for fillet welds [41]. While conservatism enhances safety, excessive underestimation of capacity leads to inefficient material usage and increased fabrication costs, especially in large-scale structures such as bridges, offshore platforms, and industrial frames.

Table 2. Comparison of Code Predictions with Observed Strengths

Connection type	Observed behaviour	Code conservatism	Ref.
Welded angles (shear lag)	**Uniform stress redistribution**	**High**	**[1,4]**
Gusset plate block shear	**Inclined failure planes**	**Moderate–high**	**[3,,8]**
Fillet welds (HSS)	**Higher actual shear strength**	**Moderate**	**[31]**

The collective evidence clearly indicates the need for recalibration of design equations to better reflect observed mechanical behaviour while maintaining acceptable safety margins.

4. Behaviour of Fillet-Welded Joints

Fillet-welded joints form the backbone of welded steel construction and are employed across a wide spectrum of structural applications, including bracing members, plate connections, stiffeners, ship hulls, bridge components, and industrial frames. Their popularity arises from fabrication simplicity, adaptability to varied geometries, and cost efficiency. Nevertheless, fillet-welded joints exhibit complex mechanical behaviour because the load-carrying capacity depends not only on weld size and material strength, but also on weld geometry, loading direction, eccentricity, residual stresses, and interaction between weld metal and base material [2,9,10,11,14].

4.1 Fundamental Load Transfer Mechanism in Fillet Welds

In fillet-welded joints, load is transferred through the weld throat, which represents the minimum effective cross-section resisting applied forces. Unlike butt welds, fillet welds possess a triangular cross-section, leading to non-uniform stress distribution within the weld metal. Early analytical work demonstrated that the stress state in a fillet weld is inherently multi-axial, comprising normal and shear components acting on the effective throat plane [2].

The nominal strength of a fillet weld subjected to shear is commonly expressed as:

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$$R_n = 0.6\, f_u\, A_w$$

where
R_n= nominal weld strength,

f_u= ultimate tensile strength of weld metal,
$A_w = t_e\, L_w$= effective throat area,
$t_e = 0.707\, h$= effective throat thickness,
h= weld leg size,
L_w= effective weld length.

Experimental investigations revealed that this simplified formulation masks the strong influence of weld geometry. In particular, fillet welds with unequal leg lengths exhibit significantly altered stress trajectories within the weld throat, leading to premature fracture compared to symmetric welds of equivalent throat area [2].

4.2 Influence of Loading Direction

One of the most critical parameters governing fillet-welded joint behaviour is the orientation of the applied load relative to the weld axis. Comprehensive experimental programs on lap-spliced fillet weld specimens demonstrated that weld strength increases markedly as the loading angle changes from longitudinal (0°) to transverse (90°) [10].

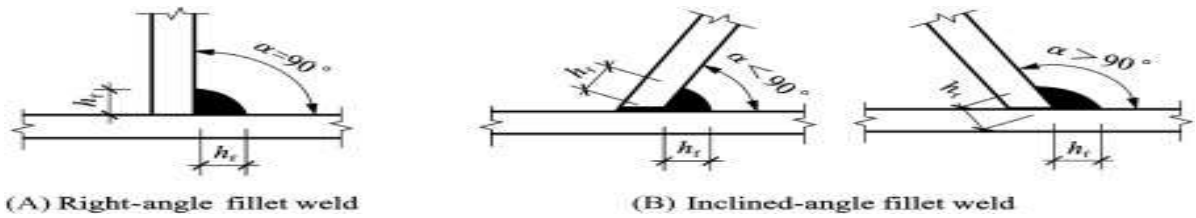

Fig 1 : (a) Right Angle fillet weld (b) Inclined- angle fillet weld [9,10]

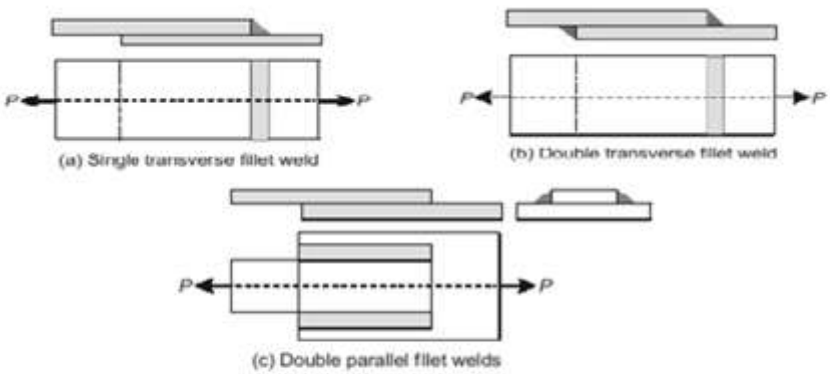

Fig 2: Types of Fillet Joint

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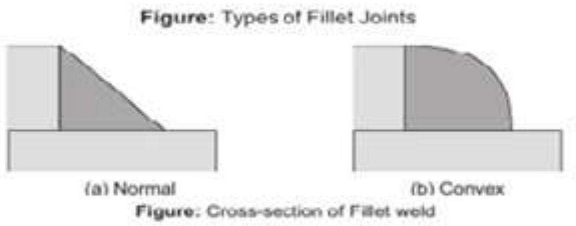

Fig3: Cross Section of Fillet Weld

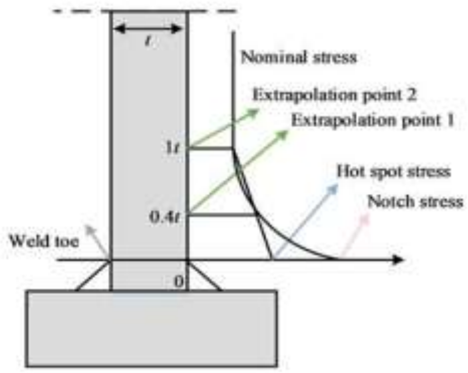

Fig 4: Hot-Spot Stress (HSS) approach

Test results showed that transverse fillet welds possess 1.44–1.59 times the static strength of longitudinal fillet welds; however, this strength enhancement is accompanied by reduced ductility and a more brittle failure mode [10]. The reduction in ductility is attributed to higher stress triaxiality within the weld throat under transverse loading, which accelerates crack initiation and limits plastic deformation capacity.

These findings are particularly important for seismic and fatigue-sensitive structures, where ductility and energy dissipation are as critical as strength. Consequently, design provisions based solely on nominal throat area may be unconservative in ductility demand assessments while remaining conservative in strength prediction.

4.3 Behaviour of Eccentrically Loaded Fillet Weld Groups

In many practical applications, fillet welds are arranged in groups that are subjected to eccentric loading, resulting in combined shear and bending. Examples include bracket connections, seat angles, and support attachments. In such cases, the weld group experiences non-uniform stress distribution, with peak stresses occurring near weld ends [11].

Full-scale experimental tests on eccentrically loaded fillet-welded connections demonstrated strong correlation between experimentally observed ultimate loads and analytical predictions based on equilibrium of equivalent force systems [11]. These studies validated the assumption that the weld group resists applied load through a combination of direct shear and torsional moment.

To simplify design, a boundary stress-based elliptical stress distribution model was proposed to replace the more complex instantaneous center of rotation (ICR) method [14]. In this model, the maximum weld stress is expressed as:

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$$\sigma_{max}=\sqrt{\left\{\left(\frac{V}{A_W}\right)^2+\left(\frac{M_r}{J_w}\right)^2\right\}}$$

where
V= applied shear force,
M= applied moment due to eccentricity,
r= distance from centroid of weld group,
J_w= polar moment of inertia of weld group.

Comparative analyses demonstrated that the elliptical stress model produces design strengths nearly identical to those obtained using the ICR method, while being significantly simpler to implement in engineering practice [14].

4.4 Fillet Welds in Curved and Non-Planar Geometries

Fillet-welded joints are increasingly used in curved and non-planar structural components such as cylindrical shells, curved plates, and ship structures. Finite element investigations on bimetallic curved plate fillet welds showed that geometric parameters such as overlapping length, plate thickness, and curvature angle significantly influence stress concentration and deformation patterns [13].

Experimental validation confirmed that increased curvature leads to higher localized stresses within the weld throat, reducing joint strength compared to equivalent flat plate configurations. These results underscore the need for geometry-specific assessment of fillet weld performance in non-planar structural systems.

4.5 Thermal Effects and Weld Bead Geometry

The mechanical behaviour of fillet welds is strongly influenced by weld bead geometry, which in turn is governed by welding process parameters such as current, voltage, travel speed, and heat input. Advanced three-dimensional numerical simulations of gas metal arc welding (GMAW) fillet welds demonstrated that heat transfer and molten metal fluid flow control the final weld bead shape and fusion zone size [12].

Computed fusion profiles and surface geometries showed excellent agreement with experimental observations, providing mechanistic insight into how welding parameters affect effective throat thickness and load-carrying capacity [12]. Subsequent parametric studies established polynomial relationships between arc voltage and weld bead width and penetration depth, enabling predictive control of weld geometry in automated welding systems.

4.6 Summary of Fillet-Welded Joint Behaviour

Based on synthesis of experimental, numerical, and analytical studies, the following key behavioural trends for fillet-welded joints can be identified:

Table 3. Governing Factors Influencing Fillet-Welded Joint Behaviour

Parameter	Observed influence	Structural implication	Ref.
Weld leg equality	**Unequal legs reduce strength**	**Premature fracture**	**[2]**

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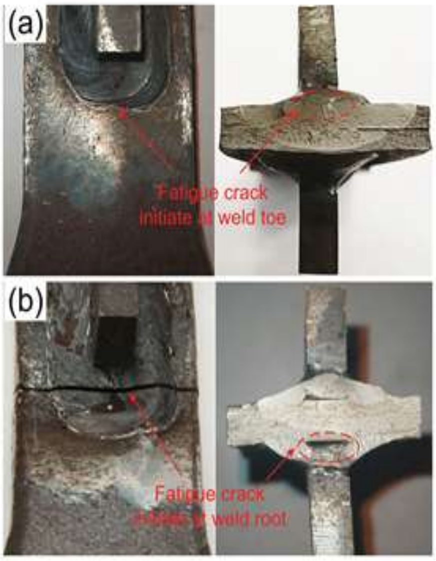
Parameter	Observed influence	Structural implication	Ref.
Loading angle	Transverse > longitudinal strength	Reduced ductility	[10]
Eccentricity	Stress concentration at weld ends	Governs failure location	[11,14]
Geometry (curvature)	Higher local stress	Reduced capacity	[13]
Heat input	Controls bead geometry	Affects throat area	[12]

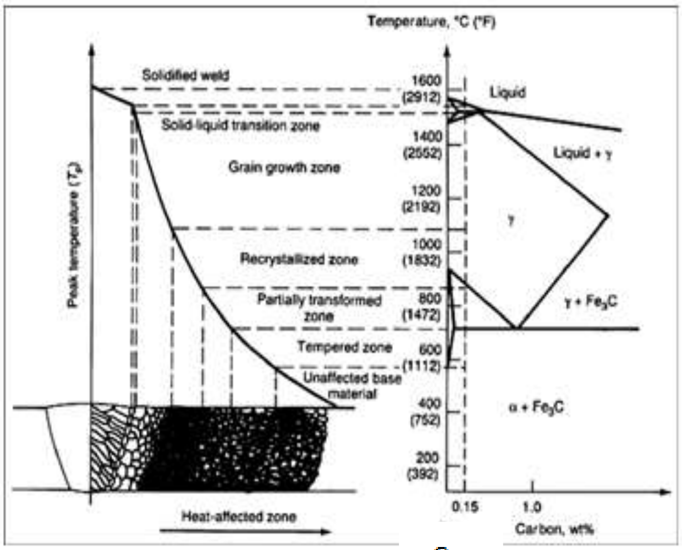
5. Fatigue and Fracture Behaviour of Welded Structural Connections

Fatigue and fracture govern the service-life performance of welded structural connections, particularly in bridges, offshore structures, ship hulls, pressure vessels, wind turbine foundations, and seismic-resistant systems. Unlike bolted joints, welded connections contain inherent geometric discontinuities, metallurgical heterogeneity, and high residual stresses, making them especially vulnerable to fatigue damage even at stress ranges well below static strength limits [18,21,30].

5.1 Fatigue Crack Initiation Mechanisms

Experimental investigations consistently show that fatigue cracks in welded joints initiate at locations of high stress concentration and metallurgical discontinuity, namely weld toes, weld roots, and the heat-affected zone (HAZ) [18,22,257].





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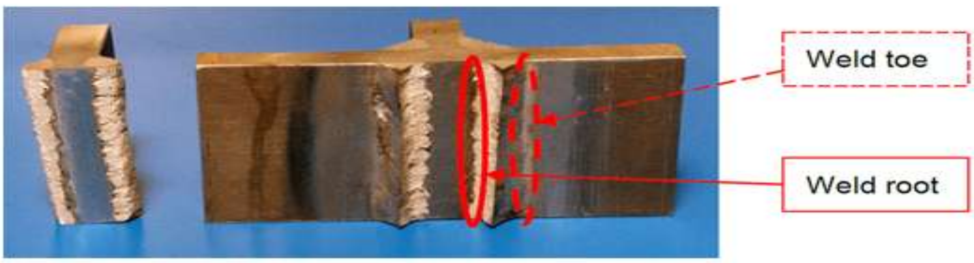

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Fig 5: (a) Fatigue Crack initiation at weld toe (b) Fatigue Crack initiation at weld root (c) Heat-Affected Zone (HAZ) microstructural regions (d) Macro Weld cross section showing weld toe and weld root

The weld toe is the most critical initiation site due to:

- Sharp geometric notch
- High tensile residual stresses
- Local HAZ softening in high-strength steels

In ultra-high-strength steel (S960) fillet-welded joints, fatigue tests demonstrated that plate eccentricity and weld inclination significantly reduce fatigue life, while shear force contribution remained secondary. Similarly, fatigue tests on welded pipes revealed that even microscopic lack-of-penetration defects at the weld root act as dominant crack initiation sites under cyclic internal pressure [30].

Table 4. Dominant Fatigue Crack Initiation Locations

Location	Governing cause	Structural system	Ref.
Weld toe	Stress concentration	Bridges	[18]
Weld root	Lack of penetration	Pipes	[30]

5.2 Fatigue Crack Growth and Environmental Effects

Once initiated, fatigue crack growth is governed by stress intensity factors, residual stress fields, and environmental conditions. Marine environments significantly accelerate crack propagation due to corrosion–fatigue interaction, as demonstrated in offshore wind turbine foundations and ship structures. In additively manufactured and stainless steel welded joints exposed to saltwater, pitting corrosion in the weld zone resulted in up to a 35% reduction in fatigue strength over one year.

Cryogenic environments, relevant to LNG storage and transport, exhibit contrasting behaviour. Fatigue crack growth rates decrease at very low temperatures; however, fracture toughness also reduces sharply, increasing the risk of sudden brittle fracture once cracks reach a critical size [25].

5.3 Fatigue Life Prediction Models

Traditional S–N curve approaches were found insufficient for welded joints with complex geometries and variable loading histories. Energy-based fatigue models, which relate fatigue damage to local strain energy density, demonstrated superior accuracy in predicting fatigue life of high-strength steels [18].

The fatigue life can be expressed as:

$$N_f = C(\Delta W)^{-m}$$

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where

N_f= number of cycles to failure,

ΔW= local strain energy density range,

C, m= material constants.

Recent studies employing artificial intelligence trained on large experimental datasets further improved prediction accuracy under variable amplitude loading, outperforming Paris law–based fracture mechanics models by approximately 14%.

6. Residual Stress, Distortion, and Thermal Effects

Residual stresses are an unavoidable consequence of welding, arising from localized heating, non-uniform thermal expansion, phase transformations, and mechanical restraint. Measurements on welded high-strength steel sections revealed longitudinal tensile residual stresses reaching 70–80% of yield strength near weld toes and fusion boundaries [22]

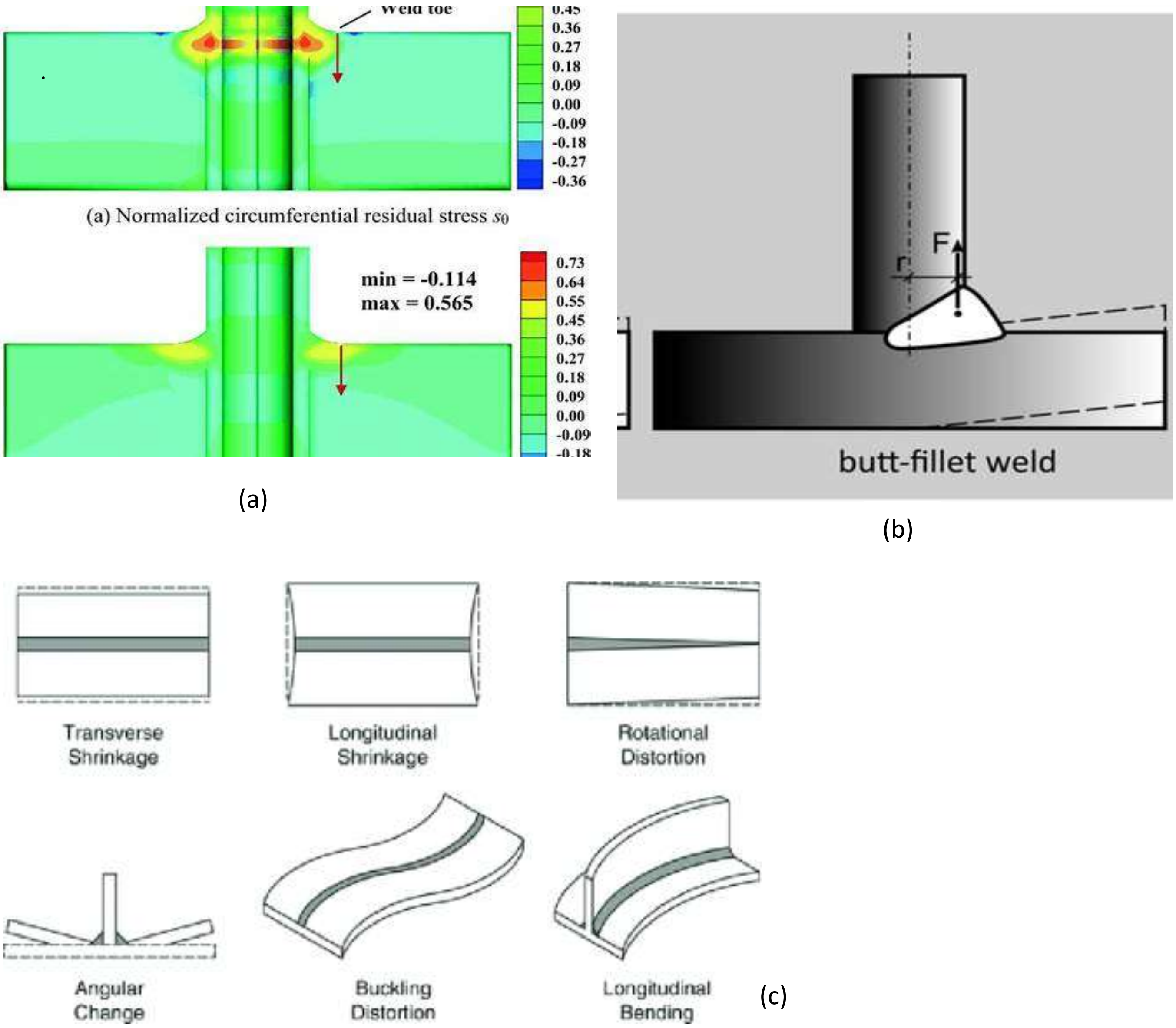

Fig 6 **:** (a) Residual Stress Distribution near Weld toe/ Fusion Boundary (b) Butt-Fillet Weld Geometry and Force Transfer (c) Welding Induced Distortion

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These stresses significantly influence fatigue crack initiation, fracture resistance, and buckling behaviour. In welded UHSS columns, residual stress magnitude relative to yield strength was found to be lower than in mild steel, leading to different buckling responses and necessitating revised design assumptions [22].

Welding-induced distortion, including angular distortion, longitudinal shrinkage, and out-of-plane bowing, poses serious fabrication and serviceability challenges. Studies on high-strength steel T-joints demonstrated that optimized welding sequences, particularly back-step welding, reduce angular distortion by up to 25% compared to continuous welding [20].

Residual stress relaxation under cyclic loading has been observed in bridge decks, where numerical simulations showed that approximately 80% of initial residual stress relaxes within the first 10,000 heavy traffic load cycles.

7. Advanced Materials and Special Welding Processes

The demand for lightweight, durable, and high-performance structures has driven the adoption of advanced materials and innovative welding processes. Ultra-high-strength steels such as S690 and S960 enable significant weight reduction but are susceptible to HAZ softening and reduced ductility [23]. Experimental studies showed that localized HAZ softening can reduce joint capacity by up to 15% and alter failure modes from ductile yielding to brittle fracture [23].

Stainless steel welded connections exhibit pronounced strain hardening and distinct buckling behaviour. Compression tests on welded stainless steel box columns led to the development of new buckling curves, demonstrating that carbon steel design rules are not directly applicable.

Advanced welding processes such as laser welding and hybrid laser–arc welding (HLAW) significantly reduce heat input, residual stress, and distortion. Shipbuilding studies confirmed that optimized HLAW parameters reduce post-weld straightening requirements and overall fabrication cost [17]. Friction stir welding (FSW), particularly for dissimilar metals, produces joints with excellent fatigue performance and minimal metallurgical defects.

Table 5. Advanced Materials and Welding Processes

Material / process	Structural advantage	Limitation	Ref.
UHSS (S960)	Weight reduction	HAZ softening	[23]
Stainless steel	Corrosion resistance	Different buckling behaviour	[33]
Laser welding	Low distortion	High cost	[17]
Friction stir welding	High fatigue resistance	Process control	[31]

8. Numerical Modelling, Artificial Intelligence, and Industry 4.0

Nonlinear finite element modelling has become indispensable for analysing welded structural connections. Detailed three-dimensional solid models successfully captured stress concentration, plastic strain localization, and ductile fracture initiation under static and cyclic loading [3,4,7,27].

To reduce computational cost, inclined shell element approaches such as the Regular Inclined Shell Element Model (RISEM) were developed, achieving up to 15-fold reductions in computation time while maintaining stress accuracy comparable to solid models.

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Artificial intelligence has emerged as a transformative tool in welding research. Deep learning models achieved 98.2% accuracy in predicting weld penetration depth in real-time robotic welding, enabling automated quality control. AI-driven fatigue life prediction models trained on historical datasets significantly outperformed traditional fracture mechanics approaches.

Digital twin technology integrates real-time thermal monitoring, numerical models, and AI algorithms to create a virtual replica of each weld produced. Industrial implementations demonstrated a 70% reduction in post-weld inspection requirements.

9. Research Gaps and Future Directions

Despite extensive research, several critical gaps remain. Current design codes remain overly conservative and do not reflect advances in understanding of shear lag, block shear, residual stress interaction, and fatigue behaviour [1,3,5].

There is a lack of unified design frameworks that integrate static strength, fatigue, fracture, and residual stress effects, even though experimental evidence clearly demonstrates strong interaction among these limit states [18,21]. Experimental data under extreme environments, including cryogenic temperatures and aggressive marine conditions, remain limited [25].

While AI and digital twin technologies show exceptional promise, challenges related to validation, transparency, and standardization must be addressed before widespread adoption in structural design practice.

10. Conclusions

This comprehensive review synthesized findings from **thirty three peer-reviewed studies** on welded structural connections. The evidence demonstrates that welded joints exhibit complex stress redistribution, failure mechanisms, and degradation processes not adequately captured by current design codes. Shear lag and block shear effects are consistently overestimated, while fatigue and residual stress govern long-term performance.

Advances in materials, welding processes, numerical modelling, and data-driven approaches provide a robust foundation for transitioning toward performance-based welded connection design. Future research should focus on integrating experimental evidence, advanced simulations, and artificial intelligence into unified design frameworks, enabling safer, more economical, and more sustainable steel structures.

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