



Delft University of Technology

Document Version

Final published version

Citation (APA)

Wu, W. (2025). *Fatigue behaviour of welded connections in steel orthotropic bridge decks: Experiments and assessments*. [Dissertation (TU Delft), Delft University of Technology]. <https://doi.org/10.4233/uuid:b7f7b552-2f18-407d-8fa0-2af271e40b86>

Important note

To cite this publication, please use the final published version (if applicable).
Please check the document version above.

Copyright

In case the licence states "Dutch Copyright Act (Article 25fa)", this publication was made available Green Open Access via the TU Delft Institutional Repository pursuant to Dutch Copyright Act (Article 25fa, the Taverne amendment). This provision does not affect copyright ownership.
Unless copyright is transferred by contract or statute, it remains with the copyright holder.

Sharing and reuse

Other than for strictly personal use, it is not permitted to download, forward or distribute the text or part of it, without the consent of the author(s) and/or copyright holder(s), unless the work is under an open content license such as Creative Commons.

Takedown policy

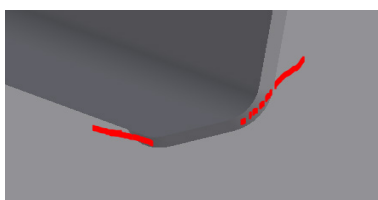
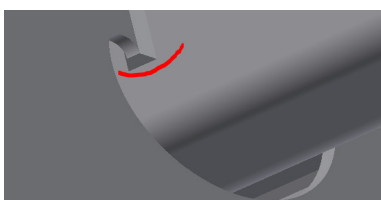
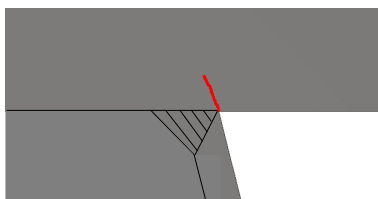
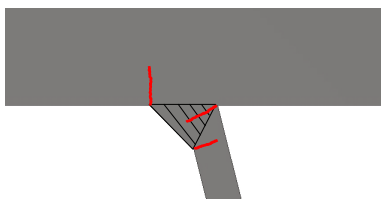
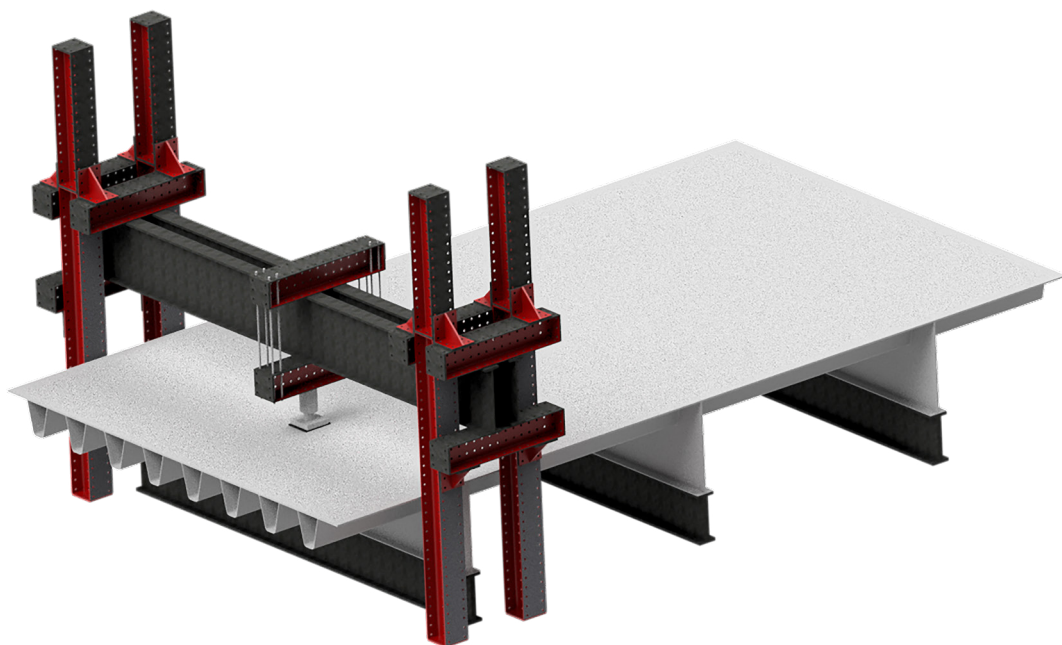
Please contact us and provide details if you believe this document breaches copyrights.
We will remove access to the work immediately and investigate your claim.

This work is downloaded from Delft University of Technology.

FATIGUE BEHAVIOUR OF WELDED CONNECTIONS IN STEEL ORTHOTROPIC BRIDGE DECKS

EXPERIMENTS AND ASSESSMENTS

Wei Jian Wu



FATIGUE BEHAVIOUR OF WELDED CONNECTIONS IN STEEL ORTHOTROPIC BRIDGE DECKS

EXPERIMENTS AND ASSESSMENTS

FATIGUE BEHAVIOUR OF WELDED CONNECTIONS IN STEEL ORTHOTROPIC BRIDGE DECKS

EXPERIMENTS AND ASSESSMENTS

Dissertation

for the purpose of obtaining the degree of doctor
at Delft University of Technology,
by the authority of the Rector Magnificus Prof.dr.ir. T.H.J.J. van der Hagen,
chair of the Board for Doctorates
to be defended publicly on Monday 28 April 2025 at 10:00 am

by

Weijian Wu

Master of Engineering in Architectural and Civil Engineering,
Tongji University, Shanghai, China,
born in Anhui, China.

This dissertation has been approved by the

promotor: prof.dr. M. Veljkovic

promotor: prof.dr.ir. J. Maljaars

Composition of the doctoral committee:

Rector Magnificus,	chairperson
Prof.dr. M. Veljkovic,	Delft University of Technology
Prof.dr.ir. J. Maljaars,	Eindhoven University of Technology
	Dutch Organisation for Applied Scientific Research

Independent members:

Dr. J.A. Fonseca de Oliveira Correia,	University of Porto, Porto, Portugal
Prof.dr. M. al-Emrani,	Chalmers University of Technology, Gothenburg, Sweden
Dr.ir. R.J.M. Pijpers,	Dutch Organisation for Applied Scientific Research
Prof.dr.ir. L.J. Sluijs,	Delft University of Technology
Prof.dr. A. Taras,	Federal Institute of Technology Zurich, Zurich, Switzerland
Prof.dr. Z. Li,	Delft University of Technology, reserve member

Keywords: Orthotropic bridge decks; welded connections; hot spot stress; structural stress; local assessment methods; crack propagation; fatigue design; residual stress; fracture mechanics; 2⁺ stage model.

Printed by: Ipskamp Printing B.V.

Copyright © 2025 by W. Wu

ISBN/EAN: 978-94-6518-061-8

An electronic version of this dissertation is available at

<http://repository.tudelft.nl/>.

CONTENTS

Summary	ix
Samenvatting	xiii
1 Introduction	1
1.1 Background	1
1.2 High cycle fatigue in OBDs	4
1.2.1 S-N curve method	4
1.2.2 Fracture mechanics method	5
1.3 Limitations of the current methods	5
1.4 Research objectives	6
1.5 Research questions	6
1.6 Outline of the dissertation	6
2 Literature review on fatigue of welded connections (in OBDs)	11
2.1 Fatigue in steel	12
2.2 Welded connections	14
2.2.1 Geometric characteristics	14
2.2.2 Defects, misalignment, and distortion	16
2.2.3 Material properties and welding induced residual stress	17
2.3 High cycle fatigue assessment methods	19
2.3.1 S-N curve	19
2.3.2 Fatigue cracking in welded connections	20
2.3.3 Nominal stress	20
2.3.4 Hot spot stress	21
2.3.5 Effective notch stress.	22
2.3.6 Averaged strain energy density factors	24
2.3.7 Notch strain	24
2.3.8 Fracture mechanics	25
2.3.9 Discussion of the existing advanced fatigue analysis models.	28
2.4 Fatigue in OBDs.	30
2.4.1 Introduction of OBDs	30
2.4.2 Examples of the fatigue assessment for OBDs	33
2.4.3 Details and cracks	33
2.5 State of the art for the studied welded connections	35
2.5.1 Stiffener-to-deck plate welded connection.	35
2.5.2 Crossbeam-to-stiffener welded connection	37

3	Weld toe crack and weld root crack in stiffener of stiffener-to-deck plate welded connection between crossbeams (C2a & C2b)	39
3.1	Introduction	40
3.2	Experimental study	41
3.2.1	Specimen	41
3.2.2	Experimental programme	43
3.2.3	Stress calculation	45
3.2.4	Experimental results and fatigue resistances	46
3.2.5	Additional experimental results in the literature	50
3.3	Finite element modelling	51
3.4	Parametric analysis using local approaches	53
3.4.1	Effective notch stress method	53
3.4.2	Averaged strain energy density factor method	59
3.4.3	Fracture mechanics method	59
3.5	Comparison of the fatigue assessment methods	63
3.6	Summary of the chapter	65
4	Weld toe crack in deck plate of stiffener-to-deck plate welded connection between crossbeams (C1a)	67
4.1	Introduction	68
4.2	Fatigue resistance analysis	70
4.3	Development of geometric correction factors	72
4.3.1	Numerical model	72
4.3.2	Crack propagation calculation using FRANC3D	75
4.3.3	Hand calculation of fatigue crack propagation	77
4.3.4	Geometric correction factors for hand calculation	78
4.4	Probabilistic fracture mechanics analysis	89
4.5	Summary of the chapter	93
5	Weld root crack in deck plate of stiffener-to-deck plate welded connection at crossbeam (C1c)	95
5.1	Introduction	96
5.2	Experimental investigation	97
5.2.1	Specimen	97
5.2.2	Instrumentation	100
5.2.3	Fatigue loading programme	100
5.3	Experimental results	103
5.3.1	Fatigue crack characteristics	103
5.3.2	(Structural) hot spot stress at the initial loading stage	108
5.3.3	Evaluation of the experimental results	110
5.4	Finite element analysis	115
5.4.1	Description of the FE model	115
5.4.2	Model validation	116
5.4.3	Local stress state	116
5.5	Summary of the chapter	122

6	Weld root crack in crossbeam-to-stiffener welded connection without cut-out (C4d)	123
6.1	Introduction	124
6.2	Specimen, setup and loading programme.	125
6.3	Strain and stress distribution	126
6.3.1	Arrangement of strain gauges	126
6.3.2	Introduction of the FE model	127
6.3.3	Strain range distribution	128
6.4	Stress evaluation using the FE method	131
6.4.1	Stress in the weld throat	131
6.5	Fatigue crack characteristics	137
6.5.1	Strain gauge detection	137
6.5.2	Naked eye inspection	137
6.6	Fatigue life evaluation.	141
6.7	Fatigue resistance of detail C4d	143
6.7.1	Fatigue evaluation using various failure criteria and assessment methods.	143
6.7.2	Statistical analysis including the results in the literature	144
6.8	Summary of the chapter	146
7	Weld toe crack in stiffener at the lower end of crossbeam-to-stiffener welded connection with cut-out (C3a)	147
7.1	Introduction	148
7.2	Experimental investigation and FE model validation	150
7.2.1	Setup and loading programme.	150
7.2.2	Strain measurement	151
7.2.3	Introduction of the FE model	155
7.2.4	Validation of the FE calculation	157
7.2.5	Fatigue crack inspection	157
7.3	Fatigue resistance.	161
7.4	Influence of the load position on the hot spot stress	162
7.5	Effective notch stress assessment of fatigue resistance	166
7.6	Fatigue behaviour of detail C3a considering WRS.	170
7.6.1	Introduction of the FE model	170
7.6.2	Material properties in the FE model	171
7.6.3	Thermal simulation	171
7.6.4	Mechanical simulation.	176
7.6.5	Mechanical response under the cyclic loading including WRS	178
7.6.6	Stress and strain in cyclic mechanical loading	179
7.7	Fatigue resistance prediction considering WRS	182
7.7.1	Notch strain method.	182
7.7.2	Fatigue crack states analysed by ΔJ	183

7.8	Long crack propagation simulation using the XFEM without WRS	191
7.8.1	FE model for the long crack propagation.	191
7.8.2	Discontinuity modelling and calculation of stress intensity factors using the XFEM	192
7.8.3	Crack propagation simulation using the XFEM	195
7.8.4	Prediction of the long crack propagation for S5 S.	196
7.9	2 ⁺ stage model	199
7.10	Summary of the chapter	202
8	Conclusions and recommendations	205
8.1	Conclusions.	206
8.1.1	Fatigue resistances of the studied details.	206
8.1.2	Fatigue performance of the studied details.	210
8.2	Recommendations for future work	215
A	Statistical analysis procedure	217
B	Additional equations related to Chapter 4	219
C	Additional experiments	221
C.1	Additional cracks observed in the experiments for detail C1c	222
C.2	Fatigue crack propagation properties of the stiffener web.	223
D	Additional numerical analysis	225
D.1	Monte Carlo simulation with the bilinear propagation model for detail C1a	226
D.2	FE calculated stress distribution of S2 in Chapter 6	228
D.3	Comparison of the element type for detail C3a	229
D.4	Validation of the welding simulation	230
D.4.1	Introduction of the welding simulation	230
D.4.2	Results of the welding simulation	231
D.5	Validation of the J-integral calculation	232
D.5.1	Introduction of the FE model	232
D.5.2	Results of the FE calculation	233
	References	235
	Acknowledgements	255
	List of Publications	257
	Education activities	259
	Curriculum Vitæ	261

SUMMARY

Steel Orthotropic Bridge Decks (OBDs) are deck plates stiffened by open or closed stiffeners. Due to the high strength/weight ratio, this type of deck plate is a popular choice for long-span and movable bridges where dead weight is important. OBDs are subjected to many traffic load cycles during the service life. The stress ranges are relatively high, with the highest stress concentrations often at welded connections. Therefore, the design of welded connections is often governed by fatigue resistance. The discovery of fatigue cracks worldwide indicates that our knowledge of fatigue resistance design in OBDs is limited. For example, unexpected cracks were observed only after several years of use in the Severn Bridge in the United Kingdom (opened in 1966), the Van Brienenoord Bridge in the Netherlands (opened in 1990), and the Humen Pearl River Bridge in China (opened in 1997). With increasing traffic, higher fatigue resistance is required. The main challenges for infrastructure worldwide are the design of safe and economical new OBDs and the accurate assessment of existing ones. The current standards for fatigue design of bridges, EN 1993-1-9: 2005, AASHTO LRFD Bridge Design Specifications: 2020, and JSSC recommendations: 2012, primarily rely on the nominal stress method or the hot spot stress method. The problems with these methods are that the nominal stress in OBDs is not well defined and that the resistances (detail categories) for the hot spot stress method are based on standard plate connections rather than the details in OBDs.

This dissertation aims to provide a comprehensive explanation of the behaviour of critical welded connections in OBDs stiffened by continuous trapezoidal stiffeners based on experimental investigations and Finite Element Analysis (FEA). This investigation focuses on the most critical welded connections, specifically the welded connections between the stiffener and the deck plate, as well as the welded connections between the crossbeam and the stiffener, utilising the author's experiments and results from the literature. The studied details in this dissertation are summarised below:

Detail	Description
C2a	Stiffener-to-deck plate weld, weld toe crack in stiffener
C2b	Stiffener-to-deck plate weld, weld root crack in weld
C1a	Stiffener-to-deck plate weld, weld toe crack in deck plate
C1c	Stiffener-to-deck plate weld, weld root crack in deck plate at crossbeam
C4d	Crossbeam-to-stiffener weld at stiffener bottom, weld root crack
C3a	Crossbeam-to-stiffener weld, weld toe crack in stiffener at lower end of the weld

Illustrations of the cracks are shown on the cover page and in Figure 1.5.

The author carries out experimental investigations on a $5.1 \times 9.4 \text{ m}^2$ full-scale OBD specimen with a 20 mm thick deck plate and 16 mm thick webs of crossbeams for details C1c (Chapter 5), C4d (Chapter 6) and C3a (Chapter 7), and on nineteen $350 \times 200 \times 200 \text{ mm}^3$ small-scale stiffener-to-deck plate connections for details C2a and C2b (Chapter 3). The 20 mm thick deck plate is used because a thicker deck plate ($\geq 14 \text{ mm}$) is becoming common in newly designed OBDs as a response to cracks found in thinner deck plates (10 mm or 12 mm). Either automatic or manual welding is used for the deck plate welds in small- and full-scale specimens (Chapters 3 and 5).

Six corresponding detail categories are established, covering the deck plate thicknesses from 10 mm to 20 mm. Among them are two connection details: C1c (Chapter 5) and C3a (Chapter 7), which were not included in either the first or the second generation of EN 1993-1-9: Eurocode 3: Design of steel structures - Part 1-9: Fatigue. The surface extrapolation approach is used to calculate the hot spot stress for the details: C2a (Chapter 3), C1a (Chapter 4), C1c (Chapter 5), and C3a (Chapter 7). The force equilibrium (pair) approach is used to calculate the structural stress for detail C2b (Chapter 3) and the nominal stress for detail C4d (Chapter 6). Fatigue resistance is evaluated using multiple failure criteria for full-scale experiments instead of using the first visible crack as the main failure criterion. The fatigue resistances of the studied welded details are relatively high compared with the fatigue resistances in the standards. The main reasons for this are: a steep stress gradient towards the hot spot (Chapters 3, 4, 5, and 7), a possibility to redistribute loads from a weakened component to adjacent parts (Chapters 5, 6, and 7), connections loaded in cyclic compression or out-of-plane bending or in a combination of the two instead of cyclic axial tension (Chapters 3 to 7), a relatively thin plate thickness of stiffener (Chapters 3 and 7), and strict requirements for the geometry of the weld between stiffener and deck plate (Chapters 3, 4, and 5). Additionally, automatic welding shows a higher and more consistent fatigue performance for details C2a and C2b (Chapter 3).

Finite element models are built using the commercial software Abaqus. The effective notch stress, averaged strain energy density factors, notch strain and fracture mechanics are used to account for the effects of penetration depth, load ratio, initial flaw, residual stress and weld profile on the fatigue behaviour of the welded connections. The Linear Elastic Fracture Mechanics (LEFM) gives good predictions for the fatigue resistances of both the weld toe (detail C2a) and the weld root (detail C2b). A LEFM hand calculation model is proposed for C1a with three different weld profiles (Chapter 4). The surface crack propagation of C1a calculated by the proposed model, together with the Paris' equation, is validated against the experiment in the literature. Probabilistic fracture mechanics analysis for detail C1a is carried out using the same method. The calculation predicts the fatigue resistances well compared with the values obtained from the experiments in the literature. The eXtended Finite Element (XFE) method is used to study the crack propagation of C1c within the LEFM framework (Chapter 5). The crack arrest and the crack path of C1c for 10 mm, 12 mm, 16 mm, and 20 mm thick deck plates are correctly predicted (validated against experimental investigations).

The residual stress due to welding is numerically studied using Abaqus for detail C3a (Chapter 7), which is further used in the fatigue analysis using the Notch Strain (NSA) and the Elastic-Plastic Fracture Mechanics (EPFM) for the fatigue initiation and

the short crack propagation, respectively. An engineering framework is proposed for the three-dimensional fatigue crack propagation using the LEFM. The crack propagation is predicted using the XFE algorithm in Abaqus, which successfully predicts the long crack propagation of C3a. The fatigue initiation stage and the crack propagation stage in the available 2 stage model are analysed by the NSA and the LEFM, respectively. The author bridges the gap between the fatigue initiation and the long crack propagation by using the EPFM. The 2⁺ stage model is therefore proposed by the author, which provides a full range fatigue analysis of metallic structures with a specific focus on welded connections (Chapter 7). The proposed model realistically considers the geometric characteristics and different material properties (*e.g.* heat affected zone) at the initiation location, and quantitatively implements the welding-induced residual stress obtained by FEA.

Recommendations are given for the fatigue design of the details based on this study, which were used as input for the technical specification “TS 1993-1-901 — Fatigue design of orthotropic bridge decks with the hot spot stress method” as part of the second generation of Eurocodes.

SAMENVATTING

Stalen orthotrope brugdekken (OBD's) bestaan uit dekplaten die zijn ondersteund door open of gesloten verstijvers. Door de hoge sterkte/gewichtsverhouding is dit type brugdek een populaire keuze voor bruggen met een grote overspanning en beweegbare bruggen waar het eigen gewicht belangrijk is. OBD's worden tijdens de levensduur blootgesteld aan vele cycli van verkeersbelasting. De spanningsintervallen zijn relatief hoog, met de hoogste spanningsconcentraties vaak bij lasverbindingen. Daarom is de vermoeiingsweerstand vaak maatgevend van gelaste verbindingen. De ontdekking van vermoeiingsscheuren wereldwijd geeft aan dat onze kennis van het ontwerp van vermoeiingsbestendigheid in OBD's beperkt is. Zo werden scheuren na enkele jaren gebruik waargenomen in de Severn Bridge in het Verenigd Koninkrijk (geopend in 1966), de Van Brienenoordbrug in Nederland (geopend in 1990), en de Humen Pearl River Bridge in China (geopend in 1997). Met toenemend verkeer, is een hogere weerstand voor vermoeiing vereist. Eén van de belangrijkste uitdagingen voor de infrastructuur wereldwijd zijn het ontwerp van veilige en economisch optimale, nieuwe OBD's en de nauwkeurige beoordeling van bestaande OBD's. De huidige normen voor vermoeiingsontwerp van bruggen, EN 1993-1-9: 2005, AASHTO LRFD Bridge Design Specifications: 2020 en JSSC-aanbevelingen: 2012, zijn voornamelijk gebaseerd op de nominale spanningsmethode of de hot spot spanningsmethode. De problemen met deze methoden zijn dat de nominale spanning in OBD's niet goed gedefinieerd is en dat de weerstanden (detailcategorieën) voor de hot spot spanningsmethode gebaseerd zijn op andere standaard plaatverbindingen in plaats van de details in OBD's.

Dit proefschrift heeft als doel het gedrag van kritische lasverbindingen in OBD's met doorlopende trapeziumvormige verstijvers uitgebreid te verklaren op basis van experimenteel onderzoek en eindige-elementenanalyses. Dit onderzoek richt zich op de meest kritische lasverbindingen: de verbindingen tussen verstijver en dekplaat en de verbinding tussen dwarsdrager en verstijver, op basis van experimenten van de auteur en resultaten uit de literatuur. De bestudeerde details in dit proefschrift zijn hieronder samengevat:

Detail	Beschrijving
C2a	Lasteenscheur aan de zijde van de verstijver van de las tussen verstijver en dekplaat
C2b	Laswortelscheur in de las tussen verstijver en dekplaat
C1a	Dekplaatscheur vanuit de lasteen van de las met de verstijver tussen twee dwarsdragers
C1c	Dekplaatscheur vanuit de wortel van de las met de verstijver ter plaatse van de dwarsdrager
C4d	Laswortelscheur vanuit de dubbelzijdige hoeklas tussen dwarsdrager en verstijver
C3a	Scheur vanuit de onderkant van de las tussen verstijver en dwarsdrager met uitsparing van de las

Illustraties van de scheuren zijn weergegeven op het voorblad en in Figuur 1.5. De auteur voerde experimenteel onderzoek uit aan een OBD-proefstuk op ware grootte van $5.1 \times 9.4 \text{ m}^2$ met een dekplaatdikte van 20 mm en een lijfplaatdikte van de dwarsdragers van 16 mm voor de details C1c (Hoofdstuk 5), C4d (Hoofdstuk 6) en C3a (Hoofdstuk 7), en aan negentien kleinschalige verbindingen tussen verstijver en dekplaat van $350 \times 200 \times 200 \text{ mm}^3$ voor de details C2a en C2b (Hoofdstuk 3). De 20 mm dikke dekplaat wordt gebruikt omdat een dikkere dekplaat ($\geq 14 \text{ mm}$) steeds gebruikelijker wordt in nieuw ontworpen OBD's als gevolg van scheuren in dunnere dekplaten (10 mm of 12 mm). Voor de dekplaatlassen in kleine en volledige samples is automatisch of handmatig lassen gebruikt (Hoofdstukken 3 en 5).

Er zijn zes overeenkomstige detailcategorieën vastgesteld, binnen de dekplaatdiktes van 10 mm tot 20 mm. Daaronder bevinden zich twee verbindingsdetails: C1c (Hoofdstuk 5) en C3a (Hoofdstuk 7), die noch in de eerste noch in de tweede generatie van EN 1993-1-9 waren opgenomen: Eurocode 3: Design of steel structures - Part 1-9: Fatigue. De oppervlakte-extrapolatiebenadering is gebruikt om de hot spot spanning te berekenen voor de details: C2a (Hoofdstuk 3), C1a (Hoofdstuk 4), C1c (Hoofdstuk 5) en C3a (Hoofdstuk 7). De krachtevenwichtsbenadering (paar) is gebruikt om de constructieve spanning voor detail C2b (Hoofdstuk 3) en de nominale spanning voor detail C4d (Hoofdstuk 6). De vermoeiingsweerstand van alle details is geëvalueerd aan de hand van meerdere faalcriteria in plaats van de eerste zichtbare scheur als het belangrijkste faalcriterium te gebruiken. De vermoeiingsweerstand van de bestudeerde gelaste details zijn relatief hoog in vergelijking met de vermoeiingsweerstand in de normen. De belangrijkste redenen hiervoor zijn: een steile spanningsgradiënt naar de hot spot (Hoofdstukken 3, 4, 5, en 7), een mogelijkheid om belastingen van een verzwakt onderdeel naar aangrenzende onderdelen te herverdelen (Hoofdstukken 5, 6, en 7), verbindingen die worden belast in cyclische druk of buiging uit het vlak of een combinatie van beide in plaats van cyclische axiale spanning (Hoofdstukken 3 tot 7), een relatief dunne plaatdikte van de verstijver (Hoofdstukken 3 en 7), en strikte eisen aan de geometrie van de las tussen verstijver en dekplaat (Hoofdstukken 3, 4, en 5). Bovendien vertoont automatisch lassen een hogere en consistentere vermoeiingsprestatie voor details C2a en C2b (Hoofdstuk 3).

Eindige-elementenmodellen zijn opgesteld met behulp van de commerciële software Abaqus. The effective notch stress, averaged strain energy density factors, notch strain en breukmechanica zijn gebruikt om rekening te houden met de effecten van inbrandingsdiepte, belastingsratio, initiele defect, restspanning en lasprofiel op het vermoeiingsgedrag van de gelaste verbindingen. De Linear Elastic Fracture Mechanics (LEFM) geeft goede voorspellingen voor de vermoeiingsweerstand van zowel de lassteen (detail C2a) als de laswortel (detail C2b). Voor C1a wordt een LEFM-handrekenmodel voorgesteld met drie verschillende lasprofielen (Hoofdstuk 4). De voortplanting van oppervlaktescheuren van C1a berekend door het voorgestelde model, samen met de vergelijking van Paris, is gevalideerd aan de hand van het experiment in de literatuur. Probabilistische breukmechanica analyse voor detail C1a is uitgevoerd met behulp van dezelfde methode. De berekening voorspelt de vermoeiingsweerstand goed in vergelijking met de waarden die zijn verkregen uit de experimenten in de literatuur. De eXtended Finite Element (XFE) methode wordt gebruikt om de scheurgroei van C1c binnen het LEFM-raamwerk te bestuderen (Hoofdstuk 5). Het stoppen van de scheurgroei en het scheurpad van C1c voor 10 mm, 12 mm, 16 mm en 20 mm dikke dekplaten zijn correct voorspeld (gevalideerd aan de hand van experimenteel onderzoek).

De door het lassen gevormde restspanningen zijn numeriek bestudeerd met behulp van Abaqus voor detail C3a (Hoofdstuk 7), dat verder is gebruikt in de vermoeiingsanalyse met behulp van de Notch StrAin (NSA) en de Elastic-Plastic Fracture Mechanics (EPFM) voor respectievelijk de vermoeiingsinitiatie en de groei van korte scheuren. Er is een beoordelingsraamwerk voorgesteld voor de driedimensionale groei van vermoeiingsscheuren met behulp van de LEFM. De scheurgroei is voorspeld met behulp van het XFE-algoritme in Abaqus, dat met succes de lange scheurgroei van C3a voorspelt. De vermoeiingsinitiatiefase en de scheurgroeifase in het beschikbare 2-fasenmodel is geanalyseerd door respectievelijk de NSA en de LEFM. De auteur overbrugt de kloof tussen de vermoeiingsinitiatie en de groei van de lange scheur met behulp van de EPFM. Het 2⁺ fasenmodel is daarom voorgesteld door de auteur, dat een volledige analyse van vermoeiing van metalen constructies met een specifieke focus op gelaste verbindingen (Hoofdstuk 7). Het voorgestelde model houdt op realistische wijze rekening met de geometrische kenmerken en verschillende materiaaleigenschappen (*bijv.* warmte beïnvloede zone) op de initiatielocatie, en implementeert kwantitatief de door de eindige-elementenmethode verkregen restspanningen van het lassen.

Op basis van deze studie zijn aanbevelingen gedaan voor het vermoeiingsontwerp van de details, die zijn gebruikt als input voor de technische specificatie “TS 1993-1-901 — Vermoeiingsontwerp van orthotrope brugdekken met de hot spot spanningsmethode” als onderdeel van de tweede generatie Eurocodes.

To my family

1

INTRODUCTION

1.1. BACKGROUND

The main characteristics of steel Orthotropic Bridge Decks (OBDs) are rapid construction and low self-weight, which make them popular for superstructures in movable and long-span bridges [1, 2], see Figure 1.1 for three examples of iconic long-span bridges with OBDs. It is a popular redecking option because dead weight can be reduced and erection is efficient [3, 4]. An excellent example of using OBDs to upgrade existing concrete bridge decks was the Golden Gate Bridge [4, 5], where the replacement was completed in 1985. Moreover, the deck plate can be built at any time in cold climates, unlike concrete, which typically requires construction temperatures above 5 °C [6]. Figure 1.2 illustrates the commonly used designs: The deck plate is stiffened by either open or closed stiffeners. The former may contain a bulb at the bottom of the stiffener and the latter can be trapezoidal or U- or V-shaped. A cut-out (or cut-outs) at the lower part of a stiffener may be applied for the convenience of fabrication [1, 2]. Fatigue cracking in OBDs was reported in up to 0.2% of the total welds in France [7], and in approximately one-fifth of welds in the Severn Bridge in the United Kingdom [8]. However, no major problems were reported due to cracking or corrosion of OBDs in the United States and Canada even after 25 years of use [9]. The use of OBDs is governed by fatigue performance considerations. The deck plate is exposed to various types of traffic loading during the service life. High stress concentrations commonly occur at welded connections where detrimental effects, *e.g.* initial flaws, high residual stresses and misalignments, adversely affect the fatigue performance. Fatigue cracks are most likely to occur at critical locations in welded connections. With increases in traffic flow and gross vehicle weight, thicker deck plates have become a popular option for new designs [10–12]. The OBD is a bridge deck system expected to provide a 100-year life when properly designed, constructed, and maintained [5, 13]. One specific requirement given in [13] is that the deck plate thickness should be at least 16 mm. However, the OBDs designed in the 20th century are commonly made with a thinner deck plate of 10 or 12 mm [1, 14]. This is the main motivation for improving knowledge and prediction methods for the fatigue

assessment of “modern” OBDs. The main focus of the experimental study in this dissertation is on modern orthotropic bridge decks. One $5.1 \times 9.4 \text{ m}^2$ full-scale OBD specimen and nineteen $350 \times 200 \times 200 \text{ mm}^3$ small-scale stiffener-to-deck plate connections, both with a 20 mm thick deck plate, are tested by the author. Figure 1.3 shows the full-scale specimen and one setup used in the experiments, with the bottom flanges fully supported.

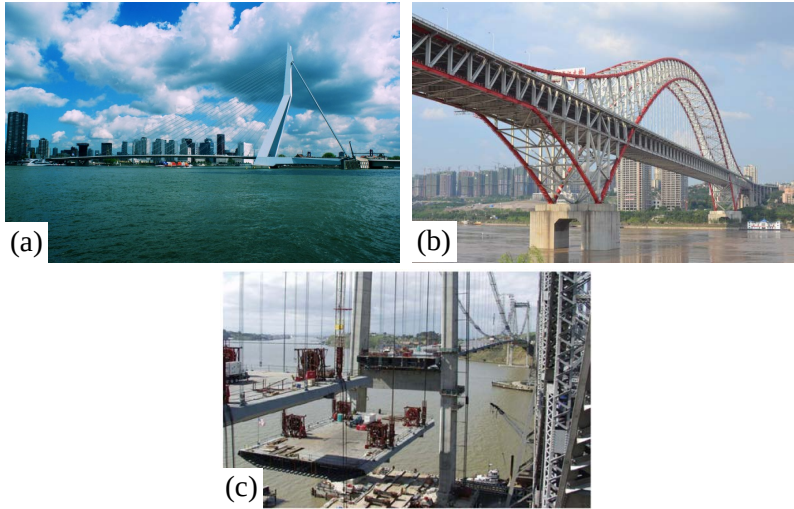


Figure 1.1: Examples of OBDs worldwide: (a) Erasmus Bridge in Rotterdam, the Netherlands (picture adapted from an online source), (b) Chaotianmen Bridge in Chongqing, China (picture adapted from an online source), (c) Erection of Orthotropic Box Girder Segment for the Alfred Zampa Memorial Bridge, the United States [2].

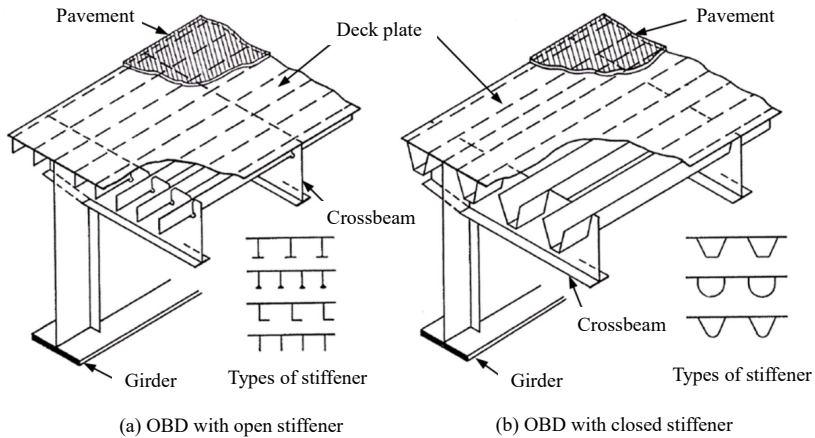


Figure 1.2: Illustration of typical OBD designs [1, 2].

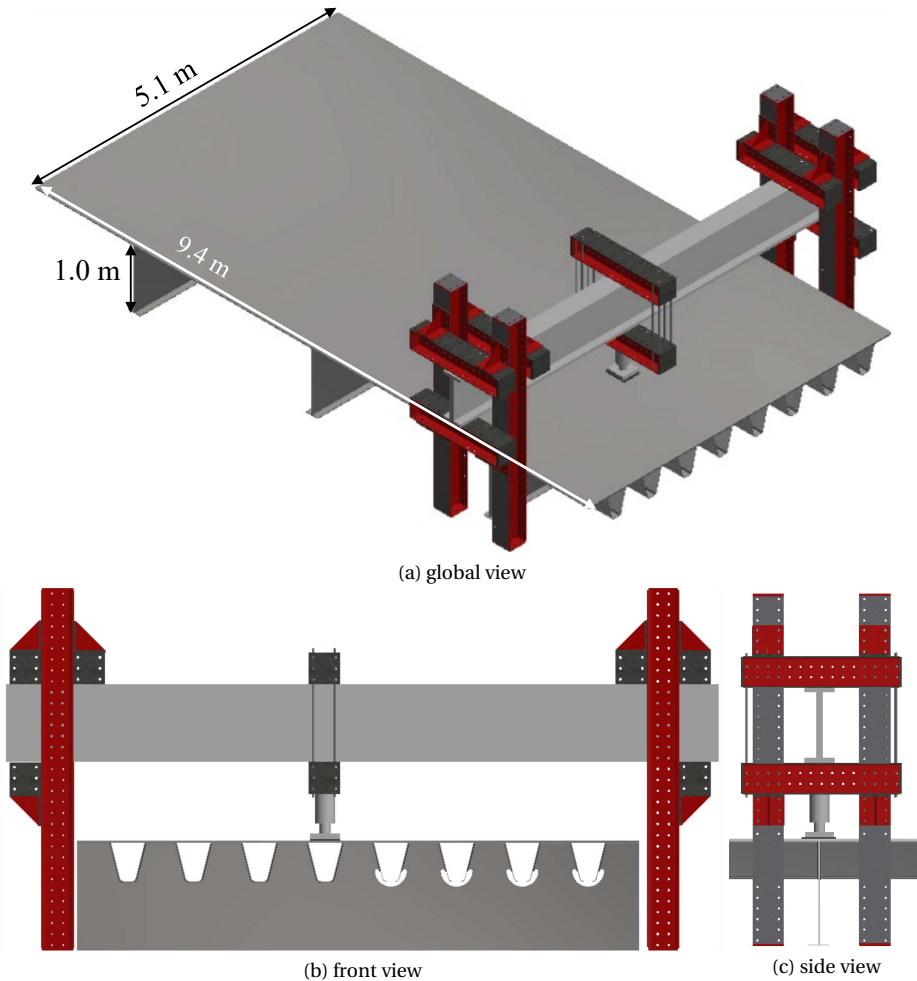


Figure 1.3: Sketch of the full-scale specimen and one setup with the bottom flanges fully supported.

With the advances in computational capability and the availability of Finite Element (FE) software, FE modelling is expected to be used in structural analysis. Reasonably accurate stress (strain) range predictions of an engineering FE model were found at critical locations when compared with the results of in situ strain measurements [15]. This provides an opportunity to use powerful FE models to analyse OBDs. This is of particular importance for fatigue analysis of welded connections where the fatigue performance is typically dominated by local stress states at crack initiation locations. Moreover, fatigue crack propagation speed can be analysed using fracture mechanics in conjunction with the FE method. Mixed-dimensional modelling, *e.g.* three-dimensional local models in conjunction with one-dimensional models, can be used to strike a balance between the accuracy and the computational cost. An increase from the thin deck plate (10 or 12 mm) to the thick deck plate (≥ 14 mm) is demonstrated by the practice necessary for modern

OBDs. In Germany, the minimum required deck plate thickness has increased from 12 mm to 14 mm since 2003 [10]. A comprehensive experimental study and assessment using global or local methods in conjunction with the FE method is essential to improve the design and maintenance accuracy. The following sections of this chapter present the existing methodology considering two types of methods for high cycle fatigue. This dissertation presents originally designed (full- or small-scale) experiments and validates the current methodology for the fatigue assessment of OBDs using the author's own experiments and results of other researchers. The corresponding gap between the problem and the current methods, research objectives, and research questions will be discussed. Finally, the outline of the dissertation, a summary of the studied details, and an overview of the conducted experiments are presented.

1.2. HIGH CYCLE FATIGUE IN OBDs

1.2.1. S-N CURVE METHOD

The S-N curve method is commonly used for high cycle fatigue design. The Basquin relationship is applied to the number of cycles to failure N_R as a function of the applied stress range $\Delta\sigma$, see Equation (1.1). The parameters m and a depend on the detail and failure mode. The characteristic value, $\Delta\sigma_C$ used in the European standard (pr)EN 1993-1-9 [16, 17], is linked to a and is defined as the stress range associated with the number of cycles that has a 95% exceedance level at $N_C = 2 \times 10^6$ assuming a fixed slope parameter $m = 3$ or 5 (fully informative) when $\Delta\sigma$ is larger than the fatigue limit. The values of a and $\Delta\sigma_C$ are obtained by the statistical analysis of experimental results.

$$\log N_R = a - m \cdot \log \Delta\sigma \quad (1.1)$$

The Nominal Stress (NS) is calculated using the forces (*i.e.* axial force or bending moment) divided by the cross section characteristics (*i.e.* area or section modulus). Due to its ease of use, this method is the basis for fatigue design in the widely used bridge guidelines:

(pr)EN 1993-1-9 [16, 17], JSSC recommendations [18], and AASHTO LRFD Bridge Design Specifications [19]. The stress in the cross section is assumed to be uniformly distributed, excluding all stress concentrations. Alternatively, the well-known (structural) Hot Spot Stress (HSS) method includes the bending and membrane stresses in the cross section while excluding the non-linear stress concentration at the weld toe [16–19]. The local stress method, Effective Notch Stress (ENS), applies a fictitious notch radius, $R_{\text{ref}} = 1$ or 0.05 mm, at the crack initiation location. Table 1.1 summarises $\Delta\sigma_C$ for the weld Toe (T) or the weld Root (R) recommended in the commonly used bridge guidelines.

The Notch StrAin (NSA) method evaluates the local strain behaviour at the crack initiation location using the elastic-plastic constitutive model. The method is adopted in the FKM guideline for the fatigue evaluation of metal components [20]. The NSA method becomes attractive when the stress levels are high and the plastic behaviour cannot be ignored.

In addition to the stress or strain methods, the averaged Strain Energy Density (SED) factor is a commonly used energy-based local method. It uses the averaged SED within a radius $R_C = 0.28$ mm with $m = 1.5$ and $\Delta W_C = 0.058$ (unit: N·mm/mm³) for steel [21].

Table 1.1: $\Delta\sigma_C$ of welded steel plate connections in the widely used bridge guidelines.

Source	Method	Application	m	$\Delta\sigma_C$ [MPa]
[17–19]	NS	T & R	3 or 5	36 to 125
[17–19]	HSS	T	3	80 [18] or 90 [17, 19] or 100 [17, 18]
[17]	ENS	T & R	3	200 or 225

1.2.2. FRACTURE MECHANICS METHOD

The S-N curve method only gives the fatigue resistance of the detail. Fracture mechanics becomes attractive as the fatigue crack propagation history can be calculated using the Paris' equation [22], see Equation (1.2). It is of special interest for welded connections as fatigue propagation is a major contributor to the total fatigue life.

$$\frac{da}{dN} = C \cdot (\Delta K)^m \quad (1.2)$$

Where ΔK is the Stress Intensity Factor (SIF) range at the crack tip, calculated by the Linear Elastic Fracture Mechanics (LEFM). The LEFM evaluates the states at the crack tip (SIF) using either analytical solutions or numerical methods [23–25]. Material parameters C and m are obtained from experiments. This method was initially developed for long crack analysis and it has been applied to fatigue resistance by assuming an initial crack at the studied location [23, 24].

1.3. LIMITATIONS OF THE CURRENT METHODS

The NS method is the dominant fatigue design method [16–19]. However, it is shown that it may be suitable only for a few fatigue details present in OBDs. The stress states of OBDs are complicated and the application of NS is challenging. The HSS method, which contains geometric characteristics, shows larger potential in the fatigue design of the welded connections in OBDs. However, only the results of standard plate connections and hollow sections have been analysed in the development of fatigue resistance curves for HSS in [16–19, 23, 26]. The HSS fatigue resistance curves for OBDs were still missing.

The S-N curves can be used in the prediction of fatigue lifetime based on the HSS. Currently, the standardised detail category: 80 MPa [18] or 90 MPa [16, 17, 19] or 100 MPa [16–18] is used for welded plate connections in the as-welded condition with limited information for the end-of-life. The influence of notch radius, initial flaw, residual stress, *etc.* may be evaluated using the local methods or the methods based on fracture mechanics. Similar to the HSS, these methods are developed by standard specimens. However, the requirements in construction, the plate thickness, and the applied load types affect the fatigue performance of a welded connection. Using the standardised category cannot accurately predict the fatigue resistance of different types of details. Validation of the local methods for their application in OBDs is necessary to fully establish this methodology in fatigue assessment.

Implementation of residual stress and elastic-plastic material properties in FE simulations gives a more realistic material response under cyclic loading. The initiation and the propagation with large plasticity can be studied using the NSA and the Elastic-Plastic

Fracture Mechanics (EPFM), respectively. However, applications of NSA or EPFM on OBDs are limited.

1.4. RESEARCH OBJECTIVES

The main research objective of this dissertation is driven by the need to improve the knowledge for optimal design and maintenance assessment. Fatigue assessment of welded connections in OBDs should be carried out using at least two research methods. This is the main principle applied in this dissertation, which influences the structure of the dissertation.

The knowledge gained is based on six welded details in modern OBDs which are selected to study the following sub-objectives:

- O1. Explain the mechanism of fatigue cracking of welded connections in OBDs.
- O2. Establish fatigue resistance curves using either the HSS or the NS method based on the author's own experiments and results in the literature.
- O3. Provide principles for economic design and maintenance.
- O4. Assess the fatigue performance using the local methods ENS, SED, and LEFM.
- O5. Demonstrate the influence of residual stress on fatigue by FE analysis using the methods NSA, LEFM, and EPFM.

1.5. RESEARCH QUESTIONS

The following research questions are considered in this dissertation:

- Q1. What are the fatigue resistance curves of welded connections based on the HSS in OBDs? Furthermore, what is the reason why they differ from the standardised fatigue detail category of 100 MPa (weld toe crack that propagates into the base metal) or 40 MPa (weld root crack that propagates into the weld throat)?
- Q2. What could explain the difference in fatigue behaviour between a small-scale specimen and a full-scale bridge deck?
- Q3. What is the influence of failure criteria on fatigue resistance curves?
- Q4. Can the fatigue resistance of details in OBDs be predicted by the local methods ENS, SED, and LEFM?

1.6. OUTLINE OF THE DISSERTATION

Figure 1.4 outlines the studied details, research questions, and employed methods in the dissertation. Detail categories for either the HSS or the NS (for the root crack in fillet-welded connection that propagates into the weld throat) are proposed for the studied details. Different criteria are used in the statistical analysis for the studied details. The designation of the details follows prTS 1993-1-901 [27] (in short TS) with illustrations summarised in Figure 1.5. Due to the urgency of providing the background of TS to

readers worldwide, journal papers have been published before the finalisation of the dissertation. A brief summary of the dissertation's structure is given below:

- **Chapter 1** briefly introduces the background, related research and design methods, knowledge gap between the problem and the current methods, research objectives, and research questions in this dissertation. The outline of the dissertation, the studied details and the overview of the experiments are presented.
- **Chapter 2** introduces the basic terms and the main concepts of fatigue analysis in welded connections. The material characteristics and stress states at welded connections are presented. The S-N curve and fracture mechanics methods are further explained. The NS, HSS, ENS, SED, NSA, LEFM, and EPFM methods are discussed. The history of OBDs, the fatigue design philosophy, and the state of the art for the studied details are given.
- **Chapters 3 to 5** analyse the fatigue behaviour of stiffener-to-deck plate welded connection between crossbeams or at crossbeam. Specifically, the weld toe crack that propagates into the stiffener and the weld root crack that propagates into the weld throat are studied in Chapter 3, the weld toe crack that propagates into the deck plate between crossbeams is studied in Chapter 4, and the weld root crack that propagates into the deck plate at crossbeam is studied in Chapter 5. The main part of the work has been published in the journal papers as follows:
 - **J1:** *Fatigue behaviour of toe and root stiffener cracks in stiffener-to-deck plate weld of orthotropic bridge decks*, [Engineering Structures](#), **305**, (2024);
 - **J2:** *Fatigue resistance of rib-to-deck welded joint in OSDs, analyzed by fracture mechanics*, [Journal of Constructional Steel Research](#), **162**, (2019);
 - **J3:** *Fatigue behaviour of root crack in stiffener-to-deck plate weld at crossbeam of orthotropic bridge decks*, [Engineering Structures](#), **306**, (2024).
- **Chapter 6** studies the root crack in crossbeam-to-stiffener welded connection with-out cut-out. Fatigue cracking process for the detail is classified into four stages. The nominal weld stress is used to establish the detail category. The structural weld stress and the ENS are used to evaluate the stress state at the weld root.
- **Chapter 7** studies the fatigue crack initiation and propagation at the lower end of crossbeam-to-stiffener welded connection with cut-out. First, experimental investigations are carried out using the full-scale specimen. The HSS and the ENS are used for the fatigue resistance curve development and influence surface analysis, and the weld profile assessment, respectively. Second, the welding procedure and the mechanical material response are simulated by the FE method. The NSA or the EPFM is applied after implementing the FE calculated residual stress. The first part of the work has been published in the journal paper as follows:
 - **J4:** *Fatigue behaviour of toe crack at the lower end of crossbeam-to-stiffener weld with cut-outs in OBDs*, [Journal of Constructional Steel Research](#), **222**, (2024).
- **Chapter 8** gives the conclusions and recommendations based on the study.

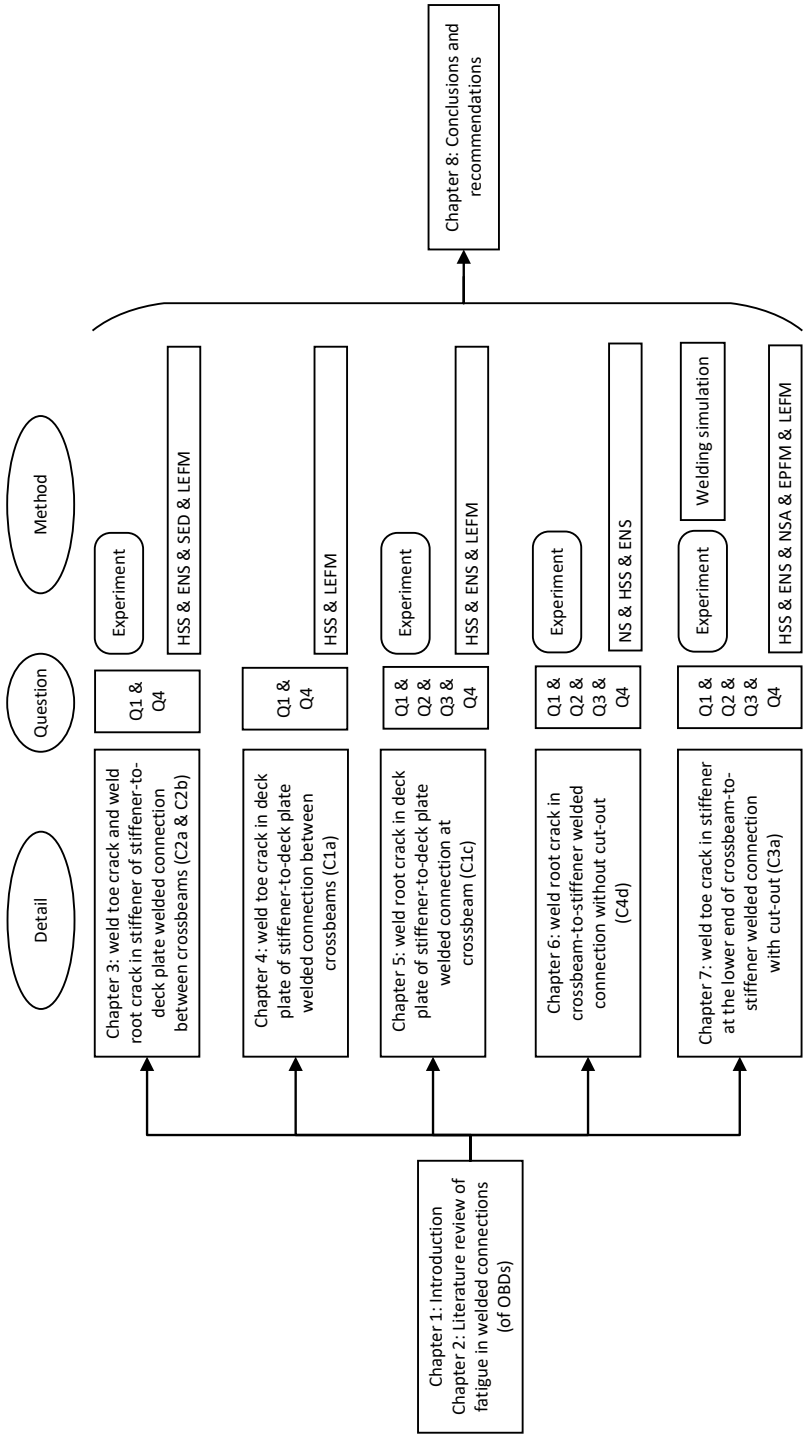


Figure 1.4: Outline of the dissertation (construction detail designation follows TS [27]).

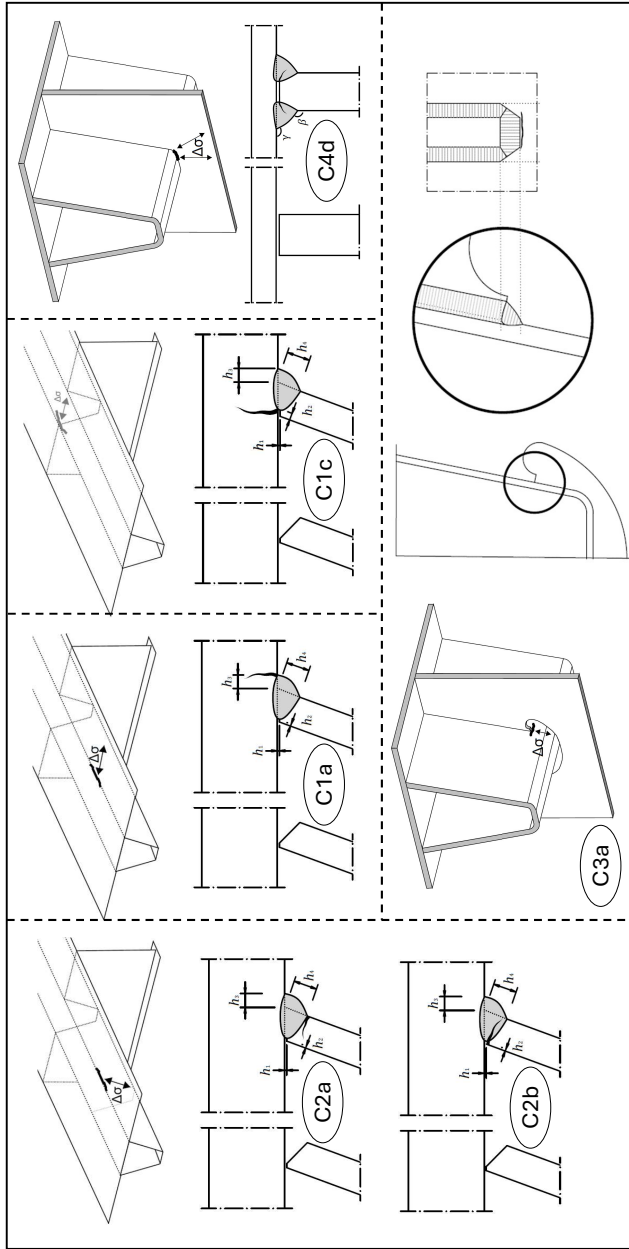


Figure 1.5: Summary of the construction details studied in this dissertation (designation follows TS [27]).

Figure 1.6 illustrates the small-scale experiments (details C2a and C2b) and full-scale experiments (details C1c, C4d, and C3a) carried out by the author for the studied details. Cyclic loads are applied at the free edge of the stiffener with the back surface of the deck plate fully supported in the small-scale experiments, Figure 1.6a. Full-scale experiments are carried out at different regions of the full-scale specimen with the lower flange of the three crossbeams fully supported, Figure 1.6b. The Crossbeam III region is loaded in fatigue stiffener-by-stiffener to study detail C1c with load centred above the stiffeners, see the blue patterns. The Crossbeam I region is afterwards loaded in fatigue to study details C4d and C3a. Cyclic loads are applied stiffener-by-stiffener from stiffener 1 to stiffener 4 to study detail C4d, see the orange patterns. The load is applied on the free edge side of Crossbeam I, centred above the studied

stiffener. Stiffeners 5 to 8 are loaded stiffener-by-stiffener to study detail C3a with two loads applied simultaneously on the free edge and the in span sides of Crossbeam I, centred above a stiffener web, see the purple patterns. Note that Crossbeam II region is loaded in static to study the stress (strain) distribution of OBDs in which the lower flange of Crossbeam II is simply supported to consider the in-plane deformation of the crossbeam. Experiments for the Crossbeam II region are outside the scope of this dissertation and will not be further described.

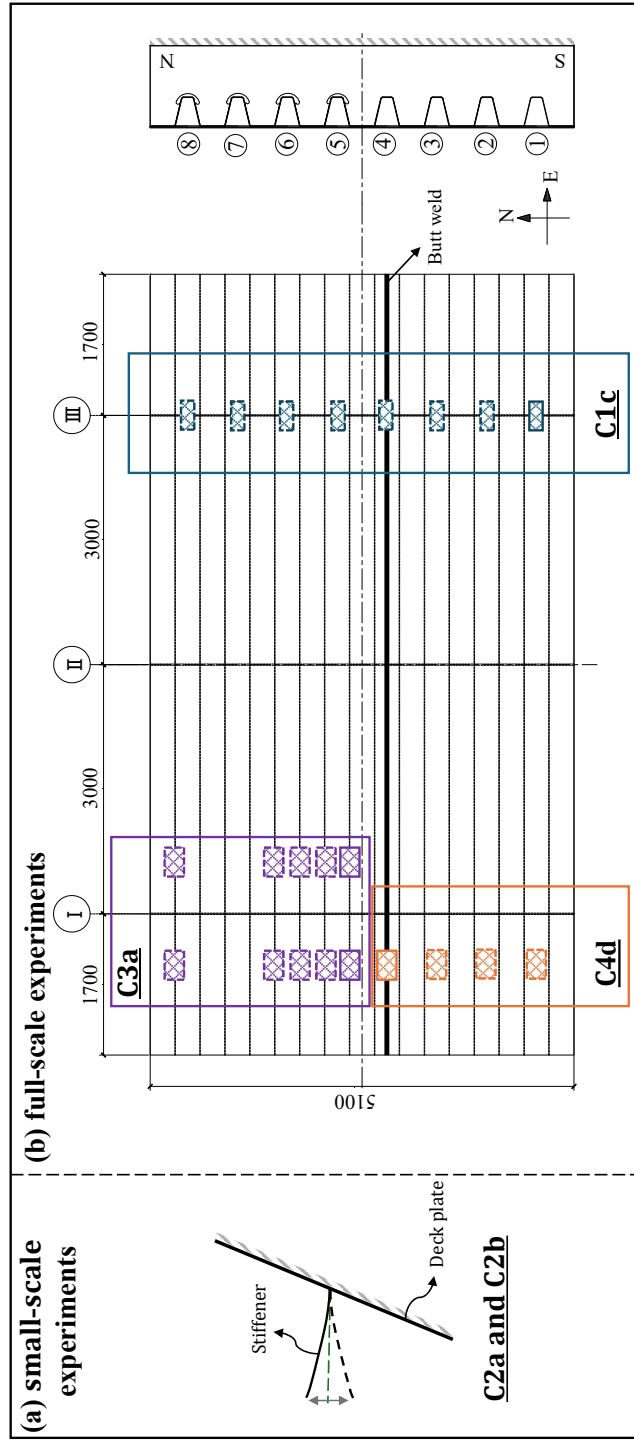


Figure 1.6: Illustration of the small-scale and the full-scale experiments carried out for the studied details by the author and reported in this dissertation (unit: mm).

2

LITERATURE REVIEW ON FATIGUE OF WELDED CONNECTIONS (IN OBDs)

Fatigue is one of the commonly seen phenomena that affect structural safety. A comprehensive discussion on the basic mechanism of fatigue in steel is given in this chapter with a special focus on welded connections. The corresponding nominal stress and local assessment methods for the constant or variable amplitude loading conditions are considered. Orthotropic bridge decks are briefly introduced. The state of the art for fatigue of the studied welded connections in orthotropic bridge decks is given.

2.1. FATIGUE IN STEEL

Fatigue is a phenomenon: *The process of progressive localised permanent structural change occurring in a material subjected to conditions that produce fluctuating stresses and strains at some point or points and that may culminate in cracks or complete fracture after a sufficient number of fluctuations* [28].

Fatigue crack nucleation is localised persistent damage in materials caused by repeated deformation under mechanical or environmental loading [28]. It starts with the dislocation movement of atoms at high strain locations. A crack occurs when the equilibrium of atoms is broken and causes a persistent change in the localised region [29]. After the nucleation of a crack, crack propagation can be divided into two stages: the short crack stage (micrometre range) and the long crack stage (millimetre range), see Figures 2.1 and 2.2 [30, 31]. In the short crack stage, persistent slip bands occur mainly due to shear stress [32]. With the increase of propagation (normally after the damage of several grains or interactions between grains), the direction changes and becomes perpendicular to normal stress in the long crack stage. Note that the average grain size of the widely used mild strength structural steel grade S355 is approximately 4 to 10 micrometres, whereas weld metals have smaller grain sizes (approximately 1 to 3 micrometres for ferritic steel weld metals) [33]. The technically or engineering defined crack initiation period normally consists of the short crack stage and the early status of long crack stage as they are neither easy to detect nor do they affect the mechanical performance. With the crack growth, the propagation rate increases and the local stiffness changes. This is normally designated as the technical (engineering) crack propagation period. Finally, fracture may happen when cracks propagate rapidly or unstably.

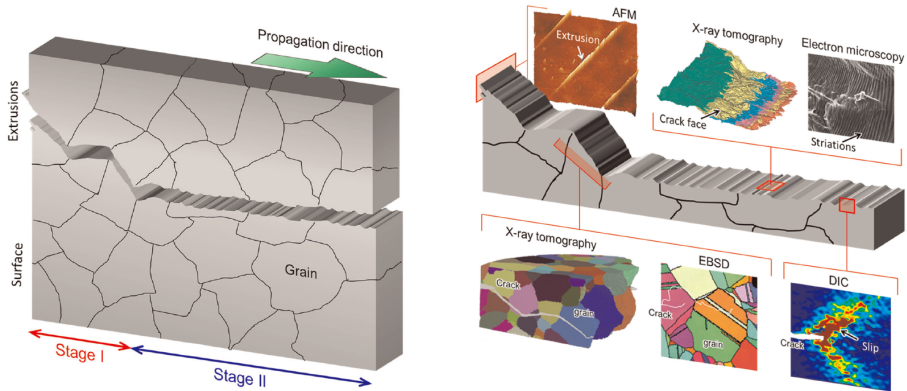


Figure 2.1: Illustration of the fatigue process in steel [30, 31].

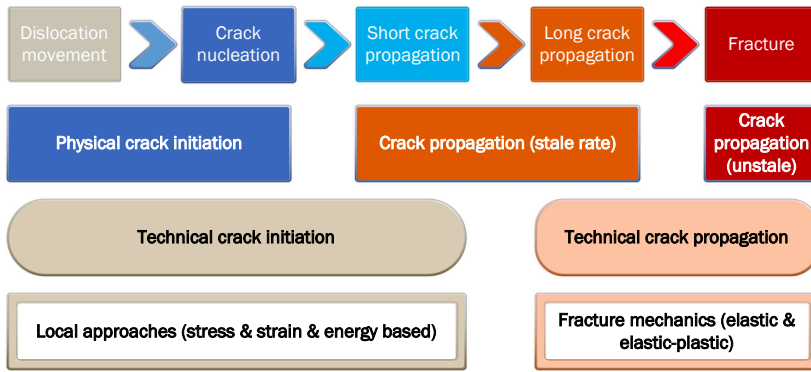


Figure 2.2: Illustration of the fatigue initiation and propagation periods in steel [31, 34–36].

Fatigue failures may occur with hundreds or thousands of cycles (low cycle), or tens of thousands up to millions of cycles (high cycle). The elastic stress-based method is used if the majority of material remains in the elastic region. Plasticity only exists in small regions at notches or crack tips. The S-N curve or the Paris' equation is commonly used in conjunction with the “driving parameter” calculated by the elastic stress-based method. The S-N curve uses the nominal stress, geometric stress, (effective) notch stress, and (averaged) notch energy in conjunction with their fatigue resistances obtained by experiments. The Paris' equation uses the stress intensity factors in conjunction with the material properties in the propagation calculation. If plasticity occurs in a large region, the elastic stress methods are insufficient to describe the mechanical behaviour under cyclic loading. Alternatively, the strain methods that include plastic characteristics become suitable. The methods can be used in conjunction with the resistance curve. The crack propagation can be analysed using the elastic-plastic fracture mechanics instead of the linear elastic fracture mechanics.

This dissertation focuses on the fatigue assessment of welded connections in bridge engineering. In the following sections of Chapter 2, the general welded connections are introduced first. Two main methods, the S-N curve and the fracture mechanics are discussed. Afterwards, fatigue assessment methods at either the global or local level are presented. The history and the mechanical performance of OBDs under traffic loading are described. The details sensitive to fatigue are summarised. Finally, the state of the art for the studied details is given.

2.2. WELDED CONNECTIONS

Structures in practice consist of elements that are linked by connections or joints. Welded connections are one of the most commonly used types in civil engineering, mechanical engineering, aerospace engineering, *etc.* Elements (normally plates) are joined together by molten filler material. Arc welding, a widely used welding process, creates high temperatures using the heat of an electric arc. Metal in the weld pool melts, which connects separate workpieces into one after cooling down. Two welding procedures, automatic and manual welding, are widely used in practice. Arc welding includes processes such as Metal Inert Gas (MIG) welding, Flux-Cored Arc Welding (FCAW), Gas Metal Arc Welding (GMAW), Shielded Metal Arc Welding (SMAW), Submerged Arc Welding (SAW), *etc.*, which are commonly used in civil engineering. Other types of welding, such as laser welding and friction welding, are receiving increasing attention. Figure 2.3 illustrates the MIG welding process, where an arc forms and a weld pool develops at the end of an electrode. Shielding gas is supplied to protect the weld pool from the surrounding atmosphere.

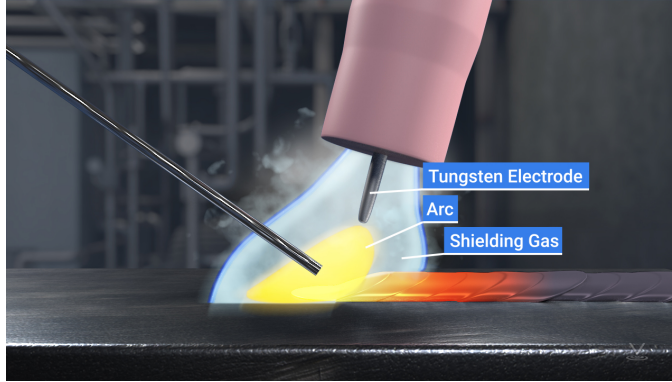
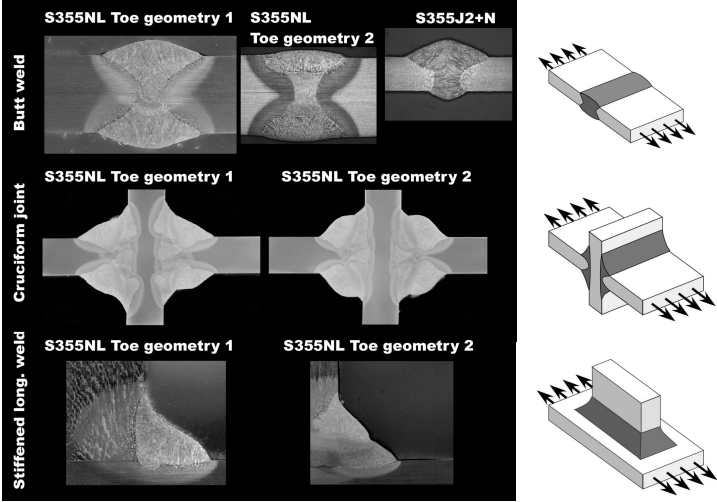


Figure 2.3: Illustration of the MIG welding process (adapted from an online source [37]).

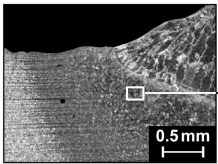
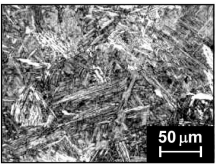
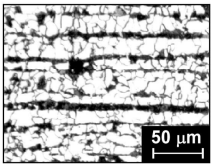
2.2.1. GEOMETRIC CHARACTERISTICS

WELD TOE

Schork *et al.* investigated three commonly used types of welded connections or joints (butt welded connections, load carrying cruciform joints, and longitudinal stiffened connections) made of the mild strength steel grade S355 [38], see Figure 2.4a for the macro sections. 3 mm thick steel plates were used to weld S355J2+N and 10 mm thick steel plates for the rest. Toe geometry 1 represents the “standard” weld profile shapes, whereas toe geometry 2 has either non-straight (too concave or too convex) or uneven weld profiles. Figure 2.4b shows the welded steel plate (Base Metal in short BM), Weld Metal (WM), and Heat Affected Zone (HAZ) at the welds. The materials are inhomogeneous and different microstructural properties are observed [39]. Also, different grey gradients are observed from the macro sections of the welded connections.



(a) macro sections [38]

	Weld toe: Overview	Detail (HAZ)	Detail (base metal)
S355NL			

(b) metallographic views [39]

Figure 2.4: Macro sections and metallographic views of commonly used types of welded connections [38, 39].

Figure 2.5 illustrates a more accurate description of the weld profiles at the weld toe. Local deviations or notches exist at the weld profile, which can be measured by the fitted arcs. The geometric characteristics at the crack initiation location (weld toe here) are described by the open flank angle α and the radius ρ (a relatively small ρ and a large ρ are illustrated for comparison). The global (weld profile) and the local (α and ρ) characteristics affect the fatigue performance of the weld toe, especially in the crack initiation and the short crack propagation periods.

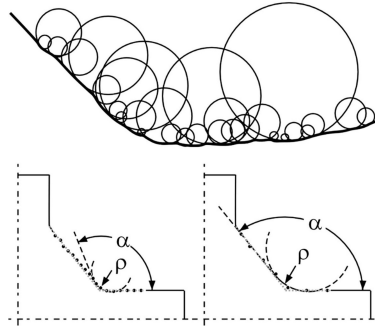


Figure 2.5: Illustration of geometric characteristics at the weld toe [38].

WELD ROOT

Figure 2.6 shows part of a fillet connection with a macro section photo and two illustrations. In addition to weld profile shapes, the throat thickness is important to the weld root as the stress states at the weld root are directly determined by the throat thickness. The connection may have a gap between the stiffener end and the plate, and be partially welded (or even fully penetrated). With a partial penetration, the actual throat thickness is larger than the theoretical (nominal) throat thickness.

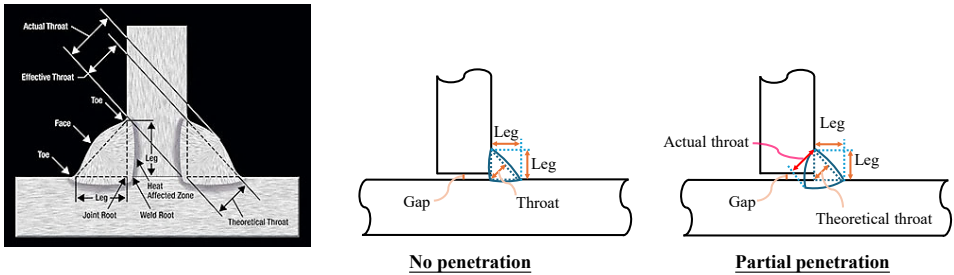


Figure 2.6: Macro section photo (adapted from [40]) and illustrations of (part of) a fillet welded connection.

2.2.2. DEFECTS, MISALIGNMENT, AND DISTORTION

Welded connections may contain undercutting and lack of penetration, have weaker or problematic material properties, crack due to cooling, see Figure 2.7.

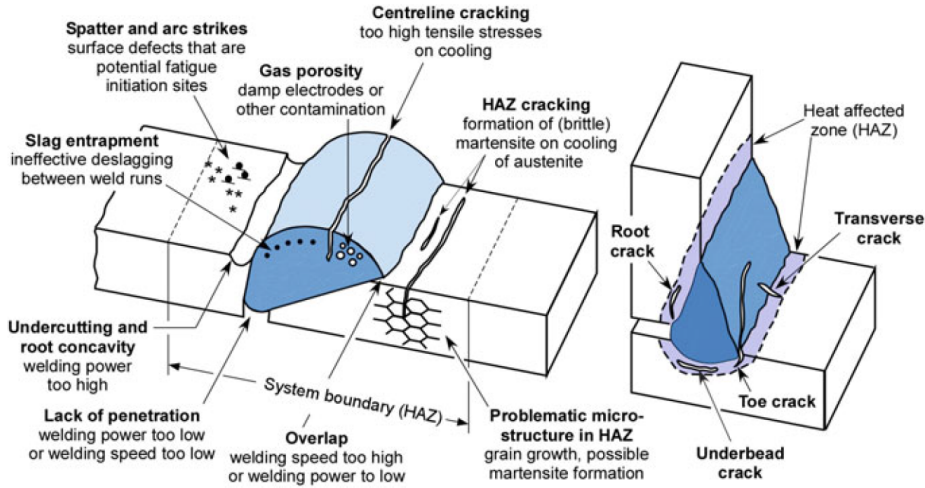


Figure 2.7: Illustration of weld defects in a weld connection [41, 42].

Figure 2.8 illustrates the misalignment and the distortion that may occur in a welded connection. The reasons can be the inaccurate placement of welded plates and the welding-induced deformation. The latter will be further discussed in the following section.

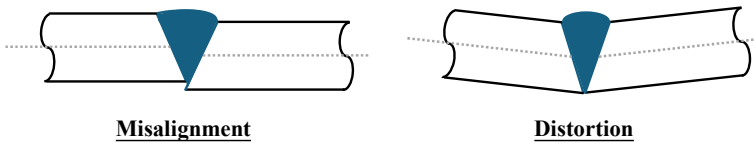


Figure 2.8: Illustration of the misalignment and the distortion in welded connections.

2.2.3. MATERIAL PROPERTIES AND WELDING INDUCED RESIDUAL STRESS

This section explains the material properties and the measured Welding induced Residual Stress (WRS) in butt welded connections made of the mild strength steel S355NL. The hardness distribution, stress-strain curves and WRS distributions from [39, 43] will be presented.

Kucharczyk *et al.* investigated the Vickers hardness and the cyclic material properties for 10 mm thick double-sided butt welded (with two cap passes) connections [39], see Figure 2.9 for one example. The BM and the regions with coarse or fine grains at either cap pass 1 or cap pass 2 side are studied. The largest hardness, approximately 400 HV1, occurs at the coarse grain region at the cap pass 2 side. The hardness reduces to the range of 210 HV1 to 240 HV1 and 180 HV1 to 210 HV1 when away in the WM and the BM directions, respectively. The maximum hardness of the two grain regions at the cap pass 1 side is below 330 HV1, smaller than the value at the cap pass 2 side. It is logical as the metal cools down under weaker constraints after the cap pass 1 welding than after the

cap pass 2 welding. Figure 2.9b shows that the HAZ has significantly higher mechanical properties than the BM. The coarse grain parts in the HAZ give the highest stress under the same strain. This trend is in line with the Vickers hardness measurements.

2

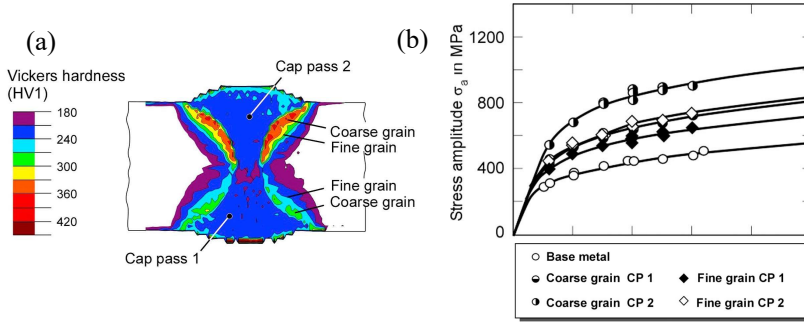


Figure 2.9: The Vickers hardness and the cyclic stress-strain curves of a double-sided welded connection using S355NL as base metal (adapted from [39]).

Figure 2.10 shows the WRS profiles in the single-sided welded butt connections made of the steel grade S355J2G3 under either strong constraints (large-scale specimen) or weak constraints (small-scale specimen) [43]. The $250 \times 60 \times 6 \text{ mm}^3$ welded connections are single-sided welded in the middle using four passes Tungsten Inert Gas (TIG) welding. The transverse residual stress (perpendicular to the weld line) and the longitudinal residual stress (parallel to the weld line) are measured at the weld toe side. High transverse (approximately 200 MPa to 450 MPa) and longitudinal (approximately 100 MPa to 200 MPa) residual stresses are observed around weld of the large-scale specimen, whereas the values at the same locations of the smaller-scale specimen are approximately 200 MPa to 300 MPa smaller.

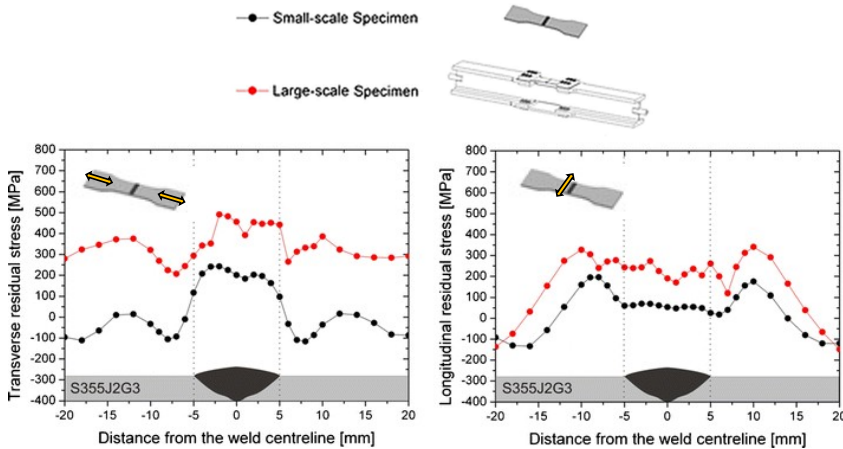


Figure 2.10: Residual stress profiles in small- and large-scale welded specimens (transverse stress on the left and longitudinal stress on the right) [43].

2.3. HIGH CYCLE FATIGUE ASSESSMENT METHODS

Figure 2.11 gives an overview of the fatigue assessment methods commonly used for welded connections [44–46]. The methods based on the S-N curve and the (modified) Paris' equation are developed for the fatigue resistance and the crack propagation assessment, respectively. The former category includes the nominal stress, structural stress, and (effective) notch stress (strain) methods. The latter predicts crack growth using either the cyclic J-integral within the elastic-plastic fracture mechanics or cyclic stress intensity factor within the linear elastic fracture mechanics framework, in conjunction with the (modified) Paris' equation. These methods are introduced and discussed in the following parts of the section.

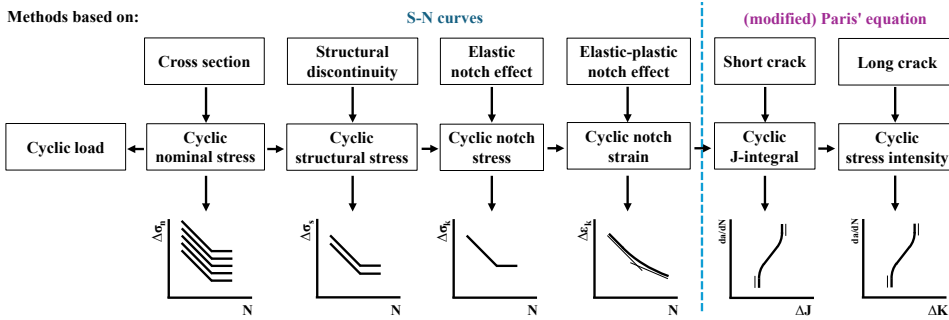


Figure 2.11: Fatigue assessment methods for welded connections [46].

2.3.1. S-N CURVE

The S-N curve is developed from the Wöhler curve, which was proposed by Wöhler when he carried out rotating cantilever fatigue bending tests on plain specimens under the constant amplitude loading [47]. This is further simplified to a linear relationship in a double-logarithmic curve by Basquin [48]. Figure 2.12 shows the fatigue resistance curve (one curve as an example) in (pr)EN 1993-1-9 [16, 17]. The stress range (normal stress, or shear stress, or in combination) and the number of cycles, both in the double-logarithmic scale, are linked by a linear curve with the slope of m_1 in the finite life region. The Constant Amplitude (CA) fatigue has a limit called the Constant Amplitude Fatigue Limit (CAFL), $\Delta\sigma_D$, at N_D . For the Variable Amplitude (VA) fatigue with normal stress ranges, the fatigue resistance curve is extended to the cut-off limit with $\Delta\sigma_L$ at N_L . A different slope, m_2 , is applied to $\Delta\sigma$ between $\Delta\sigma_D$ and $\Delta\sigma_L$. $\Delta\sigma_D$ and N_D are based on notch types in prEN 1993-1-9: 2023 [17] instead of the fixed values given in EN 1993-1-9: 2005 [16]. The characteristic reference values of fatigue resistance $\Delta\sigma_C$ at $N_C = 2 \times 10^6$ cycles are usually calculated using a lower prediction bound of survival for logarithmic number of cycles with 95% confidence [16, 17]. The partial factors for fatigue, γ_{Mf} , are recommended for two design concepts (safe life or damage tolerant) and three consequences of failure (low or medium or high) with values in the range of 1.00 to 1.35 [16, 17].

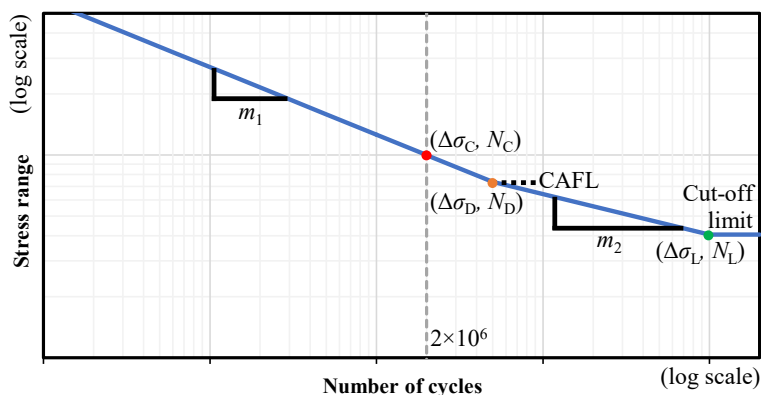


Figure 2.12: Fatigue resistance curve designated in (pr)EN 1993-1-9 [16, 17].

2.3.2. FATIGUE CRACKING IN WELDED CONNECTIONS

Weld toe cracks and weld root cracks are the two most commonly present types of crack in welded connections, see Figure 2.13 for the illustration of typical cracks in welded connections. The weld toe crack initiates from the weld toe and propagates into the main plate, Figure 2.13a to Figure 2.13e. The weld root crack initiates from the weld root and (normally) propagates into the weld throat, Figure 2.13f to Figure 2.13j.

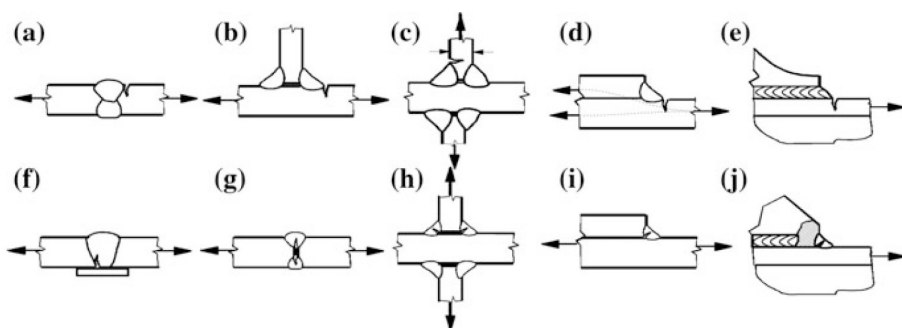


Figure 2.13: Fatigue cracks commonly present in welded connections: (a)–(e) weld toe cracks, (f)–(j) weld root cracks [23].

2.3.3. NOMINAL STRESS

The Nominal Stress (NS) is the main stress method for fatigue design, for example in the European standard (pr)EN 1993-1-9 [16, 17], United States AASHTO LRFD Bridge Design Specifications [19], and Japanese JSSC recommendations [18]. The NS method uses the stress amplitudes at the critical cross section as the fatigue driving stress ranges in conjunction with the S-N curve. Figure 2.14 gives an illustration where the NS can be calculated using the force F and the bending moment M divided by the cross section characteristics based on the plate thickness t and plate length l . The corresponding S-

N curve includes the influence of material, notch, size effect, surface, residual stress, etc. [34]. Note that the shear force may occur, which is not illustrated in Figure 2.14.

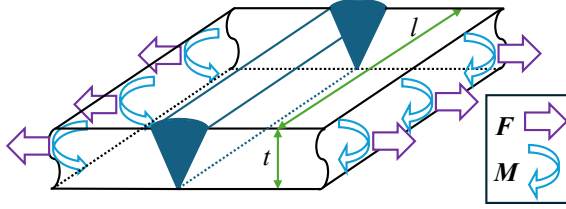


Figure 2.14: Illustration of the force and the bending moment in a welded connection.

The stresses in weld throat are used for the root crack assessment in fillet (partial penetration) welded load carrying connections. Figure 2.15 illustrates the normal weld stress σ_{wf} and the shear weld stress τ_{wf} in which a is the nominal weld thickness and l_w is the nominal weld length.

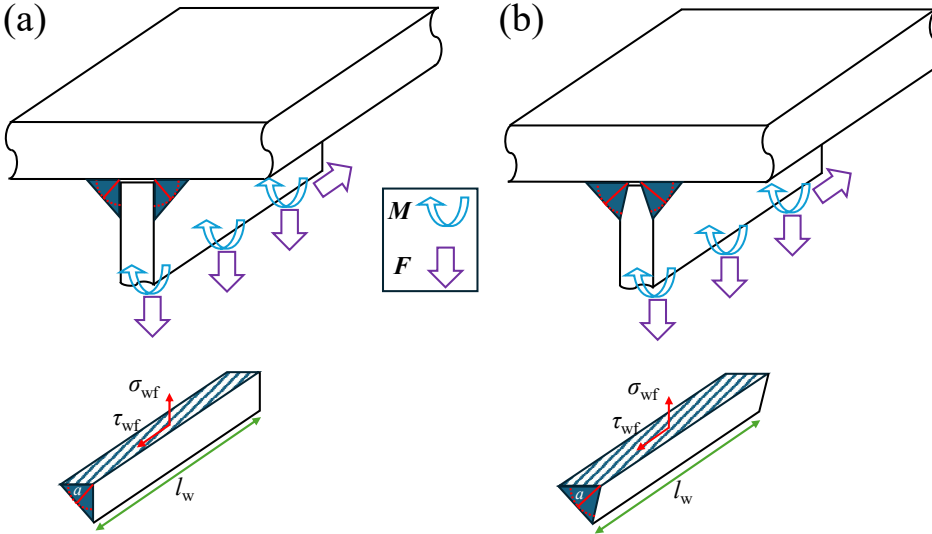


Figure 2.15: Relevant stresses in a (a) fillet or (b) partial penetration welded load carrying connection [17].

2.3.4. HOT SPOT STRESS

Despite the ease of use of the NS method, structural (geometric) stress, also called Hot Spot Stress (HSS), has been developed initially for the weld toe (hot spot) to make the design method as general as possible to predict the fatigue resistance distribution at the weld toe. The HSS (σ_{hs}) comprises the bending stress and the membrane stress in the vicinity of a weld while excluding the non-linear stress raised by local notch effects [23]. The surface extrapolation approach empirically selects the locations at certain distances at the weld toe side and extrapolates the stresses (σ_1 and σ_2) to the weld toe. Figure 2.16

illustrates the stress states and the extrapolation of HSS for the “type a” hot spot (weld toe on plate surface) [23]. The other type of hot spot (“type b”) is the weld toe at plate edge [23], which is outside the scope of the current study. The method is also adopted in [16–19] with the characteristic strength curve determined by σ_C , equal to 80 MPa or 90 MPa or 100 MPa for welded connections with standard shapes. The number of fatigue resistance curves is reduced compared with the NS method as the HSS is insensitive to geometry. However, the stress states, plate thickness, welding procedures, *etc.*, of construction detail affect the fatigue performance. The development of detail categories based on the fatigue behaviour of realistic details provides a suitable (accurate) fatigue assessment using the HSS.

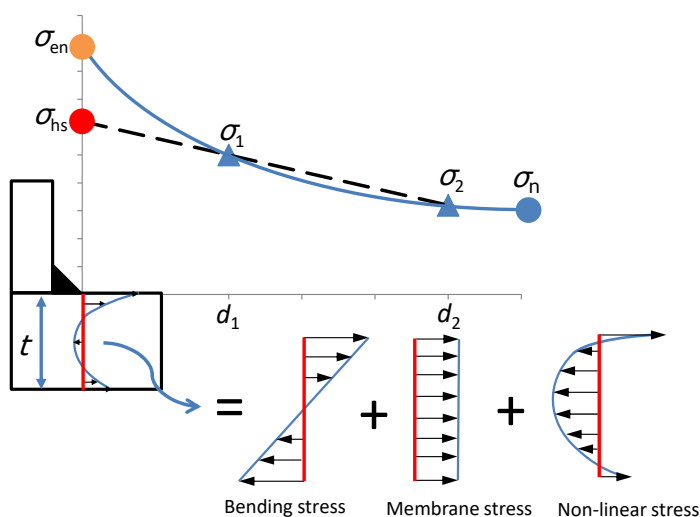


Figure 2.16: Illustration of the stress path in a component close to the weld.

Sørensen *et al.* further extended the HSS to the weld root assessment by extrapolating the stress in the weld throat to the weld root with distances $0.25a$ and $0.75a$ from the weld root [49].

The force equivalent method, developed by Dong [50], uses the nodal stress from the crack initiation cross section. The membrane and shear stresses, and the bending stress are calculated from the nodal normal and shear forces, and the bending moment, respectively. The force equivalent method can be applied to either the weld toe or the weld root assessment.

2.3.5. EFFECTIVE NOTCH STRESS

The Effective Notch Stress (ENS), σ_{en} , is a local stress that includes non-linear stress as shown in Figure 2.16. The concept initiates from the stress state assessment at the notch (weld toe or weld root). A fictitious notch with a specified radius, r_{ref} , is created at the notch, see Figure 2.17. Two generally used radii are 1.00 mm and 0.05 mm [23, 51]. The former is determined from the worst case of a real radius, resulting in the aver-

aged notch stress obtained by the integration of the stress distribution in a real notch, Figure 2.18 [51]. The latter is a compromise value for the FE calculation following the relationship between the stress intensity factor and the notch stress, and the crack tip blunting [51]. Note that $r_{\text{ref}} = 0.30 \text{ mm}$ was proposed by researchers as described in [52]. No further discussion on $r_{\text{ref}} = 0.30 \text{ mm}$ is given as it is outside the scope of the current study.

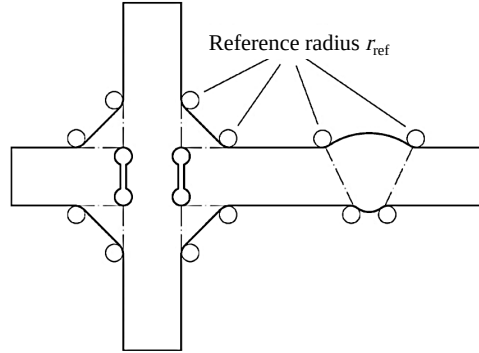


Figure 2.17: ENS at the weld toes and roots [51].

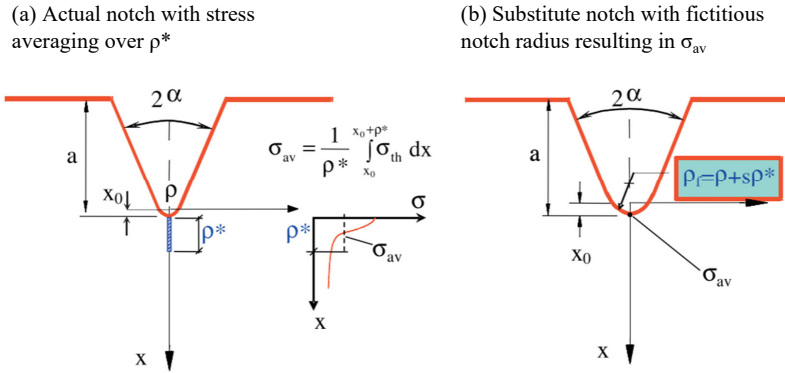


Figure 2.18: Neuber's micro-support concept [51].

Two types of criteria, the maximum principal stress and the von Mises stress, can be used for the ENS. The former suits the proportional loading better [53]. Unlike the NS and the HSS, only one fatigue resistance curve is needed for each combination (r_{ref} and the criterion).

2.3.6. AVERAGED STRAIN ENERGY DENSITY FACTORS

The ENS assumes a r_{ref} at the notch to avoid singularity. The Notch Stress Intensity Factors (N-SIFs) study the stress and strain states at the notch without using a fictitious notch [54, 55]. The N-SIF is initially developed to predict the fatigue resistance of crack initiation, whereas it can also be used to predict the total fatigue life of welded connections [55]. However, analytical solutions for N-SIFs are only available for specific open angles (2α), e.g. 135° . The averaged Strain Energy Density (SED) factor over the critical area is thus used, see Figure 2.19. The SED can be used with various 2α and is insensitive to mesh size [56]. Livieri and Lazzarin [21] studied about 650 fatigue data with either the weld toe or weld root failure and recommended a characteristic value of 0.058 N mm/mm^3 at 2 million cycles associated with a radius $R_C = 0.28 \text{ mm}$ for steel welded connections.

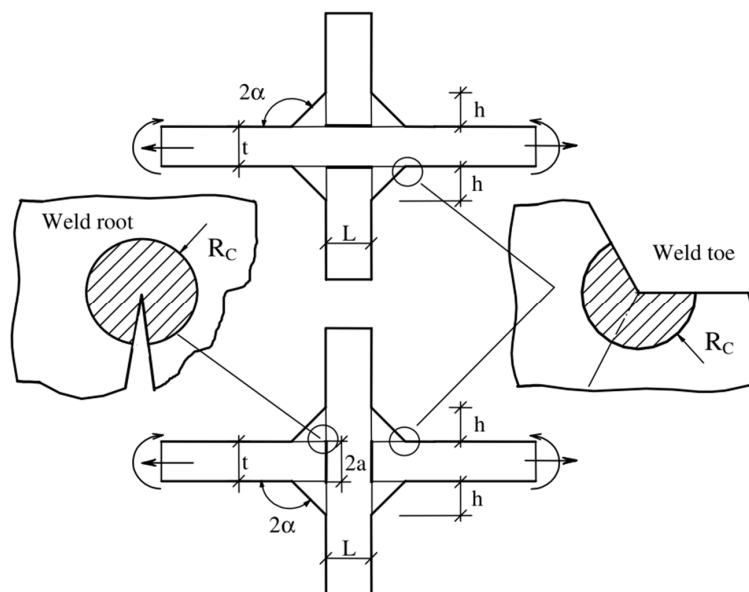


Figure 2.19: Geometrical parameters and the critical area for the weld toe and the weld root in the SED method [21].

2.3.7. NOTCH STRAIN

The linear elastic material assumption is adopted in the NS, HSS, ENS methods. However, inelastic material behaviour at the notch may occur at high load levels. Figure 2.20 illustrates the force level that causes stresses beyond the elastic limit. Cyclic elastic-plastic material behaviour (red curve) occurs close to the notch. Other parts keep in elastic (blue curve). Other characteristics (residual stress, secondary notch, etc.) further increase stress concentrations, resulting in higher stress at the notch. The strain-based method was therefore developed first for low cycle fatigue by Manson and Coffin in the 1950s [57, 58]. Afterwards, it was extended to high cycle fatigue by Morrow in 1965 [59]. It was further modified to include the mean stress effect by Smith *et al.* in 1970 [60].

The strain-lifetime curve (ϵ - N) instead of the S- N curve, should be used for the strain-based method fatigue assessments. Figure 2.21 shows one example where the horizontal and the vertical axes are the doubled number of cycles $2N$ in logarithmic and the strain amplitude $\Delta\epsilon/2$ in logarithmic, respectively. The total ϵ - N consists of the elastic strain range ($\Delta\epsilon^E/2$) and the plastic strain range ($\Delta\epsilon^P/2$). $2N$ covers both low cycle and high cycle situations. Implementation of cyclic elastic-plastic material properties allows a more realistic mechanical analysis (hardening or softening, plastic shakedown, *etc.*) at the notch. It is especially useful for the cases with large preloading (preloaded bolts) or high residual stress (welds). Note that the notch strain method is practically suitable for fatigue initiation prediction as it evaluates the strain at the initiation stage.

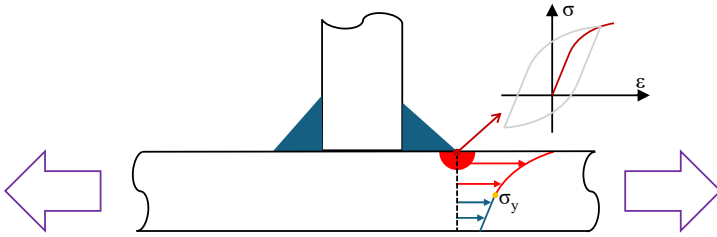


Figure 2.20: Strain-stress behaviour at the weld toe of a welded connection.

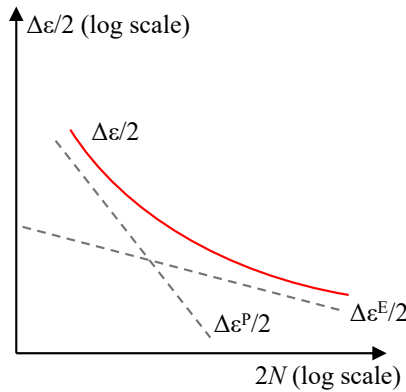


Figure 2.21: Illustration of a ϵ - N curve.

2.3.8. FRACTURE MECHANICS

Although the S- N or the ϵ - N curve is widely used in fatigue design, the behaviour of cracks cannot be described by these methods. Fracture mechanics, which was developed to describe the crack behaviour, is a powerful method when the crack propagation behaviour needs to be analysed.

Fracture can be classified into three modes: Mode I (open), Mode II (in plane shearing), and Mode III (out of plane tearing) [61], see Figure 2.22. The Mode I type crack is

the most common and dangerous one for long cracks.

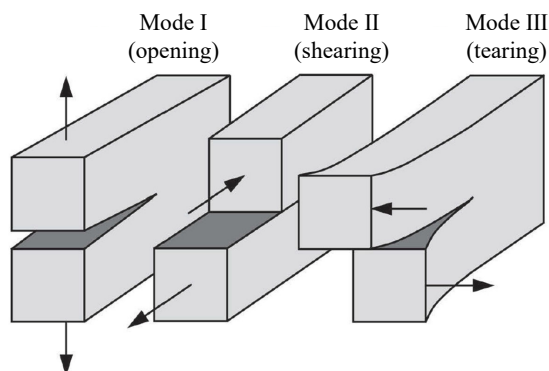


Figure 2.22: Illustration of fracture modes [61].

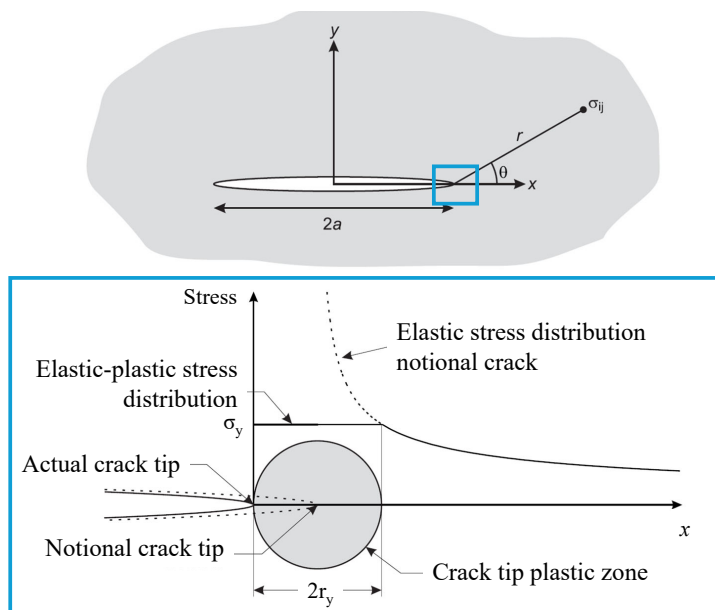


Figure 2.23: Illustration of the stress states at the crack tip of a Mode I type crack according to Irwin [62].

Figure 2.23 illustrates the stress states at the crack tip of a Mode I type crack (crack length is $2a$). A plastic zone exists at the crack tip due to the high stress raised by the sharp notch. A circle with the diameter $2r_y$ is assumed. The origin of the circle is assumed as the notional crack tip. The linear elastic mechanics results in the stress singularity at the notional crack tip.

The Small Scale Yield (SSY) with $r_y \ll a$ is required for the application of the Linear Elastic Fracture Mechanics (LEFM). The (effective) Stress Intensity Factor (SIF) is used as the crack propagation driving stress, expressed as $K_I = f\sigma\sqrt{2\pi a}$, where f is the correction factor for parameters, *e.g.* geometries and load types. With the increase of r_y , the SSY assumption is no longer valid and the plasticity cannot be ignored. Alternatively, the Elastic-Plastic Fracture Mechanics (EPFM), which takes the material inelasticity or plasticity into consideration, can be used. The J-integral [63] is a widely used crack propagation driving stress in the EPFM. The application has been extended from monotonic loading to cyclic loading for fatigue analysis using the two types of driving stresses, SIF range [23, 24, 64] and cyclic J-integral [42].

Fracture mechanics becomes an attractive option for the fatigue analysis of welded connections as the crack propagation occupies a large proportion of the fatigue life [42]. Figure 2.24a illustrates the fatigue crack propagation as a function of the SIF ranges, ΔK , which consists of three regimes [42, 65]:

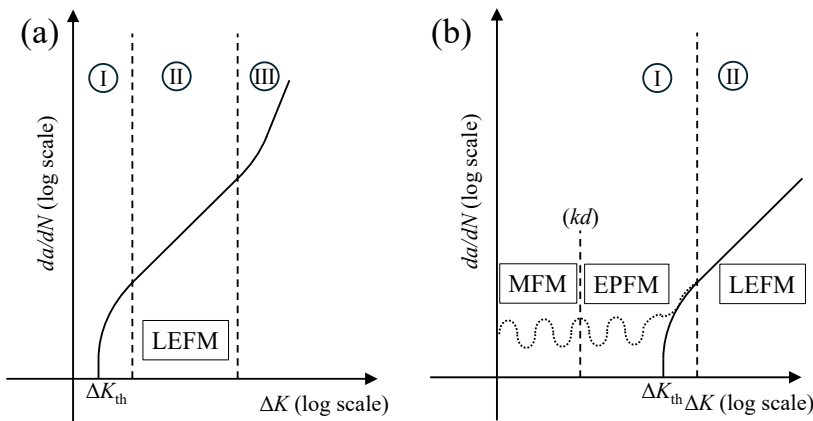


Figure 2.24: Illustration of fatigue crack propagation as a function of ΔK (reproduced from [42, 65]).

- Regime I: Micro cracks initiate and coalesce in the regime. The Micro Fracture Mechanics (MFM) can be used when the crack size is smaller than the critical value, kd , normally at the grain size level. The microstructural characteristics of steel, *e.g.* grain sizes and orientations, dominate the crack propagation behaviour in this stage. With the crack growth, plasticity becomes more extensive and can engulf several grains. The influence of microstructural barriers is reduced and steel can be treated as a homogeneous material. In this stage, the crack is small and the SSY assumption is not satisfied. The EPFM needs to be used instead of the LEFM. Note that considerable scatter still exists in the stage and the crack propagation rate is

sensitive to the stress level. A crack arrest may occur before growing into a long crack if the stress range level is not larger than the threshold value ($\Delta K \leq \Delta K_{th}$).

- Regime II: A straight line represents the double-logarithmic $da/dN - \Delta K$, described by the Paris' equation for Mode I type crack [22] when $\Delta K > \Delta K_{th}$. The line depends on the material properties and environmental effects. Other parameters, e.g. the stress ratio and the load frequency, may also affect the propagation rate. Note that a bilinear line can be used to improve the prediction accuracy.
- Regime III: The crack propagation rate accelerates and stops with a final fracture in the monotonic crack growth. The crack propagation can be described by the EPFM or damage mechanics instead of the LEFM due to existence of large plasticity. Microstructural characteristics and stress states have strong effects on the crack propagation behaviour at this regime.

2.3.9. DISCUSSION OF THE EXISTING ADVANCED FATIGUE ANALYSIS MODELS

Advanced fatigue analysis models (or methods) besides the commonly used nominal stress, hot spot stress, and effect notch stress methods were developed to improve the accuracy of the fatigue life prediction of metallic structures.

The 4R method was proposed by Ahola *et al.* [66] to implement four parameters: notch stress range $\Delta \sigma_k(r)$, applied stress ratio R , ultimate material strength R_m , and residual stress σ_{res} in the fatigue assessment. The 4R method adds the measured notch radius to the reference radius at the crack initiation in the Finite Element Analysis (FEA). The local cyclic elastic-plastic behaviour at the notch is considered and the correction of the mean stress is carried out using the Smith-Watson-Topper parameter. Effects of σ_{res} and R are accounted by using a master S-N curve.

The LEFM is used to analyse the fatigue crack propagation behaviour of structures in which initial flaws are caused in the fabrication or manufacture process [23, 24, 64]. An initial crack with a depth ranging from 0.1 mm to 0.5 mm [23, 24, 64, 67] is commonly assumed as the starting point for the fatigue propagation calculation. Either linear or bilinear propagation model may be used and the applied stress ratio $R \leq 0.5$ or $R > 0.5$ can be taken into consideration by the material constant C [24, 64].

The UniGrow fatigue crack growth model was proposed by Noroozi *et al.* [68] and Mikheevskiy *et al.* [69]. The UniGrow model uses a unified two-parameter driving force to account the mean stress in the fatigue crack propagation. The overloading and crack closure are studied by considering the elastic-plastic material behaviour at a finite notch/crack tip radius and the applied SIFs are obtained by the LEFM. The UniGrow model was improved by Pedrosa *et al.* [70], who included the strain-based crack initiation in the prediction.

The 2 stage model was proposed by Röscher and Knobloch [71] to cover both crack initiation and propagation stages using the strain-based method and the LEFM, respectively. The distinction between initiation and propagation is given by the phenomenon-based value, which is an initial crack depth of 0.5 mm in the proof of concept calculation where butt welded connections were studied. It was found that the initiation life dominates the fatigue life with small stress ranges, whereas the propagation phase gains a

larger portion when the stress ranges are large. The crack initiation and propagation life of either high quality welds or welds with large imperfections can be assessed using the 2 stage model.

However, the welding induced residual stress and its redistribution caused by the post-weld treatment or the development of crack propagation are not computationally simulated by the 4R method, the UniGrow model, and the 2 stage model.

Since the notch in practice may be larger than the conservatively assumed value of 1 mm or 0.05 mm [23] and the initial flaws may be smaller than the assumed length ranging from 0.1 mm to 0.5 mm [23, 24, 64, 67], the SSY assumption at the crack tip may not be reached, especially when the welding-induced residual stress is taken into consideration. This limits the use of LEFM when the crack is short. Alternatively, the EPFM can be used for the short crack propagation with the modified Paris' equation [72]. The plasticity caused crack closure can be studied by the cohesive extended FE model [73, 74] or the EPFM [75].

The LEFM based model is a practical option to analyse the long crack propagation. The effect of stress ratio can be corrected by the improved Paris' equation, *e.g.* the Walker's correction [76] and the overloading (plasticity caused crack closure) can be considered by the improved models, *e.g.* the Wheeler's model [77, 78].

2.4. FATIGUE IN OBDs

2.4.1. INTRODUCTION OF OBDs

An OBD is an integrated superstructure in a bridge that consists of a top deck plate, stiffened by longitudinal stiffeners and further by crossbeams as shown in Figure 1.2. Either open or closed stiffeners with various shapes have been used in history. Cut-outs at the lower region of stiffeners are made as a practical fabrication option because a good fit between crossbeam and stiffener is challenging to meet at corners. Figure 2.25 shows the commonly used longitudinal stiffeners in Germany, designated as:

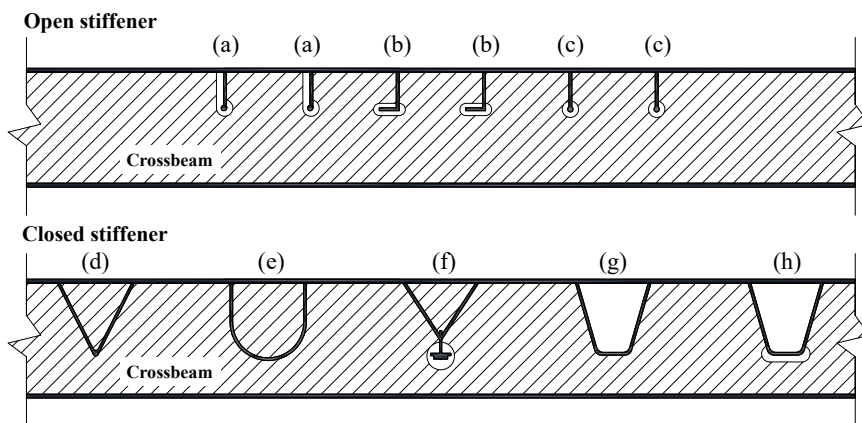


Figure 2.25: Representative designs of longitudinal stiffeners (open stiffeners at the top and closed stiffeners at the bottom) in Germany, reproduced from [10].

- (a). bulb flat-shaped continuous stiffener
- (b). L-shape continuous stiffener
- (c). flat continuous stiffener
- (d). V-shape discontinuous stiffener
- (e). U-shape discontinuous stiffener
- (f). Y-shape (champagne glass shape) discontinuous stiffener
- (g). trapezoidal continuous stiffener without cut-out
- (h). trapezoidal continuous stiffener with cut-out

Among them, trapezoidal continuous stiffeners, (g) and (h), have been the main types used since the 1980s with a span between crossbeams in the range of 1.88 to 5.00 metres [10], which will be further discussed in the following sections of this chapter.

MECHANICAL BEHAVIOUR UNDER TRAFFIC LOADING

Sub-elements in OBDs are mainly connected by welds or bolts. The mechanical behaviour of one sub-element depends on the interactions with others. This means the design method that separately analyses each sub-element assuming it behaves independently without considering the interactions is not suitable for OBDs [2]. Seven levels of deformation are designated for an OBD under traffic loading by the Federal Highway Administration manual: Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges [2], as shown in Figure 2.26. Note that similar specifications are given in EN 1993-2 [79]. Levels 1 and 2 are the local deck plate deformation and the panel deformation, respectively. Level 3 is the longitudinal flexure. Levels 4 and 5 are the in-plane flexure and the distortion of a crossbeam. Level 6 is the distortion of stiffeners due to cut-out. Level 7 is the global deformation of an OBD.

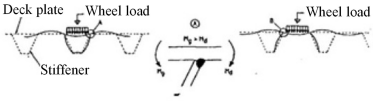
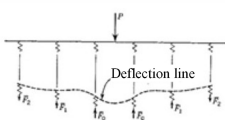
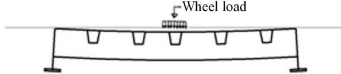
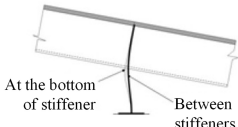
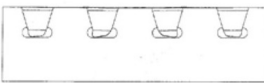
System	Action	Figure	Result
1	Local deck plate deformation		Transverse stress in deck plate, stiffener, and stiffener-to-deck plate welded connection
2	Deck plate deformation		Transverse stress in deck plate
3	Longitudinal deformation of stiffener		Longitudinal bending and shear in stiffener acting as a continuous beam on flexible crossbeam supports
4	In-plane deformation of crossbeam		Bending and shear in crossbeam acting as beam spanning between rigid girders
5	Distortion of crossbeam		Out-of-plane bending of crossbeam web at stiffener due to the rotation of stiffener
6	Distortion of stiffener web		Local bending of stiffener web due to crossbeam cut-out
7	Global deformation		Axial, bending, and shear stresses from supporting girder deformation

Figure 2.26: Deformation mechanism of an OBD under traffic loading, modified based on [2].

ACTIONS ON OBDs

Actions of traffic on OBDs are simplified using uniformly distributed load patches. Five Fatigue Load Models (FLMs) are defined in EN 1991-2 for fatigue assessment [80]:

2

- FLM1 has the configuration of characteristic Load Model 1 (for ultimate limit state verification) with reduced values. The maximum and minimum stresses should be determined from the possible load arrangements of the model on the bridge.
- FLM2 consists of a set of idealised lorries, called “frequent” lorries. Each lorry is defined by the number of axles and the axle spacing, the frequent load of each axle, the wheel contact areas and the transverse distance between wheels. The maximum and minimum stresses should be determined from the most severe effects of different lorries, separately considered, travelling alone along the appropriate lane.
- FLM3 consists of four axles, each of them having two identical wheels. The maximum and minimum stresses and the stress ranges for each cycle of stress fluctuation, *i.e.* their algebraic difference, resulting from the transit of the model along the bridge should be calculated.
- FLM4 consists of sets of standard lorries which together produce effects equivalent to those of typical traffic on European roads. A set of lorries appropriate to the traffic mixes predicted for the route should be taken into account. The stress range spectrum and the corresponding number of cycles from each fluctuation in stress during the passage of individual lorries on the bridge should be the Rainflow or the Reservoir counting method.
- FLM5 consists of the direct application of recorded traffic data, supplemented, if relevant, by appropriate statistical and projected extrapolations.

Among them, FLM3 to FLM5 are intended to be used for fatigue life assessment using the fatigue resistance curves defined in EN 1993-1-9 [16] and should not be used to check if fatigue life can be considered as unlimited [80].

The pavement stiffness may be temperature dependent [15, 81, 82]. The conservative method may exclude the effect of pavement in fatigue assessment [2]. However, the monitoring of two long-span bridges in China with OBDs shows that the fatigue damage accumulation is sensitive to temperature [83]. It is therefore important to consider the pavement with its proper characteristics [15, 81, 84, 85]. Recent developed TS [27] provides three levels of assessment methods (from most realistic to most simplified):

- Level 1: temperature-dependent pavement stiffness,
- Level 2: temperature-independent pavement stiffness,
- Level 3: dispersal of load without pavement implemented in the model.

2.4.2. EXAMPLES OF THE FATIGUE ASSESSMENT FOR OBDs

Fatigue assessment of the critical locations of bridges was carried out using the measured strain ranges in the structural health monitoring, *e.g.* the van Brienenoord bridge in the Netherlands [15], the Great Belt Bridge in Denmark [86], and the M0 Háros Danube highway Bridge in Germany [87], several long span bridges in China [83, 88]. Among them, the HSS was used for fatigue assessment using measured strain ranges close to the welds in [88]. The LEFM was used to predict the crack locations and their propagation behaviours for Bronx–Whitestone Bridge using measured strain ranges [89]. Although the structural health monitoring provides realistic strain ranges at the critical locations, it is still expensive to be largely used [90]. Alternatively, the stochastic traffic flow model calibrated against the weigh-in-motion system was used in the fatigue reliability analysis in [12].

2.4.3. DETAILS AND CRACKS

Figure 2.27 summarises the details of an OBD supported by continuous closed stiffeners. Cracks covered by prEN 1993-1-9 [17] (in blue) or by TS [27] (in yellow) or by both (in red) are illustrated. The designation follows TS if the detail is covered by TS because TS covers more details than prEN 1993-1-9. Details with underscore (C2a, C2b, C1a, C1c, C3a, and C4d) are further studied in the following chapters.

Detail 11 in Table 10.9 of [17]: deck plate to stiffener crack subject to normal stress; Detail C1a: stiffener-to-deck plate weld, weld toe crack in deck plate; Detail C1b: stiffener-to-deck plate weld, weld root crack in deck plate between crossbeams; Detail C1c: stiffener-to-deck plate weld, weld root crack in deck plate at crossbeam; Detail C2a: stiffener-to-deck plate weld, weld toe crack in stiffener; Detail C2b: stiffener-to-deck plate weld, weld root crack in weld; Detail C3a: crossbeam-to-stiffener weld, weld toe crack in stiffener at lower end of the weld; Detail C3b: crossbeam-to-stiffener weld at stiffener web, weld toe crack in stiffener; Detail C3c: crossbeam-to-stiffener weld at stiffener bottom, weld toe crack in stiffener; Detail C4a: crossbeam-to-stiffener weld at stiffener web, weld toe crack in crossbeam; Detail C4b: crossbeam-to-stiffener weld at stiffener web, weld root crack; Detail C4c: crossbeam-to-stiffener weld at stiffener bottom, weld toe crack in crossbeam; Detail C4d: crossbeam-to-stiffener weld at stiffener bottom, weld root crack; Detail C5: crossbeam-to-deck plate weld, weld toe crack in deck plate; Detail C6a: crossbeam-to-deck plate weld, weld toe crack in crossbeam; Detail C6b: crossbeam-to-deck plate weld, weld root crack; Detail C7a: deck plate splice joint in longitudinal direction welded from both sides, weld toe crack; Detail C7b: deck plate splice joint in transverse direction welded from both sides, weld toe crack; Detail C8a: stiffener splice joint welded from one side on permanent root backing, weld toe crack; Detail C8b: stiffener splice joint automatically welded from both sides, weld toe crack; Detail C9: crossbeam web at free end of the extended cut-out.

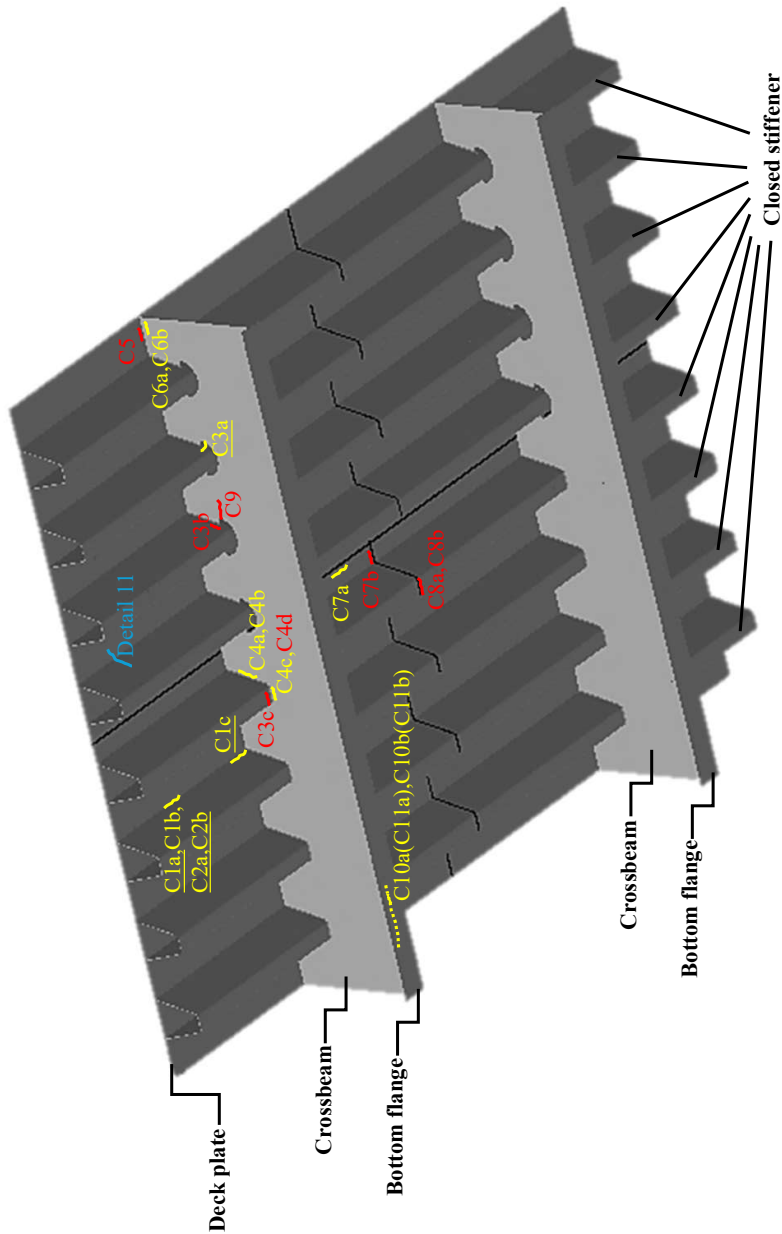


Figure 2.27: Illustration of commonly present cracks in an OBD supported by continuous closed stiffeners [17, 27].

2.5. STATE OF THE ART FOR THE STUDIED WELDED CONNECTIONS

Fatigue of steel bridges significantly affects the performance of ageing transportation worldwide [13]. Fatigue damages in steel bridges are mainly caused by: welding defects in fabrication, inappropriate structural details that result in low fatigue resistance, unforeseen stress and deformation in design at joints of members, and unexpected structural behaviour such as vibration [91]. Fatigue cracks were reported in OBDs, which often govern the service life [1, 7, 8, 14, 92]. One recent study in Japan reported inspections of 276 spans (32% of the total 856 spans) of OBDs in the Tokyo Metropolitan Expressways until March 31st 2004, in which fatigue cracks were found at 1391 locations of the 106 spans [92]. Most of the cracks were inspected at four locations (proportion in percentage):

- Location 1: Vertical stiffener-to-deck plate connection at main girder junction (40%)
- Location 2: Crossbeam-to-deck plate welded connection (rare in closed stiffeners)
- Location 3: Stiffener-to-deck plate welded connection (14%)
- Location 4: Stiffener-to-crossbeam welded connection with cut-out (20%)

Location 1 depends on the design and may not be common in European OBDs. Location 2 is rare in closed stiffeners. Locations 3 and 4 are, therefore, further studied in the dissertation.

The stiffener-to-deck plate welded connection experiences high stress ranges as it is directly under the wheel loading. The crossbeam-to-stiffener welded connection experiences multi-axial stress states. The following sections of this chapter give a state of the art discussion on the two types of connections.

2.5.1. STIFFENER-TO-DECK PLATE WELDED CONNECTION

The stiffener-to-deck plate welded connection is located below the overlay (or pavement). Wheel loads are applied on the overlay and stresses arise accordingly. Accurate calculations of stress ranges at the connections depend on the position and load of wheels, the tyre contact areas and the used calculation models [93]. Both the membrane and the bending stresses were analysed in [81, 93–96]. A LEFM based crack propagation model was developed for stiffener web cracking in [93]. The bending stress was found as the dominant component of the stress ranges in the deck plate and the stiffener web. Only the calculation for local bending in the stiffener web is given in EN 1993-2: 2006 [79] for the fatigue design of stiffener-to-deck plate welded connection between crossbeams. However, improvements in the accuracy may be reached when both membrane and bending stresses are considered [93].

Fatigue performance of connections was affected by the fabrication process and weld requirements [97]. Fillet welds were commonly used in the early designed OBDs. However, type C2b cracks were inspected in the existing bridge decks even after several years of use [1, 8, 14]. Requirements were afterwards given for the penetration ratios in bridge guidelines or codes, aiming to prevent type C2b crack and improve the fatigue resistance

of stiffener-to-deck plate welded connection. The required minimum Partial Joint Penetrations (PJPs) are 75% and 80% in the JSSC recommendations [18] and the AASHTO LRFD Bridge Design Specifications [19], respectively. (pr)EN 1993-1-9 [16, 17] recommends a lack of penetration ≤ 2 mm for partial penetration welded stiffener-to-deck plate connections. Melt-through welding is not allowed as cold lap may occur [98]. Moreover, the gap between the stiffener and the bottom of deck plate is required as ≤ 2 mm [16, 17, 19]. Double-sided welding (adding the welding at the inner side of stiffener web) was developed to prevent type C1b cracks in new OBDs [99, 100] or to repair them in existing bridges [101]. Stress ranges in the deck plate of the connection are higher at crossbeam than the values between crossbeams [1, 14, 88]. The connection at the location between crossbeams or at crossbeam is discussed separately in the following sections of this chapter.

BETWEEN CROSSBEAMS

Between crossbeams, the closed stiffener-to-deck plate welded connection consists of four details: C1a, C1b, C2a, and C2b. Fatigue crack initiates from the weld toe (C1a and C2a) or weld root (C1b and C2b), and propagates into the deck plate (C1a and C1b) or stiffener web (C2a and C2b). The HSS was adopted to evaluate the weld toes in OBDs by researchers [102–104] and the ENS was used to evaluate details C1a and C1b [95, 105, 106].

Detail C1a was experimentally studied in [105, 107–110]. Probabilistic fracture mechanics simulations with the plane strain assumption were used to predict the propagation of type C1a crack until the half thickness of deck plate in [111]. Type C1b crack was experimentally investigated in [112, 113]. Full-scale experiments with PJPs of 0%, 75%, and 100% were carried out with C1b type cracks observed in the 0% and 100% cases, and C1a and C1b types of cracks in the 75% case [114]. The crack path of C1b was tracked and the stress states close to the weld were calculated by FEA [115]. A decrease in fatigue resistance was observed with the increase of deck plate thickness [115]. Tensile residual stress exists at the weld root at the deck plate side, the initiation location of type C1b crack, which may be released with the crack propagation [114, 116]. Oppositely, compressive normal stresses (normal to the direction of deck plate thickness) were reported at the location in [117]. The welding procedure and the boundary conditions may affect the distribution of residual stress. Relaxation of residual stress may occur in the cyclic loading [99, 118].

Details C1a, C1b, C2a, and C2b were studied using component-level (small-scale) specimens in [119–121]. Higher fatigue resistance was found for either detail C1a or C1b when the PJP was 80% compared with the specimens with a PJP of 100% [95, 97, 120, 122]. Moreover, it was found that type C2b crack was more likely to occur when the PJP was small [110].

AT CROSSBEAM

Detail C1c was found to have a short crack initiation, followed by a relatively long crack propagation in [123, 124]. Probabilistic models based on the LEFM can predict the crack propagation and be updated using inspected results [125]. The crack propagation behaviour of detail C1c in the existing OBDs (10 or 12 mm thick deck plate), was predicted by the probabilistic crack propagation models to support the maintenance de-

cision [11, 90, 125, 126]. The validation or development of these propagation models using laboratory investigations becomes important to improve the accuracy of prediction [126, 127]. In the case of the 16 mm thick deck plate, curved fracture surfaces were observed (through-thickness cracks inspected at the high load level only) in [123, 124]. Reducing tensile residual stress at the crack initiation location, *i.e.* the weld root at detail C1c, may improve the fatigue performance [12, 124].

INNOVATION AND STRENGTHENING METHODS

Innovations in design or fabrication were made to improve the fatigue resistance of new OBDs and strengthening methods were developed for existing ones.

The hot-rolling technique was proposed to manufacture local thickened stiffeners with optimised shapes [84, 105, 112]. The application of concrete as an overlay was used in new OBDs. The reinforced Ultra-High Performance Concrete (UHPC) and the stiffened deck plate together worked as a composite component in which short headed studs were used to guarantee the interaction and the closed shape of stiffeners were larger than the conventional design [128–132].

A strengthening overlay was applied to reduce the stresses in the stiffener-to-deck plate connection and improve the fatigue lifetime for existing OBDs [133]. (High performance) concrete or steel plate was used as the strengthening overlay [14, 93, 134, 135]. Either thick or thin adhesive layer was used in the steel plate strengthening [133, 135, 136].

2.5.2. CROSSBEAM-TO-STIFFENER WELDED CONNECTION

Increasing the thickness of deck plate effectively reduces the stress ranges in the deck plate [95, 137]. However, the stresses in the crossbeam-to-stiffener welded connection are less affected. The critical locations of this connection are in the vicinity of cut-out or at the bottom region of the stiffener (in the design without cut-out) [138, 139]. Type C9 crack was also commonly seen [140–142] due to high tensile residual stresses (maximum value exceeding 200 MPa) and the high stress ranges induced by the traffic flow at the edge of cut-out [1, 119, 138, 142]. Fatigue behaviour of detail C3a, and the innovation and strengthening methods for the detail are further discussed in the following sections of this chapter. Detail C4d is of interest, whereas studies are lacking at the time of writing. Further discussions are therefore not given for detail C4d in this chapter.

WELD TOE CRACK IN STIFFENER AT LOWER END OF THE WELD (DETAIL C3A)

Detail C3a experiences multi-axial stress states due to the local distortion of stiffener web [143]. Various stiffener and cut-out shapes are used in design [1, 119, 138]. The combination of the two makes it challenging to use the NS for detail C3a. The detail was not covered in (pr)EN 1993-1-9 [16, 17]. Alternatively, a detail category based on the HSS was provided in the AASHTO LRFD Bridge Design Specifications [19], whereas the lowest fatigue resistance proposed for the standard welded connections was adopted. The fatigue resistance was developed using standard plate welded connections, mainly tested in tension-tension with a high load ratio (> 0.5) as a conservative option [144]. Moreover, the commonly seen stiffener thickness is 6 or 8 mm, significantly thinner than the reference thickness of 16 mm [26] or 25 mm [23]. It is known that the fatigue resistance of welded connection is affected by the plate thickness and the load

type. A thin plate or a bending load results in a higher fatigue resistance [145–147]. Relatively slow crack propagation rates were experimentally investigated for type C3a cracks in [1, 100, 119, 148–151]. Fracture mechanics calculations were carried out for detail C3a using a high proportion of bending stress (75%) in [152]. The authors stated that the crack growth slowed down when the crack tips reached the lower stress region (away from the crossbeam) [152]. As the failure criteria or inspection plans depend on the estimation of the probability of global collapse [144], a comprehensive study of the stress states of detail C3a and the crack propagation behaviour will be meaningful. At the time of writing, the HSS category for detail C3a based on a suitable failure criterion and a well collected database is lacking.

INNOVATION AND STRENGTHENING METHODS

Innovations, *e.g.* inserting inner bulkhead plates [13, 153–157], using high-performance steel [156, 157], optimisation of stiffener shape [119, 158, 159], optimisation of cut-out shape [13, 119, 153–155, 159, 160], and creating smooth transition at the weld toe [13, 154, 155], were made to improve the fatigue performance of crossbeam-to-stiffener welded connection. For instance, a large cut-out height with a smooth weld toe transition, and the inner bulkhead (attached at each stiffener web separately) were proposed by Connor [153] to replace the suggested cut-out in the second version of AASHTO LRFD Bridge Design Specifications based on parametric FEA.

Attaching Carbon Fibre Reinforced Polymer (CFRP) sheets or plates effectively strengthened the fatigue performance of crossbeam-to-stiffener welded connection either with or without cracks [161, 162]. Further improvement was achieved by using Fe-based shape memory alloys (Fe-SMA) plates to introduce pre-compression stress in the repaired regions [163, 164].

3

WELD TOE CRACK AND WELD ROOT CRACK IN STIFFENER OF STIFFENER-TO-DECK PLATE WELDED CONNECTION BETWEEN CROSSBEAMS (C2A & C2B)

Fatigue crack that initiates from the weld toe then propagates into the stiffener web or from the weld root then propagates through the weld throat is studied in Chapter 3. Nineteen stiffener-to-deck plate welded connections with a size of $350 \times 200 \times 200 \text{ mm}^3$ are examined in fatigue. Among them, nine specimens are connected by two-pass automatic weld and ten are connected by one-pass manual weld. The structural (hot spot) stress is calculated using the surface extrapolation approach and the force equilibrium approach for the weld toe and the weld root, respectively. Two detail categories are established using the author's experiments and the results in the literature. Plane strain finite element models are built to predict the crack initiation locations and the fatigue resistances with the partial joint penetration ranging from 25% to 100% using multiple local methods: the effective notch stress, the averaged strain energy density factor, and the linear elastic fracture mechanics.

This chapter is based on the journal paper: *Fatigue behaviour of toe and root stiffener cracks in stiffener-to-deck plate weld of orthotropic bridge decks*, *Engineering Structures*, **305**, (2024).

3.1. INTRODUCTION

An OBD consists of a deck plate supported by longitudinal (traffic flow direction) stiffeners (open or closed shapes) and transverse crossbeams. Directly loaded by wheels in traffic, OBDs experience millions of load cycles during their service lives. A large number of welded connections (joints) cause significant stress concentrations. The combination makes OBDs sensitive to fatigue cracking, mainly initiating from one of the welded connections. The fatigue performance now dominates the design and life-cycle maintenance. Fatigue cracks may initiate from the weld toe or root, and propagate into the base metal or weld metal. The stiffener-to-deck plate welded connection is of special interest because a large fraction of fatigue cracks were detected in this type of connection in practice [14, 122]. Figure 3.1 shows four types of cracks observed in this connection between crossbeams, named according to TS [27]. The fatigue cracks of details C1a, C1b, and C2a propagate into the base metal (main plate). The fatigue crack of detail C2b propagates into the weld metal (weld throat). Weld root cracks usually have lower fatigue resistances as compared with the cracks initiating from the weld toe because of the high notch effect, the lack of control of weld root quality, and the higher crack propagation rate in the weld metal compared with the base metal [165]. However, the local stress ranges relevant to the four types of cracks in Figure 3.1 are not (or not necessarily) equal. The geometry of weld, with the Partial Joint of Penetration (PJP) in percentage, is identified as an important parameter affecting the appearance of crack. A smooth weld profile and sufficient PJP (minimum 75% to 80% of stiffener thickness but not melt-through) appear beneficial for fatigue resistance.

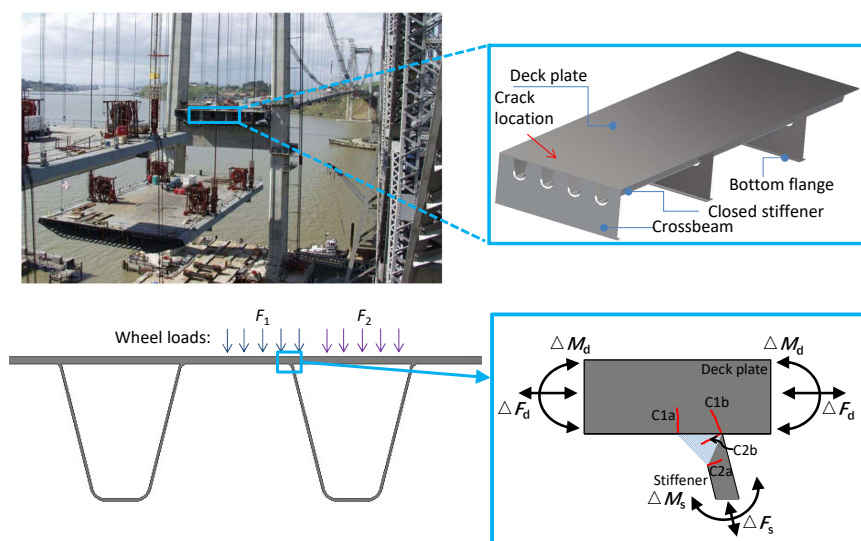


Figure 3.1: Sketch of a representative OBD and stiffener-to-deck plate cracks in details (source construction site photo [2]).

In the stiffener-to-deck plate welded connection, the structural stress method was

used for the evaluation of deck plate using the surface extrapolation approach in [105, 127, 166] or the force equilibrium approach in [143, 167]. The local stress state was evaluated by the ENS in [98, 105, 121]. Fatigue resistance calculation was conducted using the LEFM in [89, 126, 168]. However, most of these studies considered the cracks in the deck plate (types C1a and C1b). A systematic study of the cracks in the stiffener (types C2a and C2b) in modern OBDs using the methods of NS, HSS, ENS, SED and LEFM is lacking. It is unknown if (one of) these methods is suitable for these types of cracks.

Few former studies into details C2a and C2b found in the literature [119–121] were carried out with automatic weld. In actual applications, however, manual overhead welds are also applied to connect OBD panels on site. A series of experiments is, therefore, carried out by the author to study the fatigue behaviour of details C2a and C2b. Automatically welded specimens (Series 1) and manually welded specimens (Series 2) are used to study the effect of welding procedure on the fatigue resistances of the details. Plane strain FE models are built in which the ENS, SED, and LEFM methods are applied to evaluate the effect of different levels of PJP as presented in Section 3.4. The weld crack propagation is analysed by the LEFM using the eXtended Finite Element Method (XFEM). A fatigue assessment is determined from the stress state results using the local methods and the reference values for the associated fatigue resistances in [23, 165, 169]. Using the author's experiments and the results in the literature, the fatigue resistances of details C2a and C2b are quantified by statistical analysis using the surface extrapolation obtained and the force equilibrium obtained structural stress, respectively. The former is also designated as the hot spot stress in (pr)EN 1993-1-9 [16, 17] and TS [27].

3.2. EXPERIMENTAL STUDY

3.2.1. SPECIMEN

Two large welded stiffener-to-deck plate specimens are fabricated using the steel grade S355J2, with a nominal yield stress of 355 MPa [170]. Figure 3.2 provides the dimensions of the large specimens. A 6 mm thick trapezoidal closed stiffener is welded to a $700 \times 1000 \times 20 \text{ mm}^3$ deck plate each. In practice, the deck plate thickness varies significantly (between 10 mm and more than 20 mm), whereas the stiffener is more uniform (6 or 8 mm). Fatigue behaviour of OBDs with 10 mm to 12 mm thick deck plates, commonly used in the past, has been studied [1]. In modern OBDs, thicker deck plates are becoming popular due to the increase in traffic flow and the fatigue problems of existing OBDs. Deck plates ranging from 16 mm to 19 mm can be seen in modern OBDs [2, 171, 172]. In the Netherlands, the Schiphol bridge, the Suurhoff bridge and the intended replacement of the Van Brienenoord bridge use the deck plate with a minimum thickness of 20 mm. The plate thickness affects the cooling rates and thereby the welding-induced residual stress. A 20 mm thick deck plate with a 6 mm thick stiffener is selected to study the fatigue behaviour of stiffener cracks (types C2a and C2b). The effect of deck plate thickness can then be studied by comparing the results of the current study with those of [120]. Figure 3.3 shows the details of the welding procedures. The stiffener edge is chamfered to an angle of 50° . The lack of weld penetration is specified to be 1.5 mm maximum (equal to the PJP of 75% minimum) following [17, 27], whereas melt-through

is not allowed. The actual PJP is hence between 75% and 100%, but probably closer to the lower bound to reduce the risk of melt-through. The difference between the two is the welding procedure applied. Series 1 (S1) is automatically welded by two passes using the Flux-Cored Arc Welding (FCAW) and the Submerged Arc Welding (SAW), whereas Series 2 (S2) is manually welded in one pass using the Shielded Metal Arc Welding (SMAW). Table 3.1 gives the applied welding parameters, with the illustration in Figure 3.4. S1 shows a smoother weld surface than S2. Welds of both series comply with the required quality level, namely, quality level B of ISO 5817 [173]. Nine and ten small specimens are cut, shown in orange in Figure 3.2. Each specimen contains a deck plate with a size of $350 \times 200 \times 20 \text{ mm}^3$ and a stiffener of $200 \times 200 \times 6 \text{ mm}^3$.

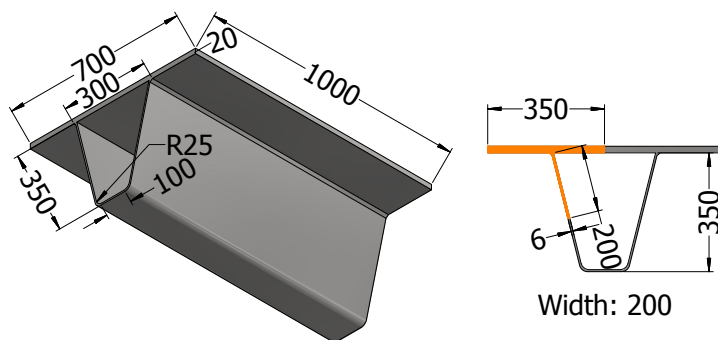


Figure 3.2: Preparation of specimens: left, uncut specimen; right in orange, specimen used in the experiments, (unit: mm).

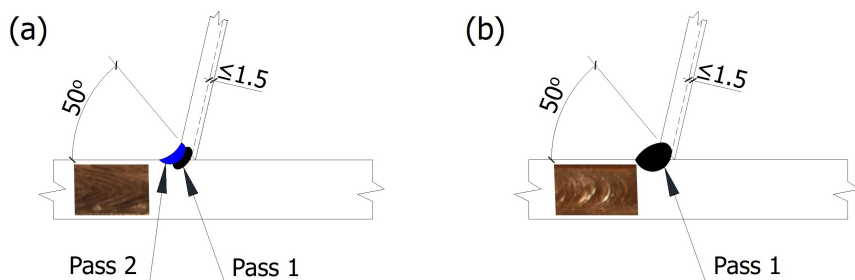


Figure 3.3: Welding procedure for: (a) S1; (b) S2, with representative weld surface photos (unit: mm).

Table 3.1: Welding parameters used in the fabrication of the specimens.

Series	1	1	2
Pass	1	2	1
Process	FCAW	SAW	SMAW
Wire diameter [mm]	1.2	3	4
Current [A]	230-240	400-420	160-170
Voltage [V]	25-27	27-28	23-24
Current type	DC+	DC+	DC+
Welding speed [cm/min]	42-48	50-55	8-10
Heat input [kJ/mm]	0.7-1.0	1.2-1.4	2.3-2.8

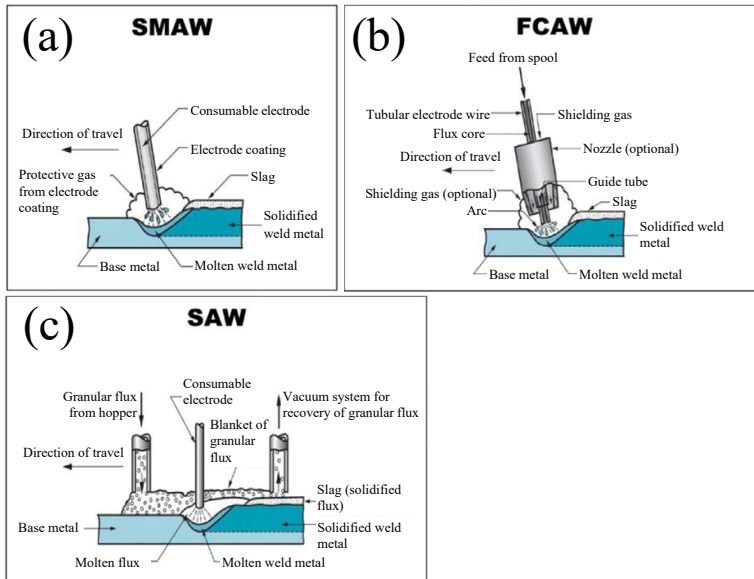


Figure 3.4: Illustration of the applied welding processes: (a) Shielded Metal Arc Welding (SMAW), (b) Flux-Cored Arc Welding (FCAW), and (c) Submerged Arc Welding (SAW) [174].

3.2.2. EXPERIMENTAL PROGRAMME

A load frame is built for cyclic loading, as shown in Figure 3.5. The deck plate is clamped onto an inclined steel plate by bolt heads so that the stiffener is positioned horizontally. Two steel rollers are placed at the edge of the stiffener where the load is applied. Cyclic loading with the load ratio $R = -1$ is applied through a hydraulic jack. The load ratio is relatively high compared with the actual stress condition in OBDs, where the compressive stress of a cycle is usually large compared with the tensile stress. Therefore, a conservative estimate of fatigue performance that covers a wide range of applications can be obtained from the experiments. The applied load ranges are between 2 to 3 kN and differ between specimens. The loading jack position varies slightly between speci-

mens. Note that the roller plate clamping may induce an unknown level of constraint. The boundary conditions of the deck plate, *i.e.* fully supported or clamping at the upper and lower edges, do not affect the stress distribution in the region of interest for details C2a and C2b. The load application in the experiments – a force applied to the stiffener web – deviates from the load applied to the deck plate in a real OBD. However, the crack driving force for this detail is the local distortion of stiffener web [16, 17, 89]. Distortion is induced in both real situations and experiments, Figure 3.6, and the local crack driving forces can be determined for both cases, as explained in Section 3.2.3.

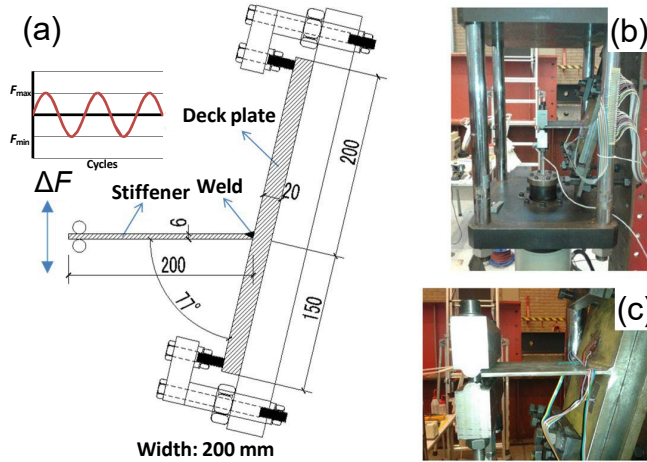


Figure 3.5: Stiffener-to-deck plate welded connection and the setup for cyclic loading (unit: mm).

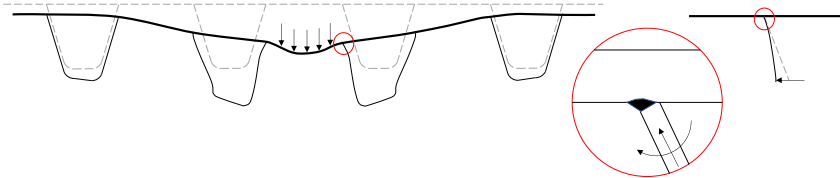


Figure 3.6: Crack driving force for details C2a and C2b due to the local distortion of stiffener web.

Electric resistance strain gauges are attached to the stiffener and the deck plate in the middle of specimen's width, see Figure 3.7. Two types of strain gauges are attached, namely, a single gauge (Type 1) and a five-in-a-row gauge with a distance of 2 mm in between (Type 2). Type 1 strain gauges with label numbers 8, 9, 12, and 13 are placed at the bottom and the top of the stiffener web with distances of 20 mm and 40 mm from the weld root. Type 1 strain gauges with label numbers 1, 2, 10 and 11 are placed at the bottom of the deck plate with distances of 8 mm and 20 mm from the weld toe or the weld root. Type 2 strain gauges are attached at the root side of the stiffener web, with the first strain gauge positioned 4 mm away from the weld root. All gauges are positioned to measure the strains perpendicular to the weld direction.

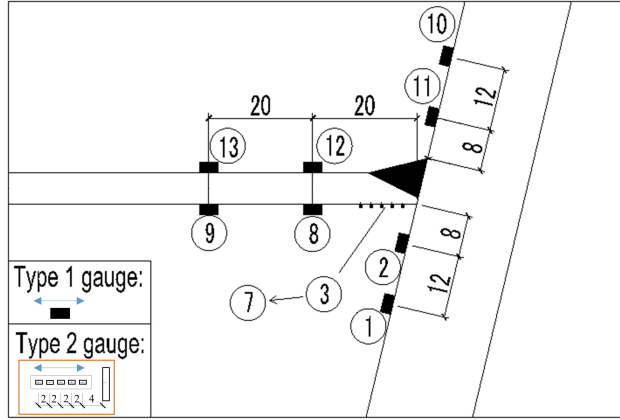


Figure 3.7: Arrangement of the strain gauges in the middle of specimen's width (unit: mm).

3.2.3. STRESS CALCULATION

Equation (3.1) calculates the hot spot stress at the weld toe (detail C2a) using the surface extrapolation approach as applied in the current study where t_s is the thickness of stiffener. The stress ranges are calculated assuming a uniaxial stress state as a conservative reference.

$$\Delta\sigma_{hs} = 1.5 \cdot \Delta\sigma_{0.5t_s} - 0.5 \cdot \Delta\sigma_{1.5t_s} \quad (3.1)$$

The structural stress, consisting of the membrane and bending components, provides a global stress measure for either the weld toe or the weld root [50]. Equation (3.2) gives the calculation of the structural stress in single-sided partial penetration welded connection following the methodology of TS [27], as shown in Figure 3.8. The stress value is a function of the axial force range ΔF and the bending moment range ΔM . The additional bending moment caused by the eccentric axial force is also accounted for. The validation of Equation (3.2) is presented in [175].

$$\Delta\sigma_{wf} = \frac{\Delta F}{b \cdot a_w} + \frac{6}{b \cdot a_w^2} (\Delta M + \Delta F \cdot e) \quad (3.2)$$

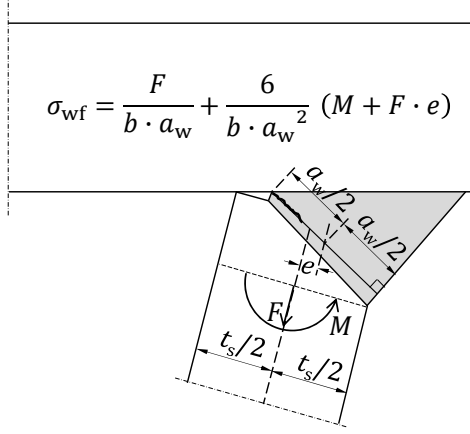


Figure 3.8: Illustration in TS [27] for defining the structural stress in single-sided partial penetration welded connection.

The stress ranges at the weld root (detail C2b), $\Delta\sigma_{wf}$, of the specimens in the current study can be calculated using Equations (3.3) to (3.5).

$$\Delta\sigma_{wf} = \Delta M \frac{6}{b \cdot a_w^2} \quad (3.3)$$

where:

$$\Delta M = 2\Delta M_1 - \Delta M_2 \quad (3.4)$$

$$\Delta M_{1(2)} = \frac{b \cdot t_s^2}{6} E \Delta\epsilon_{1(2)} \quad (3.5)$$

The bending moment range at the weld root, ΔM , is calculated by extrapolating the bending moment ranges at locations of 20 mm (ΔM_1) and 40 mm (ΔM_2) away from the weld root. These bending moment ranges are calculated from the measured strain ranges $\Delta\epsilon_{1(2)}$ from Type 1 strain gauges 8 and 9. Note that the strain ranges obtained from strain gauges 12 and 13 are the same as the values from strain gauges 8 and 9 under the applied loading condition. E is the elastic modulus of steel, for which a value of 210 GPa is assumed. The variables a_w and b are the thickness of weld and the specimen's width, respectively. The eccentricity, e , can be calculated using a_w and t_s as shown in Figure 3.8. The weld throat thickness, a_w , is taken as the average value of twenty measured fracture surfaces at an equal intermediate distance, determined after the complete failure of each loaded specimen as summarised in Table 3.2.

3.2.4. EXPERIMENTAL RESULTS AND FATIGUE RESISTANCES

Two definitions of fatigue life are commonly used in the literature to evaluate the results of details in OBDs. The first approach defines the number of cycles to failure using a change of the strain range measured by the strain gauge closest to the fatigue crack initiation location [1, 98, 123, 176]. However, this definition is sensitive to the distance

between the crack initiation location and the closest strain gauge, and to the stress gradient. The second approach defines the number of cycles to failure by the specimen rupture. This definition is often used in experiments using small-scale specimens and can only be used if crack arrest does not occur. For the loaded specimens presented in this chapter, strain gauges are only placed in the middle of the specimens, whereas the crack initiation may occur at other locations. The experiments are terminated when the specimens are ruptured, except for run-outs if rupture does not take place after approximately 10 million cycles of loading. The second approach is therefore used for the definition of fatigue life in the current study.

Table 3.2 summarises the experimental results. Various load ranges are applied as required for statistical analysis. The structural stress ranges at the weld throat $\Delta\sigma_{wf}$ range between 146 MPa and 359 MPa. The hot spot stress ranges $\Delta\sigma_{hs}$, calculated by Equation (3.1) with the detailed description in Section 3.3, range between 242 MPa and 381 MPa. Because the specimen of S1EXP3 is a run-out, the fracture surface cannot be measured and $\Delta\sigma_{wf}$ is therefore not available. The strain gauges malfunctioned in S1EXP3 and S2EXP3 (with $\Delta\sigma_{hs(/wf)}$ marked by “*”). The average stress $\Delta\sigma_{hs(/wf)}$ normalised to one kilonewton of the other specimens in the corresponding detail group are linearly scaled with the applied forces for these two specimens. Figure 3.9 shows a typical example of observed failure modes. Cracks initiating from the weld toe of the stiffener and propagating into the stiffener (type C2a) are observed near the middle of specimen. Cracks initiating from the weld root and growing through the weld throat (type C2b) are observed near the middle and edges of the specimen. Table 3.2 lists the cracks observed in the fractured specimens.

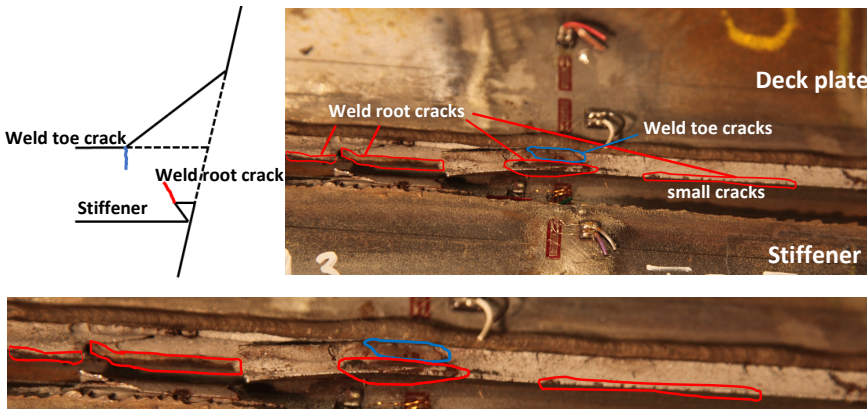


Figure 3.9: Fracture surface of the specimen in S2EXP3.

The number of cycles to failure, N_f , ranges between 0.34 million and 5.31 million. An experimental result is classified as “failure A” if N_f does not exceed 5 million for weld toe cracks or 10 million for weld root cracks. These threshold values for N_f correspond to the assumed constant amplitude fatigue limits in prEN 1993-1-9: 2023 [17]. Fractured specimens with N_f beyond the above mentioned limits are classified as “failure B”. Only results of “failure A” are used in the statistical analysis of fatigue resistance with a fixed

slope, in line with [177, 178]. Specimens showing signs of both weld toe and weld root failures are incorporated in the statistical evaluation of both locations. Specimens with only weld toe failure are considered as run-outs for the weld root failure and vice versa. Similar to the treatment of “failure B”, run-outs are disregarded in the statistical evaluation.

Table 3.2: Summary of experimental results for details C2a and C2b in the current study.

ID	ΔF [kN]	a_w [mm]	$\Delta\sigma_{wf}$ [MPa]	$\Delta\sigma_{hs}$ [MPa]	N_f	Final crack surface observation
S1EXP1	2.00	8.00	156	266	5311008	Root+Toe
S1EXP2	1.99	6.10	282	269	4462209	Root+Toe
S1EXP3	1.99	-	203*	242	9577675	Run-out
S1EXP4	2.26	6.25	315	321	1505820	Toe
S1EXP5	2.44	6.10	306	306	2204211	Root+Toe
S1EXP6	2.41	7.56	209	311	2836078	Root
S1EXP7	2.93	6.75	296	349	1278523	Toe
S1EXP8	2.93	7.48	207	321	2100102	Root+Toe
S1EXP9	2.94	7.01	276	346	1377320	Root+Toe
Avg.		6.91				
S2EXP1	2.04	6.93	215	278	1592180	Root+Toe
S2EXP2	2.04	7.61	181	261	1140391	Root+Toe
S2EXP3	1.99	6.99	199*	254*	1610468	Root+Toe
S2EXP4	1.99	8.01	146	251	1037337	Root
S2EXP5	2.64	6.10	359	356	345712	Toe
S2EXP6	2.51	6.10	332	315	490183	Toe
S2EXP7	2.52	6.84	253	317	358240	Root+Toe
S2EXP8	2.94	6.76	293	363	370673	Root+Toe
S2EXP9	2.93	7.32	242	349	400696	Root+Toe
S2EXP10	2.94	7.33	271	381	341855	Root+Toe
Avg.		7.00				

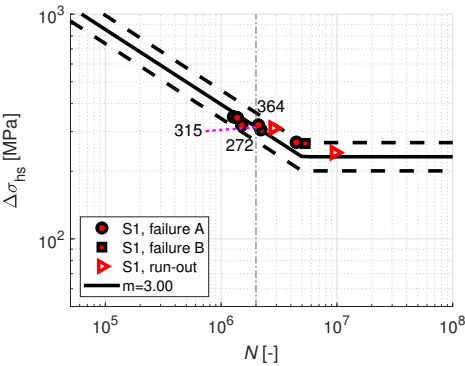
Note:

SXEXPY: represents the Series **EXP**, EXPeriment **Y**.

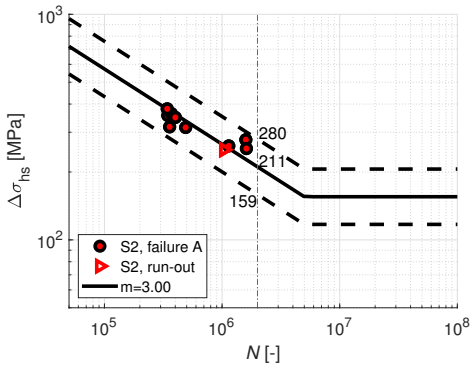
**: Value calculated using the average $\Delta\sigma$ from the corresponding detail group.

Figures 3.10 to 3.11 show the results of statistical analysis, using the Basquin relation $\log(N_f) = a - m \times \log(\Delta\sigma_{hs/(wf)})$ and assuming a fixed slope $m = 3.00$. The fatigue resistance is expressed as the stress range at which two million cycles to failure is obtained, with exceedance probabilities $P_s = 95\%$, 50% , 5% . The 95% value is further referred to as the characteristic fatigue resistance $\Delta\sigma_C$, defined in (pr)EN 1993-1-9 [16, 17] and TS [27], following the statistical analysis described in [177, 178]. A relatively high fatigue resistance with a narrow scatter band ($\Delta\sigma_C = 272$ MPa with the standard deviation $s = 0.09$) is observed for detail C2a of S1 in Figure 3.10a, as compared with S2 ($\Delta\sigma_C = 159$ MPa with $s = 0.14$), Figure 3.10b. Figure 3.11 shows the results of detail C2b. S2 gives a significantly lower fatigue resistance ($\Delta\sigma_C = 106$ MPa with $s = 0.22$) as compared with S1 ($\Delta\sigma_C = 158$ MPa with $s = 0.29$), indicating a better fatigue performance of automatically

welded connections for detail C2b. Note that both weld toe and weld root failures occur; the fact that the fatigue resistance values are different is caused by the different stress definitions of the two details.

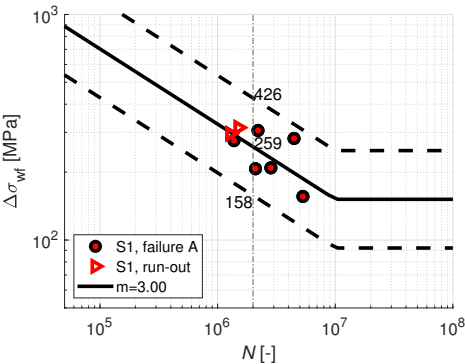


(a) S1, $s = 0.09$

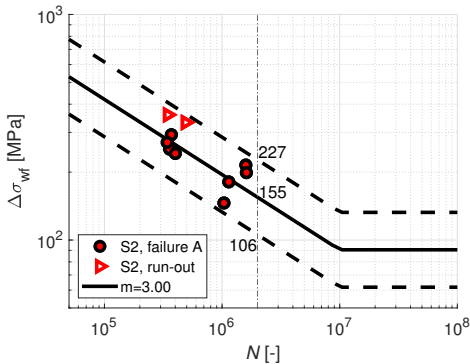


(b) S2, $s = 0.14$

Figure 3.10: Statistical analysis for detail C2a.



(a) S1, $s = 0.29$



(b) S2, $s = 0.22$

Figure 3.11: Statistical analysis for detail C2b.

3.2.5. ADDITIONAL EXPERIMENTAL RESULTS IN THE LITERATURE

YA (2009)

A similar experimental setup and specimen geometries were used at Nagoya University by Ya [120], but using thinner deck plates with automatic weld only. The specimens were 500 mm wide with a 200 mm long stiffener and a 315 mm long deck plate. The specimens were sorted into six groups with deck plate thicknesses of 12 mm, 14 mm, and 16 mm, stiffener thicknesses of 6 mm and 8 mm, and PJPs of 30%, 75%, 80% and melt-through. Only the results with PJPs from 75% to 80% from [120] are used in the current study. The experiments were carried out with the load ratio $R = -1$. Copper wires 0.04 mm in diameter were placed at the weld surface and the weld toe. The experiments were stopped when the wires were cut due to the cracking of the specimens.

A statistical evaluation of automatically welded connections is carried out on the joint database, consisting of experimental results of Ya (2009) [120] and the ones of Section 3.2.4, assuming a fixed slope parameter $m = 3.00$. Figure 3.12 shows that the characteristic values of fatigue resistance are 276 MPa and 142 MPa for details C2a and C2b, respectively. The joint database's fatigue resistances are almost equal to that of the separate database of detail C2a, see Figure 3.10a. On the other hand, the joint database gives lower values (with $P_s = 95\%$ or 50%) for detail C2b. The corresponding standard deviation increases from 0.29 (Figure 3.11a) to 0.33 (Figure 3.12b).

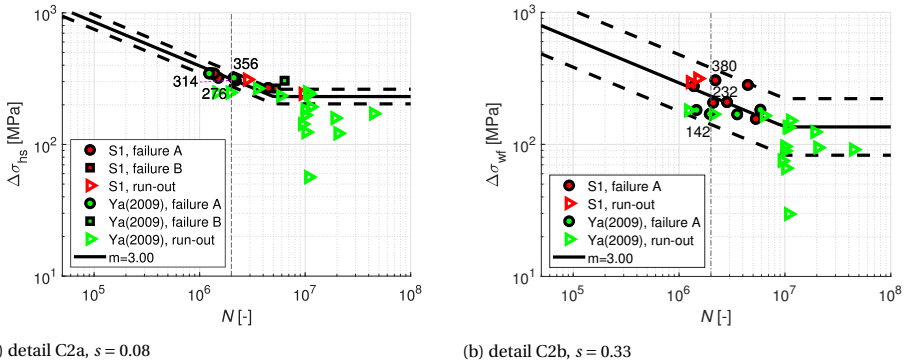


Figure 3.12: Statistical analysis for automatically welded connections, including S1 and Ya (2009) [120].

YUAN (2011) AND BRULS (1991)

Figure 3.13 shows the analysis adding the results from Yuan [121] and Bruls [119]. The stress in [121] was calculated by the FE method. Yuan [121] carried out experiments with a single stiffener-to-deck plate type connection. The stiffener was centrally loaded with $R = 0$ or -1 (only the results with $R = -1$ are used in the current study) and the deck plate was supported at its edges (at the semi-distance between closed stiffeners). The specimens failed from the weld toe (type C2a). Bruls [119] carried out similar experiments. But the stiffener was positioned eccentrically and the load was exerted on the deck plate, with $-1 \leq R \leq 0$. The specimens failed from the weld root (type C2b). The specimens with $75\% \leq \text{PJP} < 100\%$ are used in the current study. Both series used automatic welding.

Figure 3.13 shows the statistical evaluation results of the joint database. $\Delta\sigma_C$ is 158 MPa with $s = 0.32$ for detail C2a. A higher $\Delta\sigma_C = 158$ MPa with a slightly smaller $s = 0.30$ is obtained compared with the values in Figure 3.12b for detail C2b. Combining the data, the resulting characteristic values for details C2a and C2b are shown in Table 3.3.

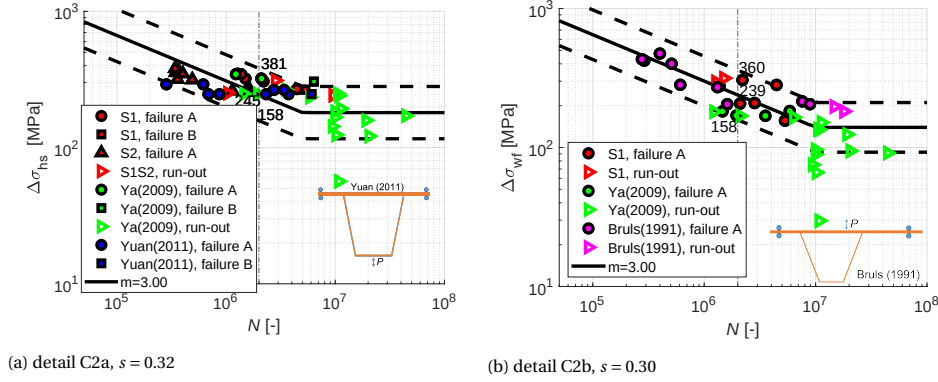


Figure 3.13: Statistical analysis for details C2a and C2b, including the results in the literature [119–121].

Table 3.3: Fatigue resistances derived from all experiments for details C2a and C2b.

Detail	Stress	Welding procedure	Characteristic value $\Delta\sigma_C$
C2a	Equation (3.1)	Automatic&manual	158 MPa (Figure 3.13a)
C2b	Equation (3.2)	Automatic	158 MPa (Figure 3.13b)
		Manual	106 MPa (Figure 3.11b)

3.3. FINITE ELEMENT MODELLING

Section 3.2 experimentally evaluates the fatigue behaviour of details C2a and C2b. Section 3.3 describes the FE study of details C2a and C2b. Plane strain FE models of the specimens used in the current study are built using the commercial software Abaqus 2019 [179]. Displacement and rotation constraints, $UX = UY = URZ = 0$, are used to the top of the deck plate over its entire surface, see Figure 3.14. 8-node plane strain elements with reduced integration, type CPE8R [179], are used with element sizes of approximately $0.5 \times 0.5 \text{ mm}^2$. The PJP equals 75%, *i.e.* the specified (minimum) ratio of the specimens. A possible contact between the stiffener edge and the deck plate is not modelled, as expressed by a 0.01 mm gap as shown in Figure 3.14. Further, the elastic modulus $E = 210 \text{ GPa}$ and the Poisson's ratio $\nu = 0.3$.

The stiffener edge is free to rotate in the FE model, and by a calibration procedure the loading position is determined such, that the computed normal strains are equal to the measured ones. The calibration results in a load position 37.5 mm away from the free edge of the stiffener. The shear strains are verified to be negligibly small in the studied case. $\sigma_{hs} = -131 \text{ MPa}$ and $\sigma_{wf} = 100 \text{ MPa}$ are obtained accordingly, normalised to a unit load of 1 kN (equivalent to 5 N/mm).

Figure 3.15 shows a comparison of the strains between the FE computed (FEA) and the experimentally measured (EXP) in which a scatter of the strain between the measurements is evident. Almost all measured strains are within the bounds set by $\pm 20\%$ offset from the FEA. In both the EXP and the FEA, the strain ranges increase at the bottom side of the stiffener when approaching the welded connection, followed by a significant decrease very close to the weld. This decrease starts from the location of approximately 10 mm from the weld root. It is caused by the lack of penetration and increase in the cross section thickness. The weld toe is, therefore, assumed at a distance of 10 mm from the weld root in calculating $\Delta\sigma_{hs}$ by Equation (3.1).

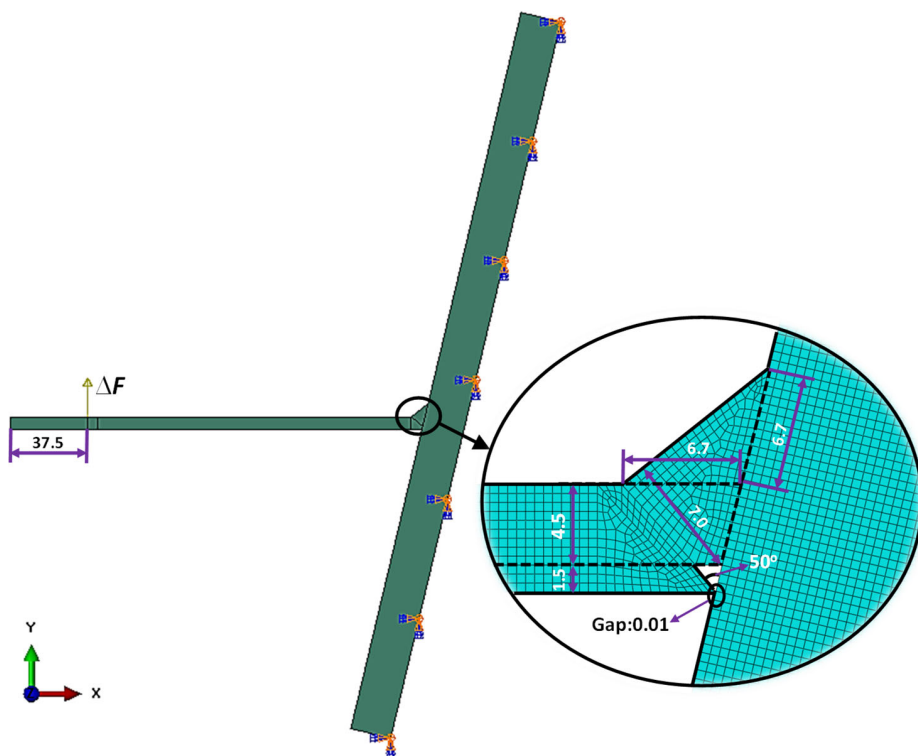


Figure 3.14: Sketch of the FE model for the author's experiments (unit: mm).

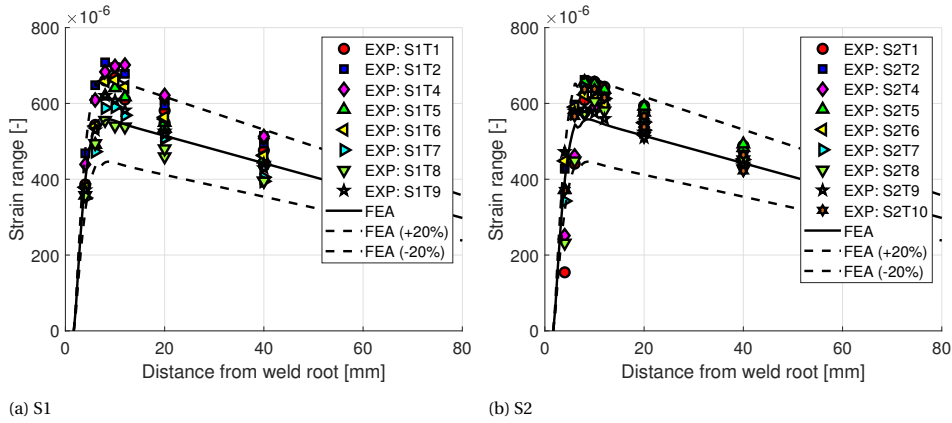


Figure 3.15: Comparison of experimentally measured and the FE calculated strain ranges at the bottom (inner) side of stiffener, normalised to the load range of 5 N/mm.

3.4. PARAMETRIC ANALYSIS USING LOCAL APPROACHES

3.4.1. EFFECTIVE NOTCH STRESS METHOD

The ENS method is developed to evaluate the fatigue behaviour of the weld toe and the weld root by the FE method using the linear elastic material [165]. A fictitious notch with a radius R_{ref} is adopted at the weld toe and the weld root to avoid numerical stress singularities. Either the maximum principal stress or the von Mises stress on the curve of notch radius is referred to as the effective notch stress. The maximum principal stress criterion is better than the von Mises stress criterion for the proportional loading [53]. The effective notch radius R_{ref} and the characteristic fatigue resistance $\Delta\sigma_C$ depend on plate thickness t [165]:

- for $t > 5$ mm, $R_{\text{ref}} = 1.00$ mm and $\Delta\sigma_C = 225$ MPa;
- for $t \leq 5$ mm, $R_{\text{ref}} = 0.05$ mm and $\Delta\sigma_C = 630$ MPa;

where the values of $\Delta\sigma_C$ are valid for the principal stress criterion, and they are defined with a survival probability of 97.7%. This method was applied to analyse the fatigue behaviour of large-scale welded joints and under multi-axial stress states in [180, 181]. The ENS method enables estimating the effect of local geometric characteristics on the fatigue resistance. In the current study, the ENS method is used mainly to study the effect of PJPs of 25%, 50%, 75%, 87.5%, and 100% on the fatigue resistances of details C2a and C2b.

Figure 3.16a gives a close-up of the welded connection in the FE model with PJP = 50% and $R_{\text{ref}} = 1.00$ mm as an example. The predefined PJP is calculated as t_p/t_s where t_p and t_s are the thicknesses of penetration part and stiffener, respectively. The element type, entire (global) geometry, load and boundary conditions are the same as in Section 3.3. Effective notch radii are applied at all four possible crack initiation locations.

The notches for crack locations C2b and C1b coincide in the case PJP = 75%, 87.5%, or 100%. They are then merged, and in addition, simulations are performed using the models with an effective notch radius $R_{\text{ref}} = 0.05$ mm, as shown in Figure 3.16b, even though the plate thickness is larger than 5 mm.

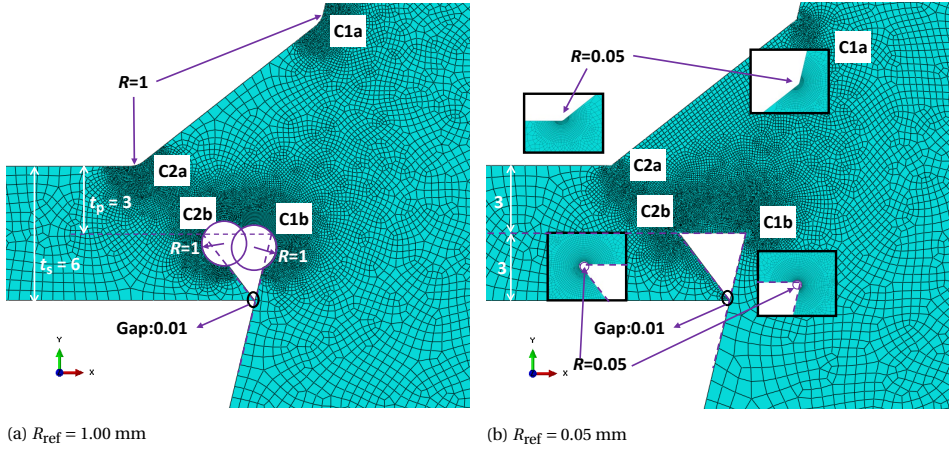


Figure 3.16: Details of the FE models for the ENS calculation with a PJP of 50% (unit: mm).

Figures 3.17 and 3.18 show the contour plots of the maximum principal stresses in absolute values in the vicinity of the welded connections with 1.00 mm and 0.05 mm radii notches under the load of 5 N/mm in the positive Y-axis direction, respectively. Either detail C2a or C2b is critical according to the FEA, depending on the PJP. These findings agree well with the observed fracture locations in the experiments, where both C2a- and C2b-type cracks are observed. The PJP appears to have a significant effect on the principal stress for both details C2a and C2b. Figure 3.17 shows that the maximum principal stress at detail C2b with the PJP of 25% is approximately 2.8 times that of the 50% PJP and approximately 5.5 times that of the 75% PJP. With PJPs of 87.5% and 100%, the values are 0.82 and 0.64 times that of the 75% PJP, respectively.

A similar trend between the minimum principal stress (corresponding to the maximum absolute values in cyclic loading) and the PJP (from 25% to 75%) is found for detail C2a. The value then keeps constant until the 100% PJP. When the notch radius $R_{\text{ref}} = 0.05$ mm is used as shown in Figure 3.18, the maximum principal stresses are 2.0 to 2.4 and 2.5 to 3.4 times larger compared with $R_{\text{ref}} = 1.00$ mm for details C2a and C2b, respectively. Note that the ratio in the fatigue resistance in [165] is 2.8.

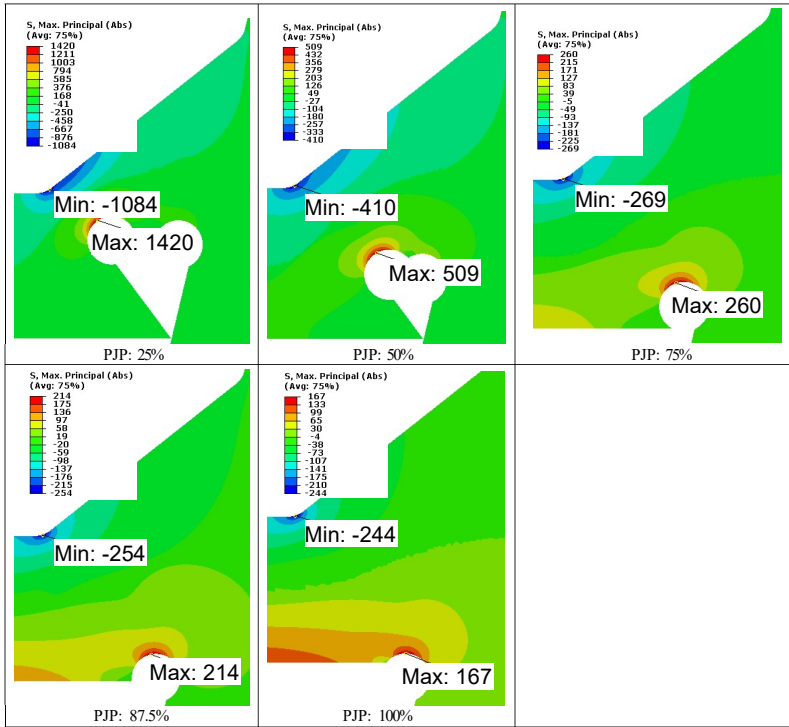


Figure 3.17: Principal stress contour plots of the welded connections with the notch radii $R_{ref} = 1.00$ mm, under the load of 5 N/mm in the positive Y-axis direction (unit: MPa).

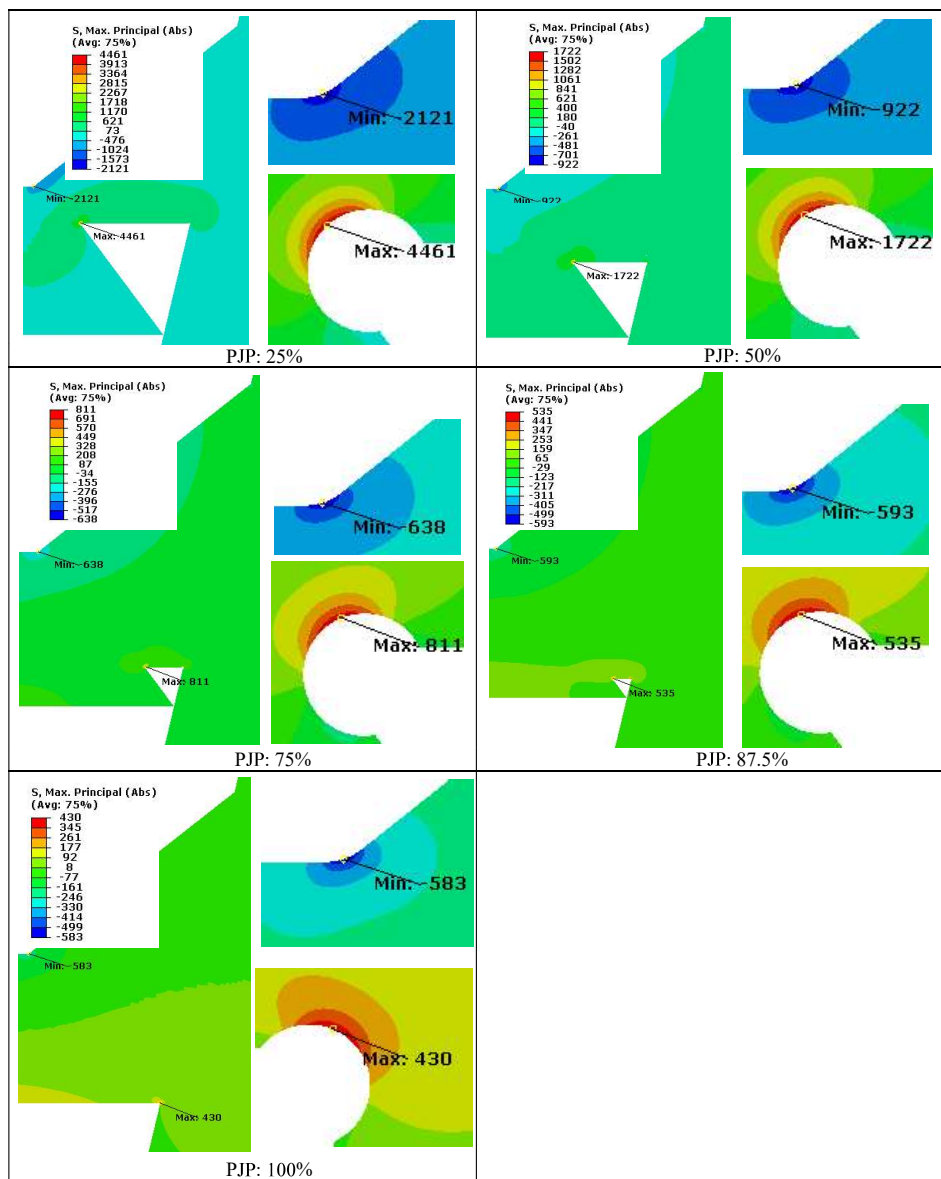


Figure 3.18: Principal stress contour plots of the welded connections with the notch radii $R_{ref} = 0.05$ mm under the load of 5 N/mm in the positive Y-axis direction (unit: MPa).

Figures 3.19 and 3.20 give the maximum principal stress (in absolute value) distributions along the notch holes. The angles at which the maximum principal stresses are located depend slightly on the PJP. The results are summarised in Table 3.4 where the absolute maximum values are shown in bold. Detail C2b is critical for PJPs between 25% and 50%, and detail C2a for PJPs between 87.5% and 100%. For the case with a 75% PJP,

the maximum ENS for a radius of 0.05 mm is at C2b, whereas it is at C2a for a radius of 1.00 mm.

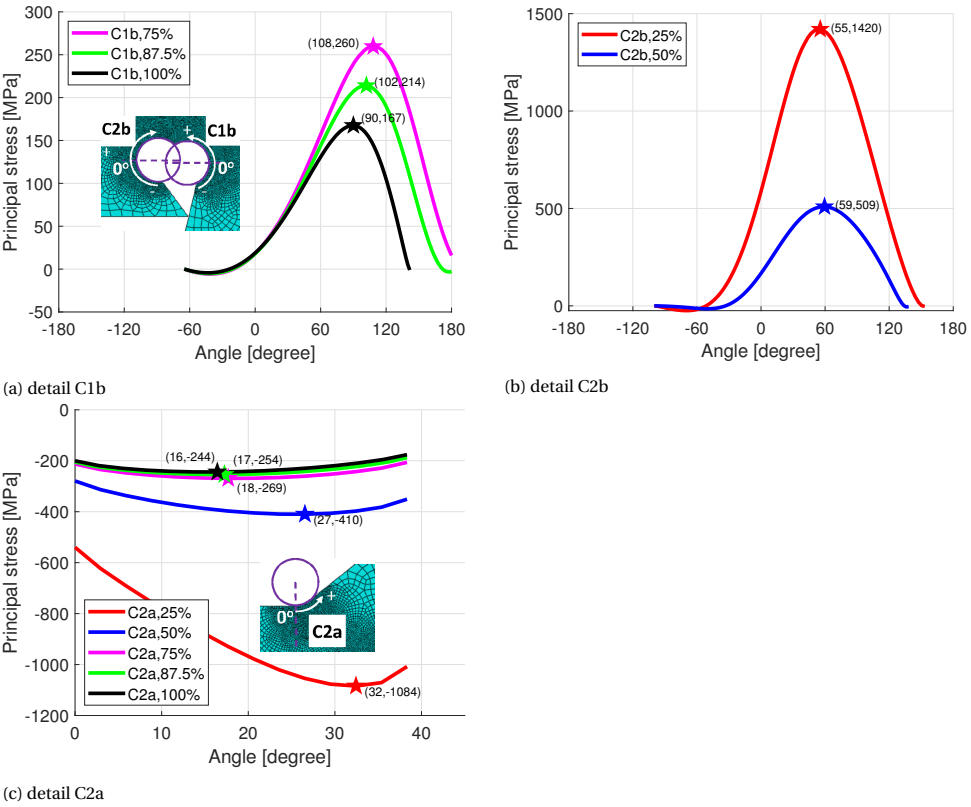


Figure 3.19: Principal stress distributions along the notch holes with $R_{\text{ref}} = 1.00$ mm under the load of 5 N/mm in the positive Y-axis direction.

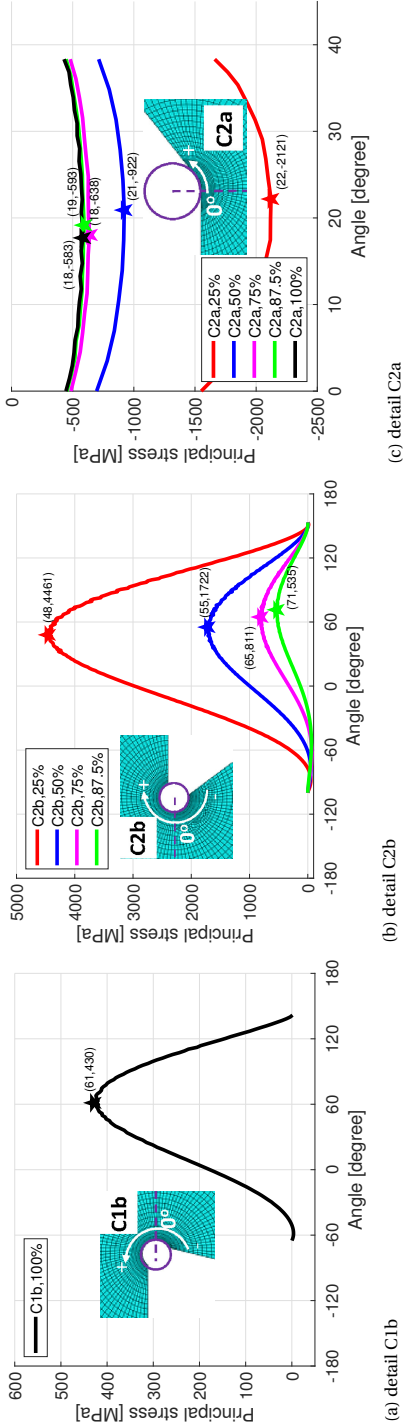


Figure 3.20: Principal stress distributions along the notch holes with $R_{ref} = 0.05$ mm under the load of 5 N/mm in the positive Y-axis direction.

Table 3.4: ENS and SED with different PIPs under the load of 5 N/mm in the positive Y-axis direction with the highest absolute (decisive) values marked in bold (unit: MPa or N·mm/mm²).

PIP	ENS, $R_{ref} = 1.00$ mm				ENS, $R_{ref} = 0.05$ mm				SED ($\times 10^{-4}$), $R_{ref} = 0.28$ mm			
	C1b	C2b	C2a	C1a	C1b	C2b	C2a	C1a	C1b	C2b	C2a	C1a
25.0%	385	1420	-1084	-70	833	4461	-2121	-150	744	15770	9501	54
50.0%	221	509	-410	-52	639	1722	-922	-97	408	2380	1819	30
75.0%	260	260	-269	-41	530	811	-638	-90	279	555	874	19
87.5%	214		-254	-35	496	535	-593	-80	261	264	762	15
100.0%	167		-244	-31	430		-583	-69	181		729	11

3.4.2. AVERAGED STRAIN ENERGY DENSITY FACTOR METHOD

Plane strain FE models similar to Figure 3.16 are built assuming a notch is replaced by a locally refined mesh region with element sizes of approximately $0.02 \times 0.02 \text{ mm}^2$ ([56] illustrates that the SED method is insensitive to element size). The SED averaged over the region enclosed by the circle with $R_{\text{ref}} = 0.28 \text{ mm}$ is calculated as the summed SED over that region divided by its area. The characteristic value of $0.058 \text{ N} \cdot \text{mm}/\text{mm}^3$ at 2 million cycles and the fixed slope parameter of 1.5 are assumed as recommended in [21].

Table 3.4 gives the SED of the aforementioned PJPs. The averaged SED for the weld root cracks (types C1b and C2b) reduces significantly with PJP increasing from 25% to 75%, and it reduces slightly with PJP increasing from 75% to 100%. For the weld toe crack (type C2a), the SED reduces significantly with PJP increasing from 25% to 50%, and then it reduces slightly with further increase of PJP. The critical location shifts from detail C2b to detail C2a with PJP exceeding 75%. The predicted critical locations are the same as the ENS with $R_{\text{ref}} = 1.00 \text{ mm}$.

3.4.3. FRACTURE MECHANICS METHOD

Fracture mechanics is an attractive method to analyse fatigue crack propagation behaviour [23, 165, 182]. In a welded component, flaws caused by manufacturing and/or fabrication can be treated as initial cracks, especially in incomplete penetration welds [183]. In that case, the crack initiation life can be ignored, and the crack propagation life is a reasonable estimate of the total life.

In the current study, the crack propagation is simulated using the XFEM, assuming the plane strain condition for the sake of numerical convenience. The XFEM module in Abaqus is used with the Virtual Crack Closure Technique (VCCT) [179], with phantom nodes to simulate discontinuities in the enriched element. The enriched element is released with zero constraints and zero stiffness after cracking.

MESH AND INITIAL CRACK

Initial cracks with a depth $a_i = 0.1 \text{ mm}$ are inserted at the locations as shown in Figure 3.21 to study detail C2a or C2b. The location and the angle of initial crack follow the maximum principal stress location. The initial angle is obtained in the ENS calculation with $R_{\text{ref}} = 0.05 \text{ mm}$ as shown in Figure 3.20. An exception is the model for simulating the C2b type crack with 100% PJP. The initial crack is applied parallel to the edge of the deck plate (corresponding to the angle of 77° in Figure 3.20a). Different types of elements and mesh sizes are applied to the region of interest (grey colour sub model in Figure 3.21) and the part further away from the welded connection (red colour). These two parts are connected by tie constraints, which implies that the edges have the same displacements [179]. The element size in the sub model is, on average, $0.1 \times 0.1 \text{ mm}^2$ with a denser mesh close to the initial crack location. The element size in the global part is approximately $0.5 \times 0.5 \text{ mm}^2$. Enriched 4-node plane strain elements with reduced integration (type CPE4R) are applied in the sub model, and regular 8-node plane strain elements with reduced integration (type CPE8R) are applied in the remaining part of the model [179].

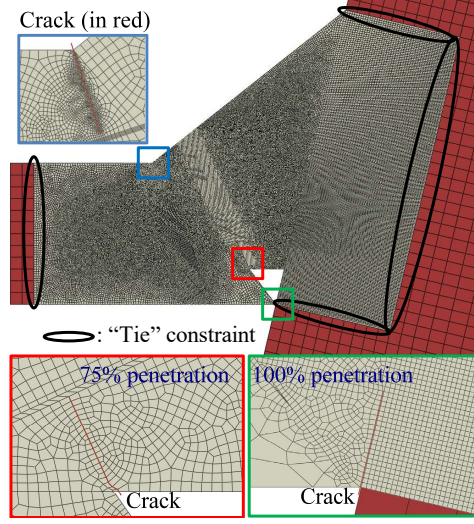


Figure 3.21: Details of the initial cracks $a_i = 0.1$ mm inserted in the XFEM (cracks presented as red wires).

CRACK PROPAGATION

In the calculation, stress intensity factor ranges are above the assumed threshold value, $\Delta K_{th} = 63 \text{ N/mm}^{1.5}$ [23], due to the high load ranges applied in the FEA according to the experiments. The Paris' equation is used to relate the stress intensity factor range to the crack extension per cycle:

$$\frac{da}{dN} = C \cdot (\Delta K_{eq})^m \quad (3.6)$$

where the equivalent stress intensity factor range ΔK_{eq} is calculated by:

$$\Delta K_{eq} = \sqrt{\Delta K_I^2 + \Delta K_{II}^2} \quad (3.7)$$

in which the ΔK_i of fracture modes I and II are calculated by:

$$\Delta K_i = K_{i,max} - K_{i,min}, \quad i = I, II \quad (3.8)$$

The following material parameters are used based on the IIW recommendations [23]:

- $C = 5.21 \times 10^{-13}$ (units: N and mm) for a probability of survival of 97.7%.
- m is 3.00.

The subroutine "UMIXMODEFATIGUE" is used to define the crack propagation model in Abaqus [179]. It uses the strain energy release rate G instead of the stress intensity factor:

$$\frac{da}{dN} = C_3 \cdot (\Delta G_{eq})^{c_4} \quad (3.9)$$

$$G_i = K_i^2 / E' \quad (3.10)$$

where:

$$E' = \frac{E}{1 - \nu^2}, \text{ plane strain} \quad (3.11)$$

$$\Delta G_{eq} = \Delta G_I + \Delta G_{II} \quad (3.12)$$

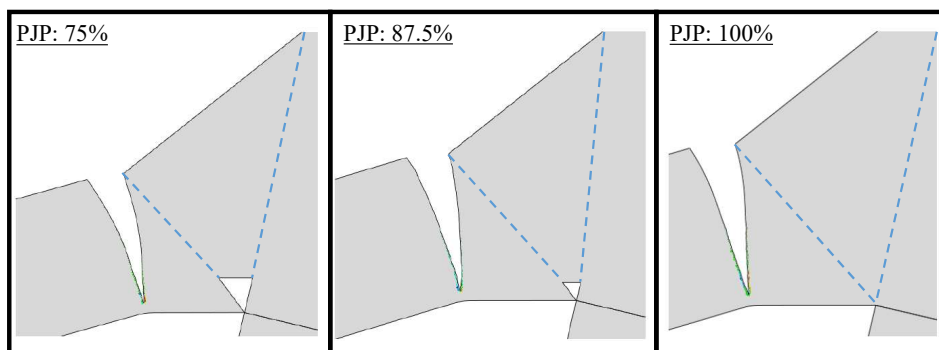
The load ratio $R = -1$ is applied, the same as in the experiments. Two simulations are carried out, which may imply an upper- and a lower bound of reality:

- Fracture Mechanics 1 (FM 1): The cracked part is assumed to be closed and does not propagate in the compressive semi-cycle.
- Fracture Mechanics 2 (FM 2): The cracked part remains open during loading. The complete cycle is effective in crack propagation.

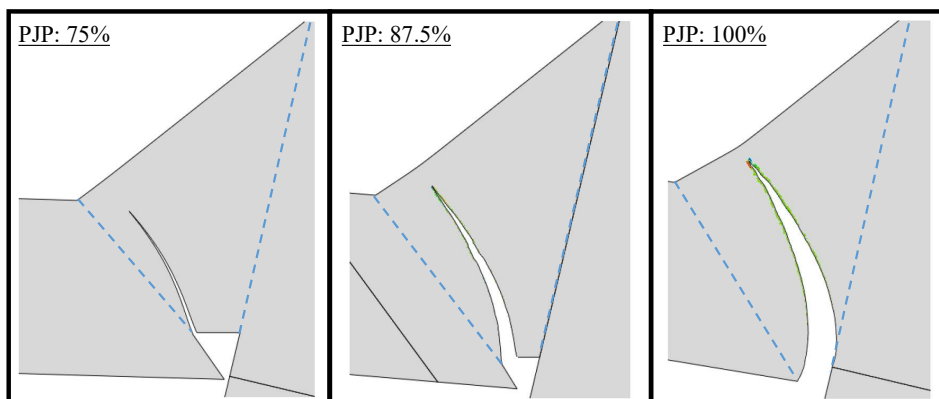
A more detailed analysis including crack closure or the effective stress intensity factor ranges [184] is not carried out because of the large uncertainties of welding-induced residual stress. The Paris' equation constant C for high stress ratio situations is used. The crack increment applied in the simulation is controlled up to 0.03 mm at maximum. The Maximum Tangential Stress criterion is used for the crack propagation direction [179].

RESULTS OF THE XFEM SIMULATION

Figure 3.22 shows the predicted crack paths of the XFEM simulations with PJPs of 75%, 87,5% and 100%. The crack propagates to the bottom (inner) side of the stiffener (Figure 3.22a) and the weld throat (Figure 3.22b) for the weld toe and the weld root cracks, respectively. The predicted fatigue lives under the applied load $F_y = 5$ N/mm are shown in Figure 3.23. The fatigue lives are defined with the number of cycles corresponding to half of plate thickness and half of throat thickness for detail C2a and detail C2b, respectively. The predicted fatigue lives of detail C2a are independent of the PJPs considered, whereas those of detail C2b depend on PJP. These findings are qualitatively in agreement with the results of the ENS analysis. The predicted fatigue lives of the two cracks are approximately equal for the case with a PJP of 75%, which are also observed in the ENS analysis with $R_{ref} = 1.00$ mm. Shorter lives are predicted for detail C2a than for detail C2b in case of higher PJPs, again in line with the ENS analysis.



(a) detail C2a



(b) detail C2b

Figure 3.22: Typical crack propagation paths according to the XFEM simulations with PJPs of 75%, 87.5%, and 100%.

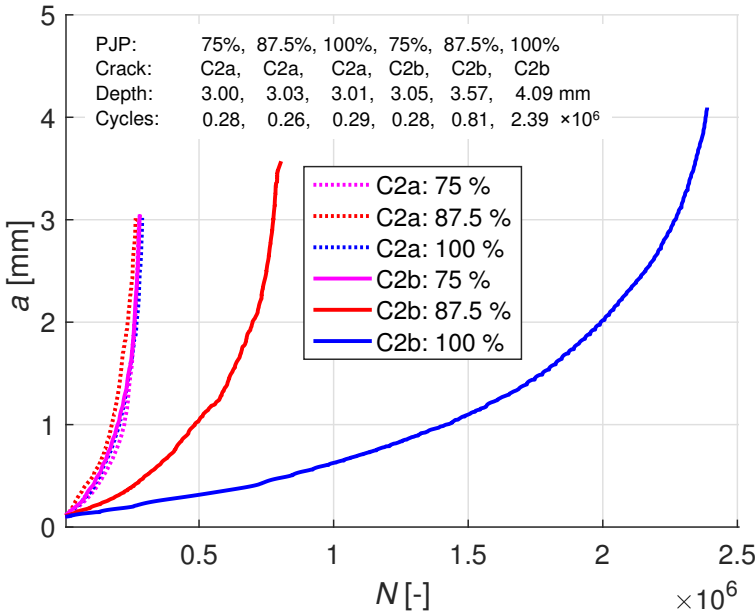


Figure 3.23: Crack propagation simulations of FM 1, with $C = 5.21 \times 10^{-13}$, $m = 3.00$.

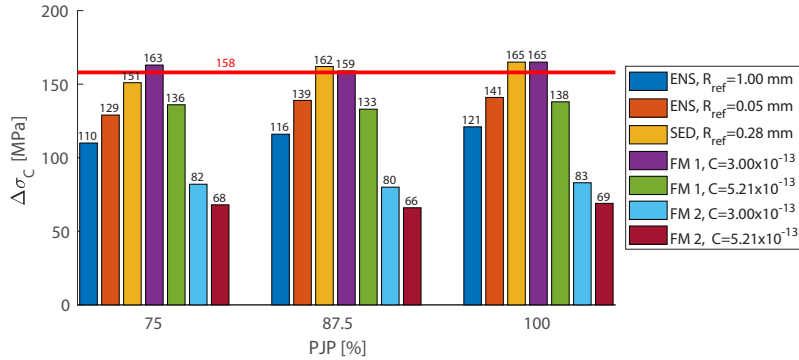
3.5. COMPARISON OF THE FATIGUE ASSESSMENT METHODS

A comparison between the experimentally obtained fatigue resistances of the specimens in Section 3.2 and the predicted resistances according to the local methods in Section 3.4 is presented in this section. The weld throat thicknesses of detail C2b in the FE models are obtained by measuring the distance between the crack initiation location and the fracture surface in the models. The corresponding $\Delta\sigma_{wf}$ in the numerical simulation is calculated under the plane strain condition.

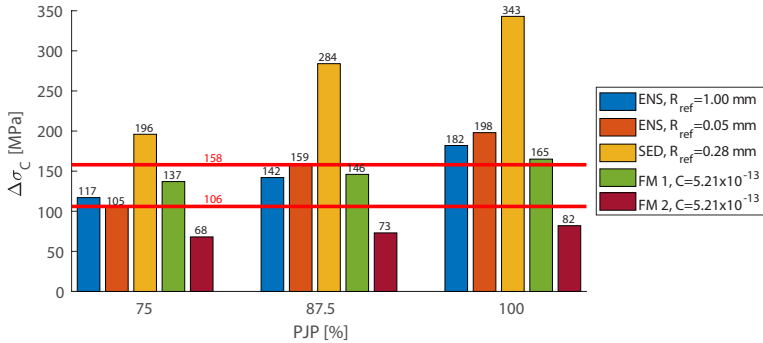
Figure 3.24 shows the values for $\Delta\sigma_C$, calculated by the local methods, with a 97.7% prediction bound, and the values determined by the experiments in Section 3.2, with a 95% prediction bound, all at 2 million cycles. In addition to the commonly recommended $C = 5.21 \times 10^{-13}$, the value $C = 3.00 \times 10^{-13}$ is also used for the weld toe crack that propagates into the base metal [23] and therefore included here.

FM 1 (assuming crack closure at the external compressive stress) with $C = 3.00 \times 10^{-13}$ gives excellent predictions for detail C2a. The differences between the predicted resistances and the experimentally obtained values in Section 3.2 are within 5% with PJPs of 75%, 87.5%, and 100%. The results with the 100% PJP are added for reasons of comparison; the experiments have a PJP between 75% and 100%, but probably closer to the lower bound because melt-through is not allowed (see Section 3.2). For detail C2b and a PJP of 75%, the FM 1 predictions are 13% conservative and 29% unconservative for automatic weld and manual weld, respectively. The imperfection in the manually welded connections may be larger than $a_i = 0.1$ mm, resulting in a lower fatigue resistance experimentally obtained in Section 3.2 compared with the predictions. The results of PJPs of 87.5%

and 100% are added for comparison only, because the weld toe cracks are predicted for such high PJPs. The assumption that the cracked part keeps open under compressive stress appears to underestimate the fatigue resistance predictions of small-scale specimens under high load ranges. The ENS method gives conservative predictions for detail C2a, with $R_{ref} = 1.00$ mm giving lower predictions than $R_{ref} = 0.05$ mm. The SED method predicts lower characteristic fatigue resistances in the range of 151 MPa to 165 MPa. The differences are within 5% of the experimental results. The resistance predictions for detail C2b show a significant increase with increasing PJP. The SED, with a predicted characteristic fatigue resistance of up to 284 MPa, overestimates the resistances with factors of 1.2 and 1.8 for automatic weld and manual weld, respectively, assuming an experimental PJP of 75%. The overestimation is even larger for the PJP of 87.5% or 100%, however, the methods predict type C2a weld toe cracks for these PJPs, so these values are added for comparison only. Nonetheless, the large over-prediction of the experimental fatigue resistances indicates that the SED is not suitable for this type of crack.



(a) detail C2a



(b) detail C2b

Figure 3.24: Comparison between the characteristic fatigue resistances calculated by the local methods (columns) and based on the experiments (lines) for details C2a and C2b ($P_s = 95\%$ in the experiments, and $P_s = 97.7\%$ in the predictions).

3.6. SUMMARY OF THE CHAPTER

Two series of experiments on stiffener-to-deck plate welded connections are conducted to investigate cracks that initiate from the stiffener weld toe and then propagate into the stiffener web (type C2a), and ones that initiate from the weld root and then propagate through the weld throat (type C2b). One series comprises automatically welded specimens, and the other series comprises manually welded specimens. All experiments are carried out in bending with fully reversed load cycles. The Partial Joint Penetrations (PJPs) of the specimens are between 75% and 100%. The following conclusions are drawn from the experimental investigations:

1. Both C2a- and C2b-type cracks are observed in the experiments. Hence, for the load condition and the PJPs applied, their fatigue performance is approximately equal, despite the different local geometries. Type C2b is more dominant in case of small PJPs and vice versa.
2. In the author's own experiments: The automatically or manually welded connections have a characteristic fatigue resistance of 272 MPa ($s = 0.09$) or 159 MPa ($s = 0.14$), respectively, for detail C2a using the (structural) hot spot stress method. The characteristic fatigue resistance of detail C2b is 158 MPa ($s = 0.29$) or 106 MPa ($s = 0.22$) for automatic weld or manual weld, respectively, using the force equilibrium-based structural stress over the throat of weld. Especially, the automatic weld gives significantly higher characteristic fatigue resistances than the recommended values in [16–19] (ranging from 80 MPa to 100 MPa).
3. Using the author's own experiments and the results in the literature, the recommended characteristic fatigue resistances are:
 - **160 MPa** for detail C2a with automatic or manual weld
 - **140 MPa** for detail C2b with automatic weld
 - **100 MPa** for detail C2b with manual weld
4. No influence of the thickness of deck plate ($12 \text{ mm} \leq t_d \leq 20 \text{ mm}$) on the characteristic fatigue resistances of details C2a and C2b is observed.

FE simulations based on the plane strain condition are carried out using the Effective Notch Stress (ENS), the averaged Strain Energy Density (SED) factor, and the Linear Elastic Fracture Mechanics (LEFM) methods, the latter assuming an initial crack depth of 0.1 mm. The effect of PJP is assessed by these methods, bridging the existing knowledge of standard welded connections and the fatigue performance of welded connections in OBDs. An increase in PJP from 25% to 75% significantly reduces the ENS and the SED for both details C2a and C2b. For PJP exceeding 75%, detail C2a is predicted as critical and no obvious change of the ENS and the SED is observed.

When adopting the assumption that the crack is closed under external compressive stress, the LEFM simulation with an initial crack size of 0.1 mm gives excellent predictions for the fatigue resistance of detail C2a (automatic or manual welding) with an overestimation of less than 5% using $C = 3.00 \times 10^{-13}$ (units in millimetre and Newton) and $m = 3.00$. The LEFM method considered in the current study using $C = 5.21 \times 10^{-13}$ (units

in millimetre and Newton) and $m = 3.00$ is useful for automatically welded C2b with PJPs from 75% to 100%. An initial crack size assumption of 0.1 mm may lead to a 29% over-prediction of the fatigue resistance for manually welded C2b. The SED method also gives good predictions for detail C2a, whereas the ENS method gives conservative predictions by up to 18% or 30% using the notch with radius of 0.05 mm or 1.00 mm, respectively. For detail C2b, assuming a PJP of 75%, the resistance prediction with the ENS method is suitable for the manual welding procedure. The weld toe crack (type C2a) is satisfactorily predicted as critical for larger PJPs. The SED method is considered unsuitable for this type of weld root crack (type C2b) because of the large over-predictions compared with the experimental results.

4

WELD TOE CRACK IN DECK PLATE OF STIFFENER-TO-DECK PLATE WELDED CONNECTION BETWEEN CROSSBEAMS (C1A)

Fatigue resistance of the crack that initiates from the weld toe and propagates into the deck plate between crossbeams (type C1a) is studied in Chapter 4. Statistical analysis is carried out for detail C1a using the experiments selected from the literature. One specimen in the literature is used to study the crack propagation behaviour, for which a finite element model is built and the crack propagation is successfully predicted by fracture mechanics in conjunction with the crack propagation model. Geometric correction factors are further developed for the fracture mechanics based hand calculation model with a simplified shape (semi-ellipse) crack considering the straight, convex, or concave weld profile. The hand calculation model with the developed geometric correction factors for the straight weld profile implemented successfully predicts the crack depth propagation and the corresponding load cycles, whereas the predicted crack length growth is slower than the experimental results. Monte Carlo simulations are conducted using the hand calculation model with the geometric correction factors implemented. The generated samples are grouped into the statistical analysis, which gives excellent agreement with the values obtained based on the experiments. The effects of weld profile and deck plate thickness are studied using Monte Carlo simulations. Two standardised fatigue resistances are established for detail C1a with deck plate thicknesses ≤ 14 mm and > 14 mm, respectively.

Section 4.3 is based on the journal paper: *Fatigue resistance of rib-to-deck welded joint in OSDs, analyzed by fracture mechanics*, *Journal of Constructional Steel Research*, **162**, (2019).

4.1. INTRODUCTION

Figure 4.1 illustrates the deformation of stiffener-to-deck plate welded connection under the tyre loading. Four types of cracks (types C1a, C1b, C2a, and C2b) may occur between crossbeams and one type of crack (type C1c) at crossbeam. Fatigue resistance of type C1a crack is studied in this chapter. This type of crack initiates from the weld toe and propagates into the deck plate between crossbeams. Either compressive or tensile stress arises at detail C1a, see Figures 4.1a and 4.1b for the driving bending moments. As introduced in Chapter 3, the compressive stress of a cycle is usually large compared with the tensile stress in a real OBD under traffic loading, and the behaviour of types C1a, C1b, C2a, and C2b cracks are driven mainly by bending moments [81, 93–96].

4

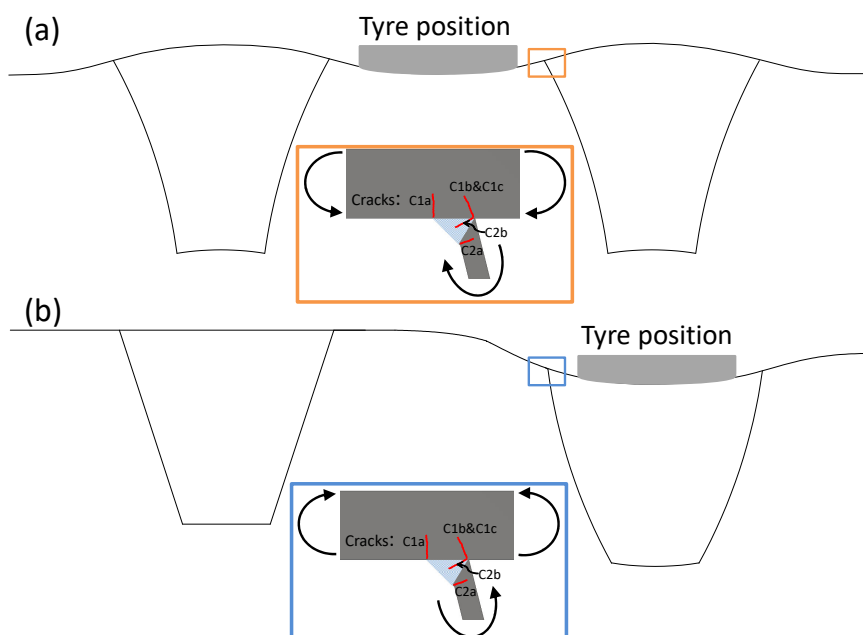


Figure 4.1: Fatigue cracks and the driving bending moments in a stiffener-to-deck plate welded connection under the tyre loading.

Small-scale specimens are widely used to study detail C1a as full-scale experiments are costly and have high requirements for laboratory conditions. Although the stress ratios between the stiffener and the deck plate in small-scale specimens differ from full-scale ones, it is less critical when both the cracks at the stiffener side (details C2a and C2b) and at the deck plate side (details C1a and C1b) are analysed using the TS methodology [27]. The Finite Element (FE) method is adopted in TS [27] that provides realistic stress states for both deck plate and stiffener sides. Thus, it is suitable to study each type of detail separately as long as the stress states of the studied detail are the same as (or close to) the realistic values.

Welded connections inevitably contain initial flaws. The propagation phase occupies a large proportion of total fatigue life. Therefore, fracture mechanics which is developed for crack propagation becomes attractive to the fatigue resistance analysis of welded connections. Moreover, suitable repair and inspection plans can be made based on the assessment of inspected cracks and the prediction of their growth. Aygul *et al.* [67, 185] studied the three dimensional fatigue crack propagation at the weld toe of a bridge detail caused by distortion. Nagy [127] carried out case studies of an existing OBD bridge in Belgium using the extended finite element method. Sanches *et al.* [186] and Correia *et al.* [187] carried out a probabilistic analysis for the crack initiation and propagation of riveted and notched connections, respectively. Lukic and Cremona [188], and Maljaars *et al.* [126] applied probabilistic fracture mechanics to predict the fatigue resistances of welded connections.

The state of a crack tip can be described by stress intensity factors in fracture mechanics [61]. The crack propagation rate (da/dN) can be calculated in conjunction with the Paris' equation as the fatigue propagation model [22]. Either analytical solutions or numerical (*e.g.* FE) methods can be used in the calculation of stress intensity factors. The numerical methods are more suitable for complex geometric and (or) stress situations. The J-integral [63] and M-integral [189] are two main contour integrals [190, 191] used in the numerical methods. Within the LEFM framework, stress intensity factors can be easily obtained by the two integrals. The simplified hand calculation equations have been proposed in guidelines, *e.g.* BS 7910 [24] for typical cases. The equations are developed by fitting the numerical calculated results. Moreover, it is well known that weld profile has an effect on fatigue resistance [17]. However, stiffener-to-deck plate welded connection with various weld profiles is currently not pre-qualified in the guidelines.

In this chapter, the fatigue resistance of detail C1a is statistically analysed based on small-scale experiments selected from the literature. Advanced FE models are created for one specimen in [127]. Using the multi-point extrapolation approach, the crack propagation in the specimen is successfully predicted by fracture mechanics based on the FE calculations. Afterwards, parametric analysis is carried out for detail C1a using simplified semi-elliptical cracks with the straight, convex, or concave weld profile. Geometric correction factors are developed for detail C1a based on the empirical equations proposed by Newman and Raju [192]. The empirical equations were developed for surface cracks in finite-width plates under tension, bending, or a combination thereof. Afterwards, the hand calculation model, in which the geometric correction factors are implemented to empirical equations and the fatigue propagation is calculated in conjunction with the propagation model, is used to predict the crack propagation. Probabilistic fracture mechanics, in which samples are generated by the hand calculation model using Monte Carlo sampling technique, is used to predict the fatigue performance of detail C1a. The generated samples are evaluated by comparing the statistical results with the values based on the experiments. The effects of weld profile and deck plate thickness on fatigue resistance are studied by the validated probabilistic fracture mechanics. Based on this study, two fatigue resistances are standardised for detail C1a with deck plate thicknesses ≤ 14 mm and > 14 mm, respectively.

4.2. FATIGUE RESISTANCE ANALYSIS

Figure 4.2 shows the experimental setup and one specimen used in [127]. The joint-level specimen was fabricated from the structural steel grade S355 with a nominal yield stress of 355 MPa [170]. The 15 mm thick deck plate is stiffened by a 6 mm thick longitudinal closed stiffener. The trapezoidal stiffener has an upper width of 300 mm, a lower width of 150 mm, a height of 275 mm, and a chamfer radius of 24 mm. At the left and right hand sides, the extended widths of the deck plate are 150 mm and 300 mm, respectively. The specimen is 400 mm in the longitudinal weld direction. The gap between the deck plate and the stiffener web, and the lack of penetration are both controlled within 2 mm. Constraints are applied at the left hand side using bolted connections and at the right hand side using rolled plates, respectively. Cyclic loads are applied from the hydraulic actuator, which is connected to the deck plate by steel plates, with a load ratio $R = -1$ or 0. The experiments are aimed to study the fatigue propagation behaviour of the weld toe crack at the stiffener-to-deck plate connection. The beach mark technique, which alternatively applies high and low amplitude load ranges both with the predefined number of cycles, is adopted in the loading programme. This technique controls the fatigue crack propagation rate by the load range in the experiment, which results in visible marks, so called beach marks. After the experiment, the beach marks are measured and linked with the corresponding load cycles, the loading history of the crack is therefore obtained.

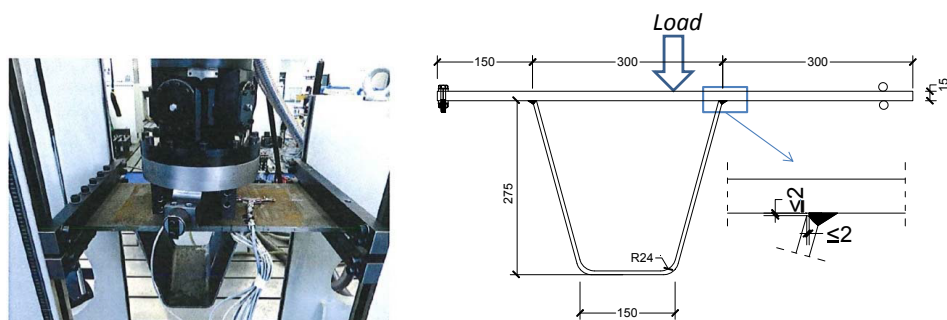


Figure 4.2: The photo and sketch of the setup and one specimen used in [127] (unit: mm).

Figure 4.3 summarises other similar experiments in the literature [105, 107, 108, 110, 121, 166] that study the fatigue behaviour of detail C1a. The deck plate is either simply supported or clamped by rollers at sides, and cyclic load is applied within the longitudinal stiffener. One exception is the setup used in Yuan (2011) [121] in which the load is applied from the bottom of the stiffener. It should be noted that narrow specimens are used in Yuan (2011) [121] and Li (2018) [110] with widths of 150 mm and 108 mm in the longitudinal weld direction, respectively. For others, the widths of the specimens range from 400 mm to 600 mm. The mild strength steel is used in all the experiments. Table 4.1 lists the available material properties provided in [105, 108, 110]. $\Delta\sigma_{hs}$ is linearly extrapolated using the measured strain ranges multiplying the elastic modulus of steel at the predefined locations, an exception is Yuan (2011) [121], in which the stress ranges are calculated by the FE models. The penetration ratio affects the fatigue perfor-

mance of the detail. The Partial Joint of Penetration (PJP) in the range between 75% and 95% is required in prEN 1993-1-9 [17] for partial welded stiffener-to-deck plate connection. This criterion is used in the current study. Only the experimental results with PJP approximately larger than 70% ($PJP \geq 67\%$ in [127]) and load ratio $R < 0$ are selected for statistical analysis.

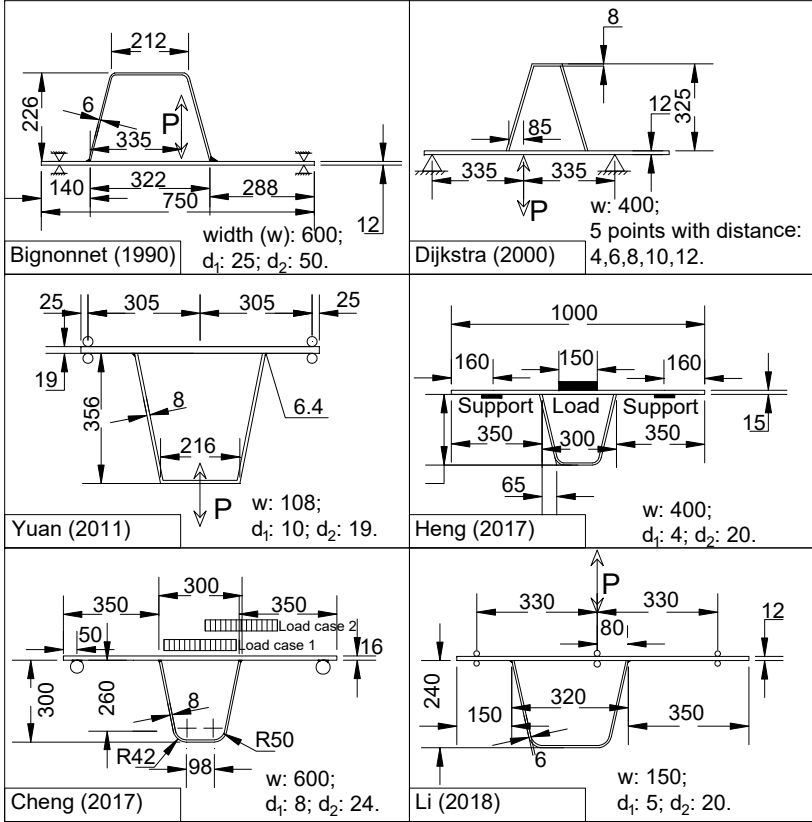


Figure 4.3: Summary of the setups and specimens used in the small-scale experiments for detail C1a (unit: mm) [105, 107, 108, 110, 121, 166].

Table 4.1: Material properties of the specimens in the literature [105, 108, 110].

Source	σ_{yield} [MPa]	σ_{ultimate} [MPa]
[105], 6 mm thick stiffener	400	495
[105], 16 mm thick deck plate	353	508
[108]	345	510
[110]	281	424

Figure 4.4a summarises the results from Bignonnet (1990) [107], Dijkstra (2000) [166], Yuan (2011) [121], Nagy (2017) [127], and Li (2018) [110]. $R = -1$ was used for all the experiments except in Dijkstra (2000) [166], where $R = -0.66$. The fixed slope $m = 3.00$ (fully informative prior), in line with (pr)EN 1993-1-9 [16, 17], is used in the statistical evaluation with the procedure described in [177, 178]. This results in a hot spot stress range $\Delta\sigma_{hs} = 196$ MPa at 2 million cycles with a 50% exceedance fraction ($P_s = 50\%$). The characteristic ($P_s = 95\%$) and upper bound ($P_s = 5\%$) values are 141 MPa and 273 MPa, respectively. Thin deck plates ($t_d = 10$ or 12 mm) have slightly higher values: 149 MPa, 196 MPa, and 258 MPa for $P_s = 95\%$, 50%, and 5%, respectively. The standard deviation s is 0.25 for the 10, 12, 15, or 19 mm thick deck plates, which decreases to 0.20 for the 10 or 12 mm thick deck plates.

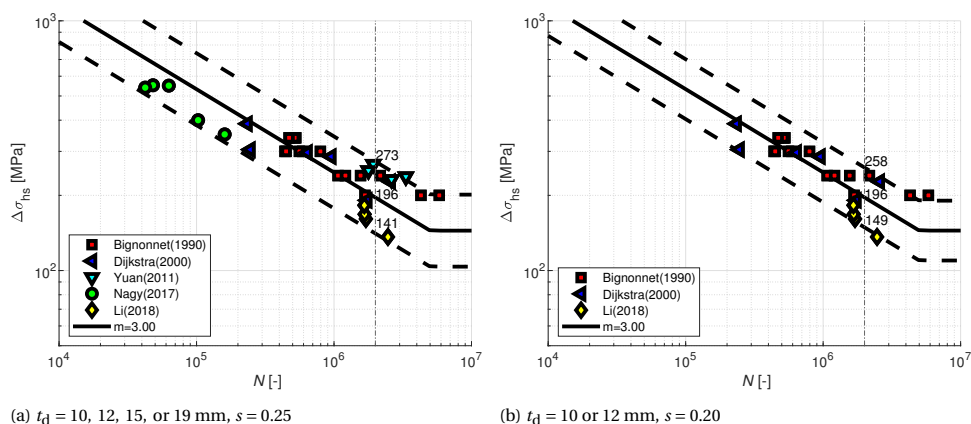


Figure 4.4: Statistical analysis for detail C1a using the experiments selected from the literature with load ratio $R < 0$ [107, 110, 121, 127, 166].

4.3. DEVELOPMENT OF GEOMETRIC CORRECTION FACTORS

4.3.1. NUMERICAL MODEL

LOADING AND BOUNDARY CONDITIONS

Figure 4.5 shows a three-dimensional FE model created by the commercial software package Abaqus 6.14-1 [190] with the dimensions, loading, and boundary conditions introduced in Figure 4.2. The detail is simplified with PJP = 100% in the current study. This simplification is suitable as the crack initiates from the weld toe at the deck plate side. The stress state at the weld toe keeps approximately constant when $PJP \geq 75\%$. The load range of 31 kN (minimum 0 kN and maximum 31 kN) is applied on the FE model following specimen 10 (S10) in Nagy's study [127]. Contact of the loading plates and the deck plate is simulated in Abaqus [190] using the "hard contact" in the normal direction and the "penalty function" with a friction coefficient of 0.3 in the tangential direction. The initial clearance between the loading plates and the specimen is assumed as 0.25 mm.

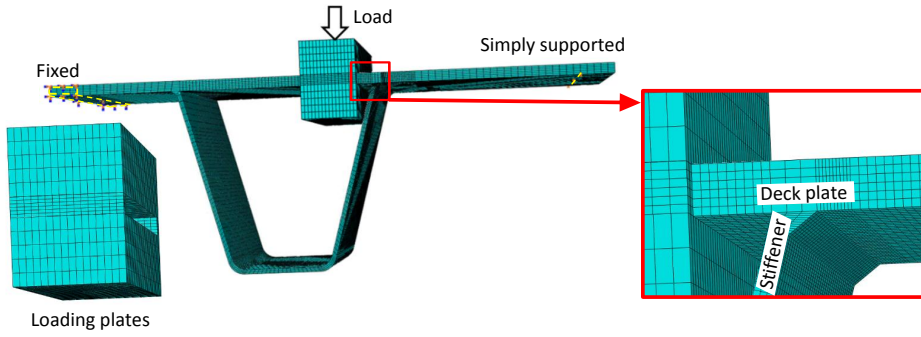


Figure 4.5: FE model without crack for the setup and specimen in [127].

MESHING AND ELEMENT TYPES

Linear elastic material with the elastic modulus $E = 210$ GPa and the Poisson's ratio $\nu = 0.3$ is used in the FE model. 20-node quadratic brick elements (type C3D20) are used for the specimen which are suitable to model the local stress concentration of linear elastic material [190]. 8-node linear brick elements (type C3D8) are used for the loading system to reduce computational cost [190]. Figure 4.6 shows the sketches of the element types used in the FE model.

Figure 4.7a shows a half of the inserted crack with the depth a and the semi-length c at the weld toe location, perpendicular to the bottom surface of the deck plate. When the crack is inserted into the model, a sub model is created by software FRANC3D [191], which is connected to the rest part by merging the nodes at intersections. The sub model is meshed by 10-node quadratic tetrahedron elements (type C3D10) and will be remeshed during the crack propagation. Around the crack front, “one ring” of 15-node quadratic triangular prism elements (type C3D15) and two “adjacent rings” of type C3D20 elements are created [190], see Figure 4.7b. The three rings of elements are used to calculate stress intensity factors in FRANC3D [191], which will be described in Section 4.3.2.

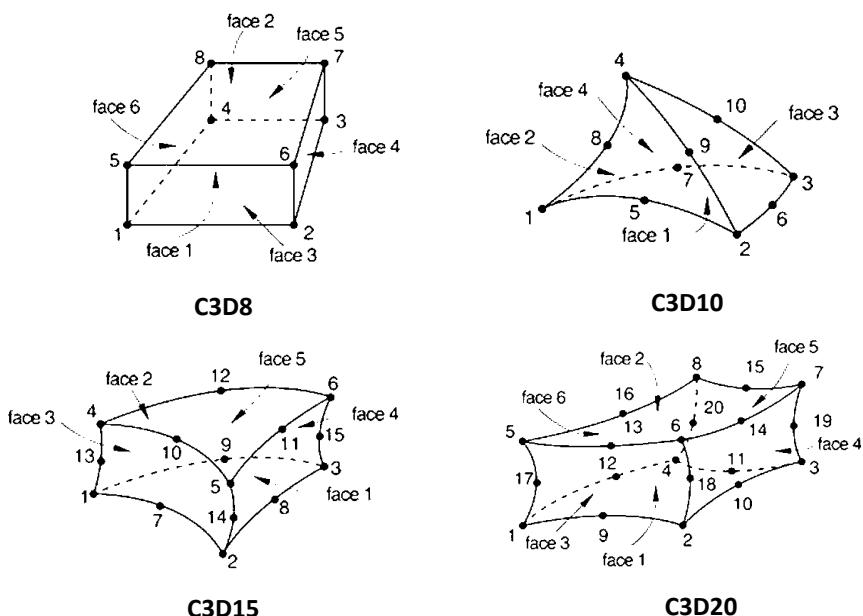


Figure 4.6: Sketch of the elements used in the FE model [190].

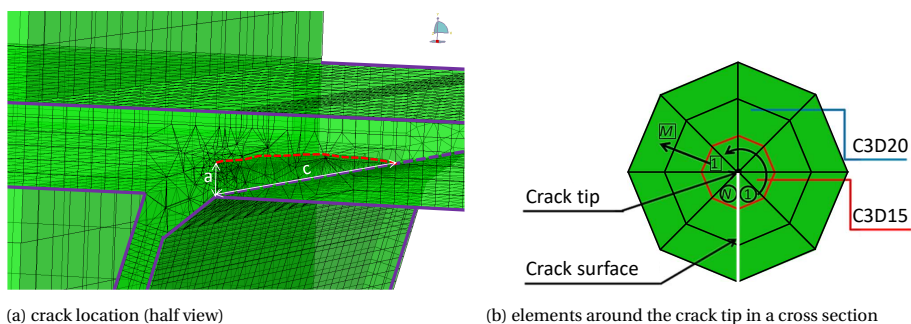


Figure 4.7: FE model with an inserted crack in the specimen in [127].

4.3.2. CRACK PROPAGATION CALCULATION USING FRANC3D

STRESS INTENSITY FACTOR CALCULATION

The M-integral is used to calculate stress intensity factors for a mixed-mode problem. A relationship between the J-integral and the M-integral is shown by Equations (4.1) and (4.2). When considering two independent equilibrium states with superscripts “1” and “2” representing two independent equilibrium states, superposition of the two states leads to a new equilibrium state “0” [189]:

$$J^{(0)} = J^{(1)} + J^{(2)} + M^{(1,2)} \quad (4.1)$$

where:

$$M^{(1,2)} = \int_{\Gamma} (W^{(1,2)} dy - [T_i^{(1)} \frac{\partial u_i^{(2)}}{\partial x} + T_i^{(2)} \frac{\partial u_i^{(1)}}{\partial x}] ds) \quad (4.2)$$

in which $W^{(1,2)}$ is the mutual potential energy density of an elastic body, $T_i^{(1)}$ and $T_i^{(2)}$ are surface tractions, $u_i^{(1)}$ and $u_i^{(2)}$ are displacement vectors [189]. Γ is an arbitrary path close to the crack tip. The origin of the coordinate system is located at the crack tip. ds and ∂x represent the differential of arc length along path Γ and the component in the X-axis direction, respectively.

The M-integral is numerically calculated by Equation (4.3) in the FE method.

$$M^{(1,2)} = \sum_{n=1}^N \sum_{m=1}^3 [F_n H_m] \Delta S_n \quad (4.3)$$

in which F_n is the integral function, H_m is the weight coefficient of Gaussian quadrature formula (m is the number of circles), and ΔS_n is a segment of the integration path within n th element (n is the number of elements in one ring) [189, 193].

According to Yau *et al.* [189] and Wawrzynek *et al.* [193], stress intensity factors K_I , K_{II} , and K_{III} , can be calculated by the M-integral using Equation (4.4). Parameter A_q is the area of virtual extension along the crack front and superscripts 2a, 2b, 2c represent fracture modes I, II, and III, respectively. Note that when the crack front is at (or close to) surface, $(1 - \nu^2)/E$ needs to be replaced by E due to the change from the plane strain to the plane stress condition.

$$\begin{pmatrix} 2(1 - \nu^2)/E & 0 & 0 \\ 0 & 2(1 - \nu^2)/E & 0 \\ 0 & 0 & 2(1 + \nu)/E \end{pmatrix} \begin{Bmatrix} K_I \\ K_{II} \\ K_{III} \end{Bmatrix} = \begin{Bmatrix} M^{1,2a} / A_q \\ M^{1,2b} / A_q \\ M^{1,2c} / A_q \end{Bmatrix} \quad (4.4)$$

CRACK PROPAGATION CALCULATION

When applying fracture mechanics to fatigue life prediction, an initial crack needs to be inserted as a starting point. The relationship between the crack propagation rate da/dN and the stress intensity factor range ΔK , is empirically described by the Paris' equation (the most basic fatigue propagation model) [22], see Equation (4.5), where C and m are empirical material parameters obtained from experiments [61]. In this section, ΔK is

assumed to be larger than the threshold value ΔK_{th} due to relatively high load levels applied in the studied experiment [127].

$$\frac{da}{dN} = C(\Delta K)^m, \Delta K > \Delta K_{th} \quad (4.5)$$

Figure 4.8 shows the multi-point extrapolation approach used in FRANC3D [191] for crack propagation analysis. In every increment, the crack propagation at the deepest point, Δa_m is predefined. Afterwards, the propagation at other points is obtained by Equation (4.6), which is derived from Equation (4.5). In the current study, the material parameters are set as $C = 2.47 \times 10^{-13}$ (units in millimetre and Newton) and $m = 3.00$ based on the values provided in [127], and the effective stress intensity factor range (ΔK_{eff}) is equal to the stress intensity factor range in mode I (ΔK_I).

$$\Delta a_i = \Delta a_m \cdot \left(\frac{\Delta K_{eff,i}}{\Delta K_{eff,m}} \right)^m, i = 1 \dots n. \quad (4.6)$$

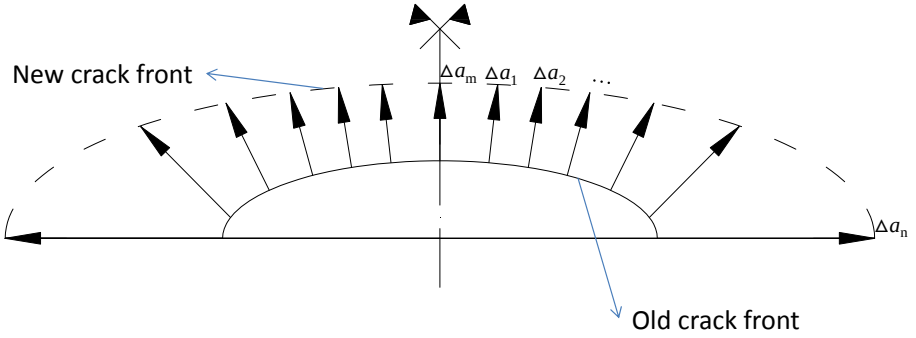


Figure 4.8: Sketch of the crack propagation approach used in FRANC3D [191].

Figure 4.9 compares of the predicted marks (solid green lines) and the measured beach marks (dash black lines) of S10 in Nagy's experiments [127]. Good agreement is found between the predictions and the measurements, in both crack shapes and fatigue lives. This indicates that it is suitable to apply the multi-point extrapolation in conjunction with the Paris' equation to predict the fatigue crack propagation of type C1a crack. Note that the intersection angle of the crack front and the surface, θ , is small. A properly assumed crack shape from the beginning is important to the simulation of crack length.

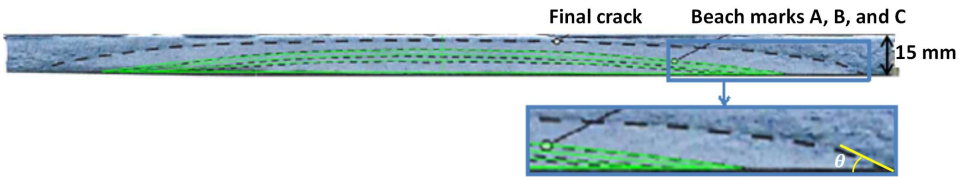


Figure 4.9: Comparison of the predicted cracks (solid green lines) and the measured beach marks (dash black lines) of S10 in [127].

STRESS INTENSITY FACTOR RANGES ALONG THE CRACK FRONT

Figure 4.10 shows the distribution of ΔK_I along the crack fronts at the calculated seven increments. In the beginning, the maximum values locate at the deepest points ($\phi = \pi/2 = 1.57$) of the crack fronts. When the crack depth a grows close to the half thickness of the deck plate ($a = 7.06$ mm), ΔK_I increases faster at $\phi = 0.6\pi/2 = 0.94$. It indicates a change of crack shape. At $\phi = \pi/2 = 1.57$, ΔK_I grows from $689 \text{ N/mm}^{3/2}$ to $748 \text{ N/mm}^{3/2}$ (increment of $59 \text{ N/mm}^{3/2}$) when a increases from 4.88 mm to 7.06 mm and then grows at a slower rate until $a = 8.88$ mm (increment of $13 \text{ N/mm}^{3/2}$). Increments of $77 \text{ N/mm}^{3/2}$ and $84 \text{ N/mm}^{3/2}$ are found at $\phi = 0.6\pi/2 = 0.94$ in the first four and last three increments, respectively.

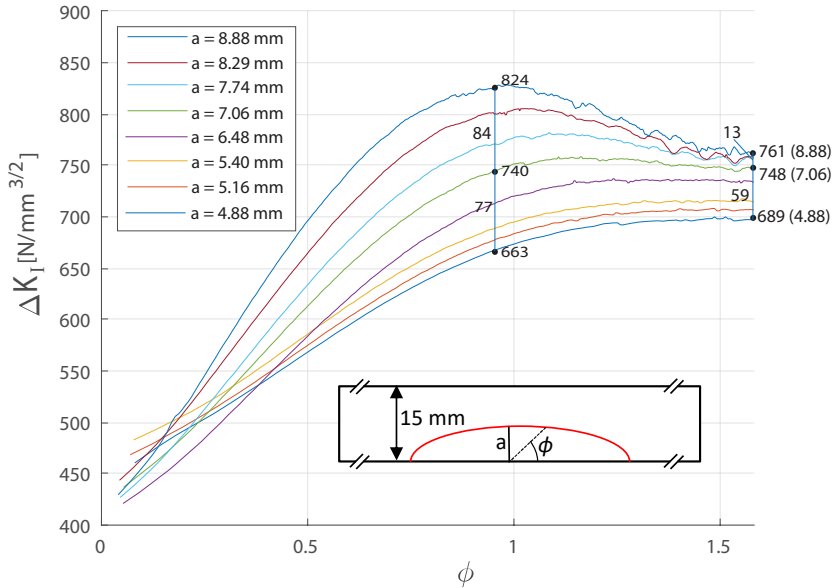


Figure 4.10: ΔK_I at the calculated crack fronts.

4.3.3. HAND CALCULATION OF FATIGUE CRACK PROPAGATION

Although the crack extrapolated by multi-points in conjunction with the Paris' equation gives good predictions for both crack shapes and fatigue lives, a crack following the shape observed in the experiment needs to be introduced. In current design guidelines, e.g. according to the IIW recommendations [23] and BS 7910 [24], the surface crack shape is described by the depth a and the semi-length c of a semi-ellipse, see Figure 4.11. In order to be consistent with engineering practice, the simplified shape (semi-elliptical) cracks are used in the following sections of this chapter. Only two points, the deepest point and the surface point, are used to identify the crack shape. Similar to the aforementioned approach, the increment in depth Δa is pre-defined and the corresponding load cycles are calculated by the Paris' equation. The propagation of semi-length Δc can be calculated by Equation (4.6) accordingly. One major advantage of this simplification is that stress intensity factors can be calculated by empirical equations [23, 24, 192]. This

approach gives a good prediction of semi-elliptical crack shape propagation under tension and bending [194].

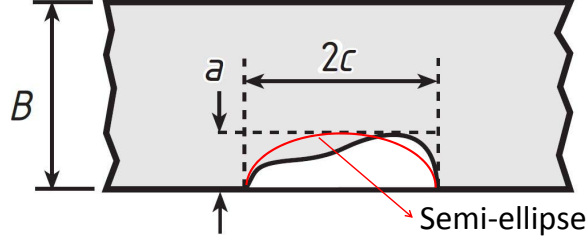


Figure 4.11: Typical crack shape simplification in BS 7910 [24].

Equations (4.7) and (4.8) are developed to calculate the stress intensity factor range of a semi-elliptical surface crack in a plate [24, 192]. When analysing different shapes of connections that consist of complex details, it is necessary to correct the basic Equation (4.7) with the geometric correction factor M_k , see Equation (4.9). Under tension and bending, the membrane correction factor (M_m) and the bending correction factor (M_b) need to be used, respectively.

$$\Delta K_I = (M_m \Delta \sigma_m + M_b \Delta \sigma_b) \cdot \sqrt{\frac{\pi a}{Q}} \quad (4.7)$$

where $\Delta \sigma_m$ and $\Delta \sigma_b$ are the membrane and the bending stress ranges, respectively. Q is the flaw shape parameter according to [192]:

$$Q = 1.0 + 1.464 \frac{a}{c} \quad (4.8)$$

$$\Delta K_I = M_k (M_m (1 - \Omega) + M_b \Omega) \Delta \sigma_{hs} \sqrt{\frac{\pi a}{Q}} \quad (4.9)$$

in which $\Delta \sigma_{hs}$ is the hot spot stress range and Ω equals $\Delta \sigma_b / (\Delta \sigma_b + \Delta \sigma_m)$, the proportion of bending stress in $\Delta \sigma_{hs}$ ($\Delta \sigma_{hs} = \Delta \sigma_b + \Delta \sigma_m$).

Note that the deck plate is strengthened by longitudinal stiffeners, which causes local stress concentrations at detail C1a. Bending stress is the dominant component at the crack initiation cross section. Ω is therefore set to 1 in the current study.

4.3.4. GEOMETRIC CORRECTION FACTORS FOR HAND CALCULATION

Figure 4.12 shows three types of weld profiles, Type 1, Type 2, and Type 3, studied in this section. Type 1 profile represents the most commonly assumed weld profile. Type 2 and Type 3 profiles replace the straight line by a concave and convex arc as the most favourable and unfavourable cases, respectively. The effect of weld profile deviation on the fatigue resistance is studied. Geometric correction factors M_k for the three types of weld profiles are obtained based on the FE calculations. σ_{hs} calculated from the Type 1 model is therefore applied to curve fitting for all three types of weld profiles. It is known

that the strength of weld throat may be weakened for Type 2 profile due to the removal of weld material. Research of that topic is left outside the scope of the current study.

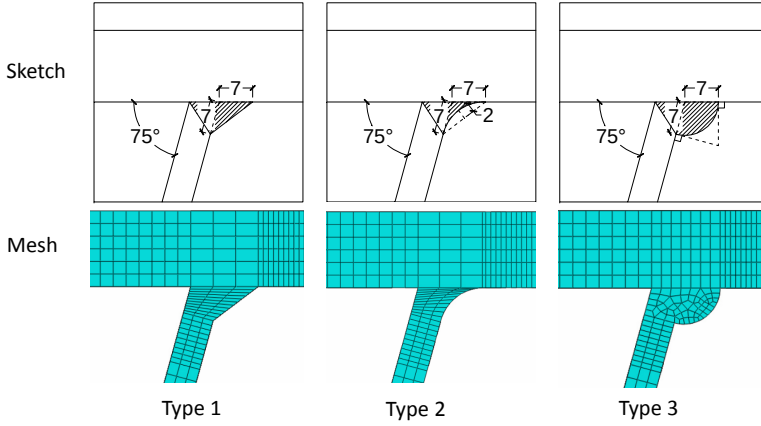


Figure 4.12: Sketch of the weld profiles used in the FE models (unit: mm).

TYPE 1 WELD PROFILE

For Type 1 weld profile, stress intensity factors along the crack front are calculated with different crack shapes (ratio $a/c = 0.067, 0.1, 0.2, 0.5$) and sizes (relative crack depth a/t_d ranging from 0.0067 to 0.73) based on the initial model (without crack) introduced in Section 4.3.1. Although $c/a = 2, 5, 10, 15$ are originally used when inserting cracks in the FE models, they are transferred to a/c in order to keep in line with the parameters in the guidelines [23, 24]. The crack depth increment Δa is set as 0.1 mm when $a \leq 1.0$ mm and increases to 1.0 mm when $a > 1.0$ mm.

Figure 4.13 shows ΔK_I at three points ($\phi = 0, \pi/4$, or $\pi/2$) calculated by the FE models under the load range of 1 kN. The point $\phi = \pi/4$ is intentionally shown for comparison. ΔK_I increases with the crack size. A linear increase trend is observed when $0.1 \leq a/t_d \leq 0.5$. At points $\phi = \pi/4$ and $\pi/2$, the values are rather close until $a/t_d = 0.6$, from where ΔK_I starts to decrease at $\phi = \pi/2$. At surface ($\phi = 0$), ΔK_I are smaller than the values at $\phi = \pi/4$ or $\pi/2$.

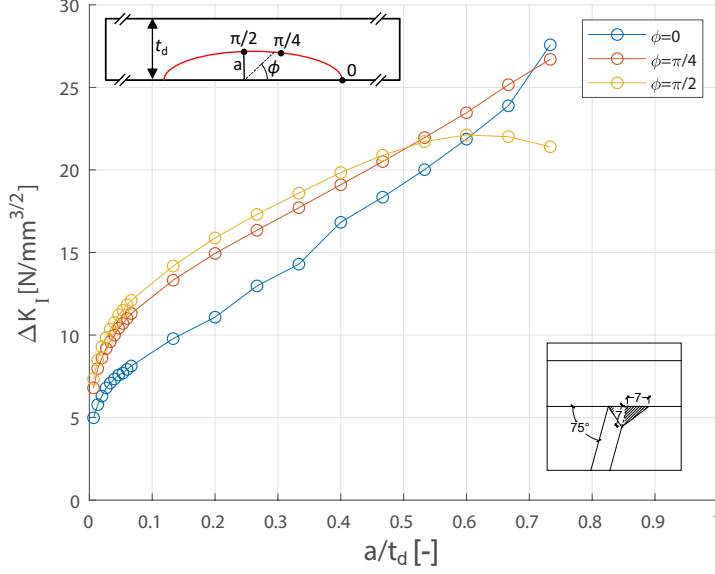


Figure 4.13: ΔK_I at $\phi = 0, \pi/4$, or $\pi/2$ under the load range of 1 kN with $a/c = 0.1$.

The geometric correction factor M_k is calculated by Equation (4.10), which is derived from Equation (4.9), as follows:

$$M_k = \frac{\Delta K_I}{(M_m(1 - \Omega) + M_b \Omega) \Delta \sigma_{hs} \sqrt{\frac{\pi a}{Q}}} \quad (4.10)$$

in which ΔK_I and $\Delta \sigma_{hs}$ are obtained from the FE calculations. M_m and M_b are calculated following the procedure in Appendix B. Ω is set to 1 as explained in Section 4.3.3.

The calculated results are plotted in Figure 4.14 and described by the fitted curves that are plotted in dash lines. Based on the calculated M_k , the hand calculation equations for the correction factor at the deepest point $M_{k,a}$ (at $\phi = \pi/2$) and the surface point $M_{k,c}$ (at $\phi = 0$) can be obtained.

Figure 4.15 shows two fitted curves for $M_{k,a}$: The dash curve in cyan colour for the $a/c = 0.1$ shape cracks and the dash curve in blue colour for all the considered cracks. A small difference between the two is observed when $a/t_d < 0.1$. The blue curve has slightly smaller values than the cyan curve when $a/t_d > 0.1$. For shallow cracks with $a/c = 0.0667$, a clear reduction of $M_{k,a}$ is found when $a/t_d > 0.6$. It is explained as the effect of stress redistribution caused by the change in structural integrity with the propagation of crack. The fitted curve in the case of $a/c = 0.1$ is used for all the considered cases in the calculation of $M_{k,a}$. It is suitable because the fatigue propagation period with small cracks dominates the total fatigue life.

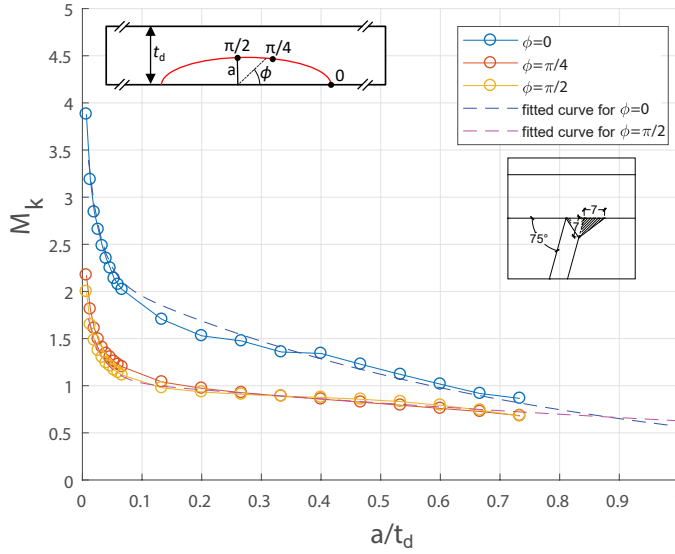


Figure 4.14: Geometric correction factors for Type 1 weld profile at $\phi = 0, \pi/4$, or $\pi/2$ with $a/c = 0.1$.

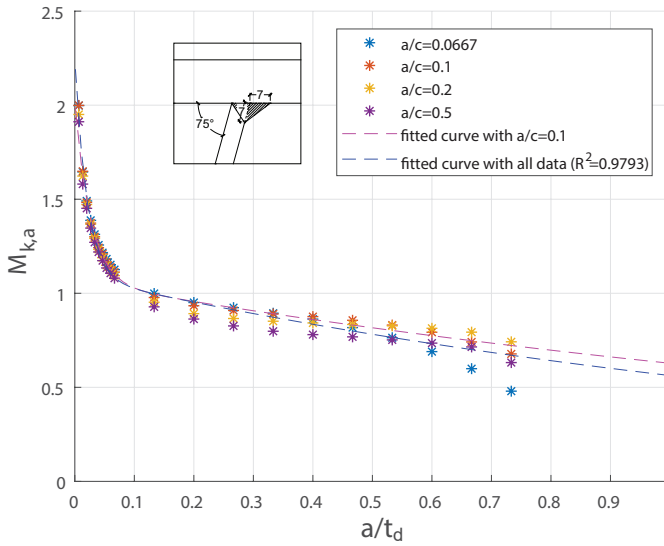


Figure 4.15: Geometric correction factor $M_{k,a}$ with various crack shapes.

Figure 4.16 shows the FE calculations and the fitted curves for $M_{k,c}$. For different crack shapes, differences can be observed even in small cracks. For instance, $a/c = 0.0667$ has higher $M_{k,c}$ when $a/t_d < 0.2$. The values reduce faster with the propagation of crack in the $a/c = 0.0667$ case compared with the $a/c = 0.1$ case. Moreover, the coefficient of determination is 0.7729, lower than the value 0.9793 in the fit of $M_{k,a}$. A crack shape correction factor, M_{sc} , is therefore added to the calculation of $M_{k,c}$ with detailed description given below.

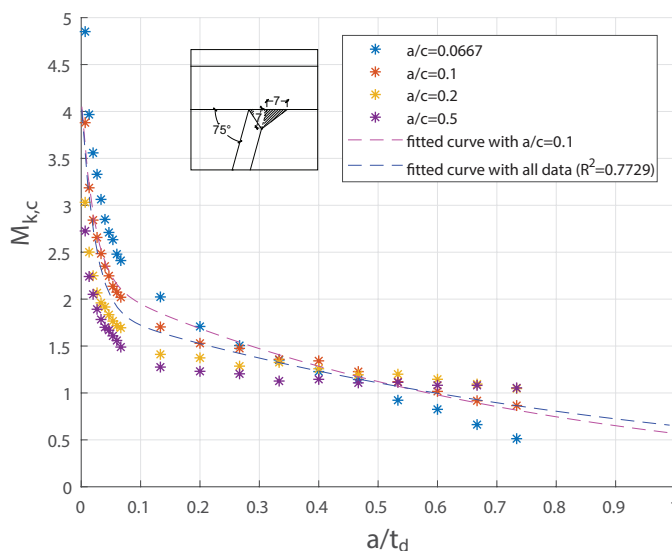


Figure 4.16: Geometric correction factor $M_{k,c}$ with various crack shapes.

By introducing the geometric correction factor M_k , the calculation procedure using the fitted curves is given as follows. The calculation can be divided into two steps:

Step 1. Calculate $M_{k,a}$ and $M_{k,c}$ with crack shape $a/c = 0.1$ using Equation (4.11). The correction factors are $M_{k,a}$ and $M_{k,c01}$ where subscript "01" means $a/c = 0.1$. For $M_{k,a}$, subscript "01" is not written because it is assumed that the $a/c = 0.1$ case is used for all the considered cases.

Step 2. Add M_{sc} to the calculation of $M_{k,c}$, see Equations (4.12) and (4.13).

Note that a maximum difference of ± 0.12 ($(M_{k,c, \text{fitted}} - M_{k,c, \text{FE}}) / M_{k,c, \text{FE}}$) is observed when comparing the analytically obtained results using the fitted curves with the values from the FE calculations as shown in Figure 4.17. See Table 4.2 for the numerical values of the constants p_1 to p_5 used in the calculation of $M_{k,a}$ and $M_{k,c}$.

$$\begin{aligned} M_{k,a} &= p_1 \cdot e^{p_2 \cdot a/t_d} + p_3 \cdot e^{p_4 \cdot a/t_d} \\ M_{k,c01} &= p_1 \cdot e^{p_2 \cdot a/t_d} + p_3 \cdot e^{p_4 \cdot a/t_d} \end{aligned} \quad (4.11)$$

$$M_{k,c} = M_{sc} M_{k,c01} \quad (4.12)$$

where:

$$M_{sc} = p_1 \cdot e^{\frac{-a/c}{p_2}} + p_3 \cdot e^{\frac{-a/t_d}{p_4}} + p_5 \cdot e^{\frac{-a/c}{p_2}} \cdot e^{\frac{-a/t_d}{p_4}} \quad (4.13)$$

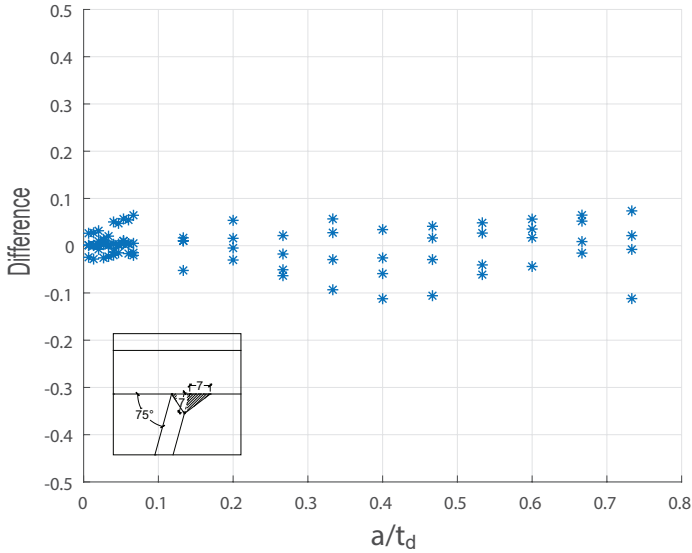


Figure 4.17: Difference between the analytically obtained results and the results from the FE calculations.

Table 4.2: Constants for fitting geometric correction factors M_k .

Fitted Constant	Type 1		Type 2		Type 3		Types 1, 2, 3
	$M_{k,a}$	$M_{k,c01}$	$M_{k,a}$	$M_{k,c01}$	$M_{k,a}$	$M_{k,c01}$	
p_1	0.96	1.93	-0.47	-0.90	1.73	4.256	7.43
p_2	-38.26	-46.63	1.07	1.32	-60.97	-61.91	0.05
p_3	1.06	2.21	-	-	1.116	2.425	0.71
p_4	-0.53	-1.36	-	-	-0.6482	-1.607	-1.36
p_5	-	-	-	-	-	-	-5.51
R-square	0.9936	0.9986	0.99	0.98	0.9919	0.995	0.9551
Equation	(4.11)	(4.11)	(4.14)	(4.14)	(4.11)	(4.11)	(4.13)

TYPE 2 WELD PROFILE

Type 2 weld profile represents the most favourable weld profile with smaller intersection angles between the base metal and the weld metal. Figure 4.18 shows M_k at the three points of Type 2 weld profile as a function of a/t_d . Among them, M_k at $\phi = \pi/2$ and $\phi = \pi/4$ are almost equal. Compared with Type 1 weld profile, $M_{k,a}$ (at $\phi = \pi/2$) are smaller with the values approximately equal to 1 when a/t_d is close to 0. It indicates that the local stress concentration at the weld toe of Type 2 weld profile is effectively reduced, which is beneficial for fatigue resistance. At $\phi = 0$, $M_{k,c}$ is close to 1.3. At all three points, M_k is reduced with the increase of crack. Linear curves are used to fit M_k . $M_{k,a}$ and $M_{k,c}$ can be calculated using Equations (4.13) and (4.14) with numerical values given in Table 4.2.

$$\begin{aligned} M_{k,a} &= p_1 \cdot a/t_d + p_2 \\ M_{k,c01} &= p_1 \cdot a/t_d + p_2 \end{aligned} \quad (4.14)$$

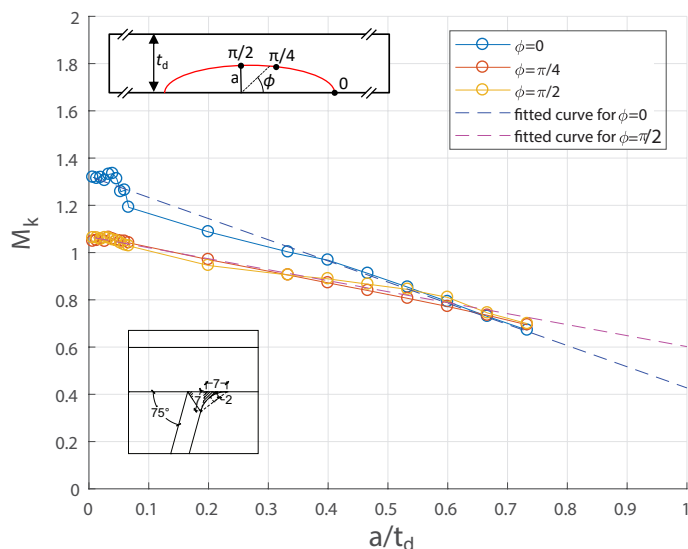


Figure 4.18: Geometric correction factors for Type 2 weld profile at $\phi = 0, \pi/4$, or $\pi/2$ with $a/c = 0.1$ for Type 2 weld profile.

TYPE 3 WELD PROFILE

Figure 4.19 shows M_k at the three points of Type 3 weld profile (the most unfavourable case). M_k shows a similar trend to Type 1 weld profile as large values are observed when cracks are small. Equations (4.11) and (4.13) are used to fit the curves and numerical values can be found in Table 4.2.

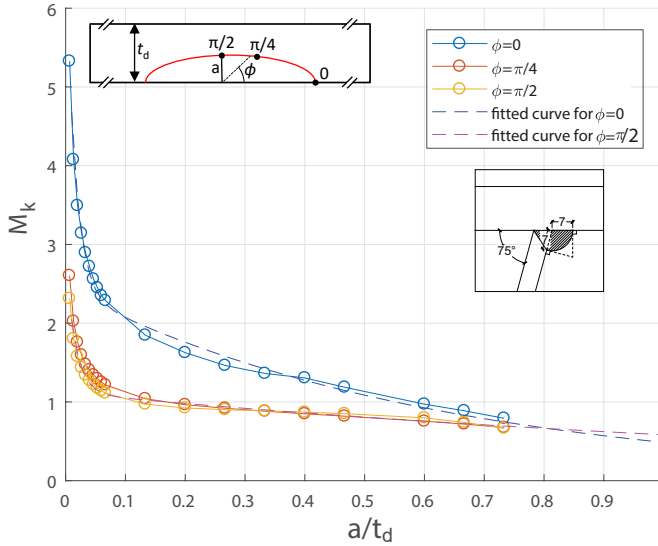


Figure 4.19: Geometric correction factors for Type 3 weld profile at $\phi = 0, \pi/4$, or $\pi/2$ with $a/c = 0.1$.

COMPARISON OF THE THREE WELD PROFILES

Figure 4.20 compares M_k for the three types of weld profiles. $M_{k,a}$ are very close for Types 1 and 3 weld profiles, whereas the difference is slightly larger for $M_{k,c}$. As already discussed, Type 2 profile has smaller values compared with Types 1 and 3 weld profiles.

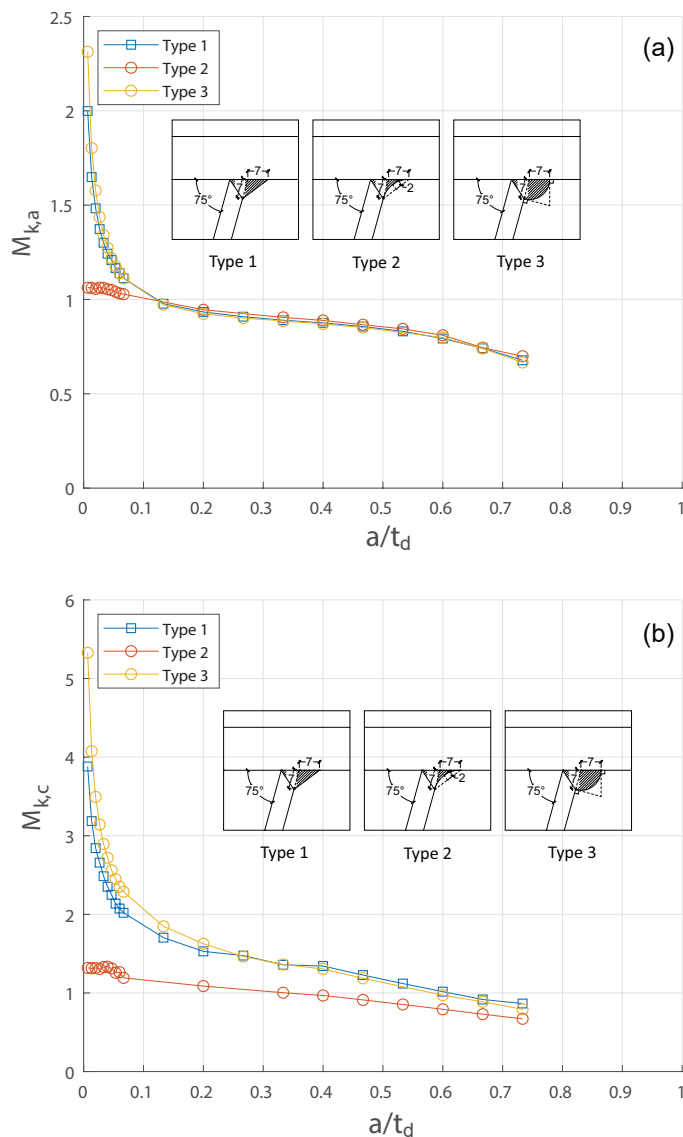


Figure 4.20: Comparison of M_k for the three types of weld profiles with $a/c = 0.1$: (a) $M_{k,a}$ (at $\phi = \pi/2$), and (b) $M_{k,c}$ (at $\phi = 0$).

VALIDATION AGAINST THE EXPERIMENT

The experiment S10 in Nagy's study [127] is again used to verify the results of the hand calculation with the geometric correction factors implemented. The following parameters are obtained from or based on [127]:

- The beach marked cracks are obtained from Figure B.9 and the corresponding number of cycles are found in Figure 7.2. The length of the beach mark cracks are extracted from Figure 8.13.
- An initial crack with $a_0 = 0.13$ mm (close to the value recommended by IIW [23]) and $c_0 = 137.20$ mm (close to the measured semi-length of the beach mark) is used for the crack propagation using the Paris' equation. Although the fitted initial crack depth $a_i = 0.0428$ mm in Table 8.6, the optical microscope investigation of the local fracture surface gives a value around 0.3 mm to 0.4 mm [127]. It is therefore challenging to select an initial crack depth from [127]. Alternatively, a widely accepted value is used for a_0 .
- The crack depth propagation parameters, $C_a = 2.47 \times 10^{-13}$ (units in millimetre and Newton) and $m = 3.00$ are used based on the best-fit P3 in Table 8.6. The crack length propagation parameters are, however, not found in [127]. The values $C_c = 5.21 \times 10^{-13}$ (units in millimetre and Newton) and $m = 3.00$ are therefore used following [126].

Note that the figures and tables in the above itemised list refer to reference [127] instead of this dissertation. Only the beach marks with both the crack depth and the crack length available in [127] are presented in the current study. The definitions of the parameters can be found in Section 4.4. Moreover, only one crack propagation is calculated in the current study. This simplification is believed to be suitable as small difference was observed in fatigue life estimation when comparing the predictions based on the multi-crack propagation with the results based on the single crack propagation assumption [195].

The hand calculation procedure for fatigue crack propagation is shown as:

- Step 1. Identify parameters: the hot spot stress range $\Delta\sigma_{hs}$, the bending stress proportion Ω , the increment at the deepest point Δa_i , the initial crack depth a_0 and semi-length c_0 , and the propagation parameters $C_{a(c)}$ and $m_{a(c)}$.
- Step 2. Assign variables: the counter i (initial value is 1), the crack depth a (initial value is a_0) and semi-length (initial value is c_0), and the number of cycles N (initial value is 0).
- Step 3. Calculate $\Delta K_{a(c)}$ using Equation (4.9) in which $M_{k,a(c)}$ can be calculated using Equations (4.11) to (4.14). $\Delta K_{I,a(c)}$ are used as type C1a crack is a Mode I crack.
- Step 4. Calculate the increment of load cycles ΔN_i using Equation (4.5).
- Step 5. Calculate the increment of semi-length Δc_i using derived Equation (4.5) with ΔN_i obtained in Step 4.

Step 6. Update the crack size with $a + \Delta a_i$ and $c + \Delta c_i$, the number of cycles with $N + \Delta N_i$, and the counter with $i + 1$.

Step 7. Go to Step 2 in case that all predefined criteria are not satisfied, *e.g.* $a < t_d$. Otherwise the calculation is terminated.

Figure 4.21 compares the predicted crack propagation by hand calculation with the experimentally obtained beach marks in S10.

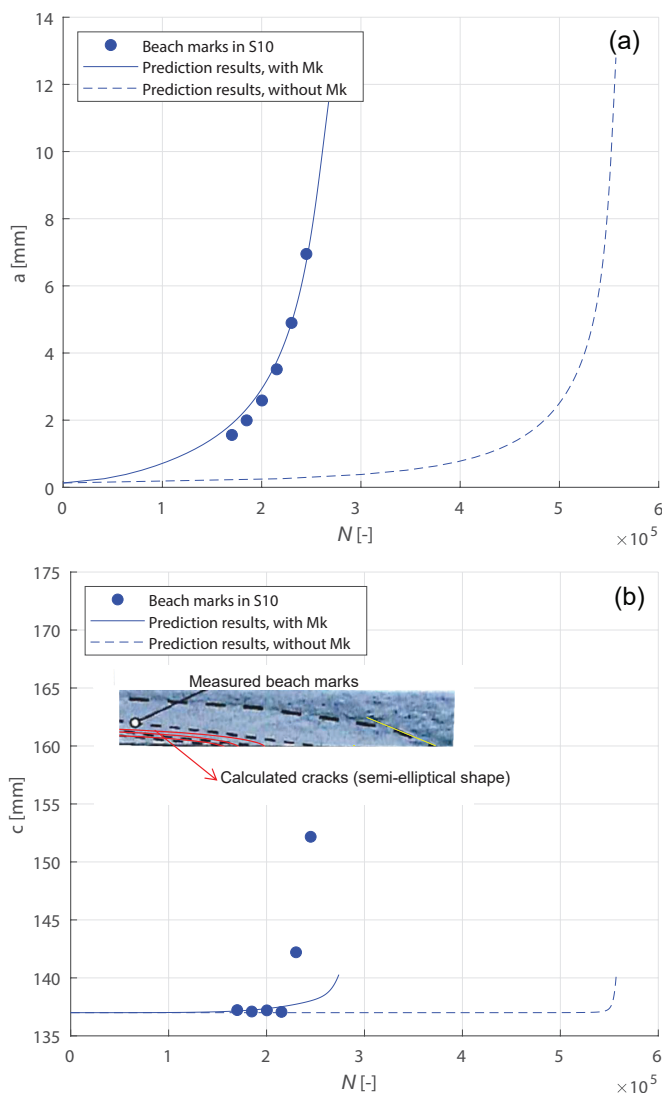


Figure 4.21: Comparison of the predicted crack propagation by hand calculation and the experimentally obtained beach marks in S10 [127].

The predictions agree well with the experimental measurements. The predicted crack depth, a , propagates exponentially with loading, whereas the predicted semi-length c grows slower than the experimental results. This is understandable if we recall that the end angles θ in Figure 4.9 are much smaller than the assumed $\theta = \pi/2$ which is the pre-defined value in the hand calculation model. A smaller θ results in a smaller $\sin(\theta)$ and therefore a larger Δc . In case that the geometric correction factor M_k is not applied in calculation (dash lines), the crack propagation becomes slower in both depth and length directions. The fatigue life is therefore overestimated. Note that the increase of crack depth a dominates the total fatigue life when the crack is so shallow. The accurate prediction of crack depth propagation has guaranteed the fatigue life prediction.

4.4. PROBABILISTIC FRACTURE MECHANICS ANALYSIS

4

FATIGUE LIFE CALCULATION

Probabilistic fracture mechanics is used to predict the fatigue resistance of detail C1a using the hand calculation in conjunction with Monte Carlo simulations. Fatigue life N_f can be divided into the crack initiation period N_i and the crack propagation period N_p , see Equation (4.15). A conservative way to calculate the fatigue life of welded connection is to assume that there are initial small cracks at the initiation locations. The fatigue life N_f is therefore equal to the propagation period N_p , which is used in the current study.

$$N_f = N_i + N_p \quad (4.15)$$

Fatigue crack propagation can be calculated using the linear propagation model as described by Equation (4.5) [22]. Hand calculation with the developed M_k is used to calculate $\Delta K_{a(c)}$. The linear propagation model has been verified in Section 4.3.4. Besides, the bilinear propagation model is provided in BS 7910 [24]. The probabilistic fracture mechanics analysis with the bilinear propagation model gives a similar prediction compared with the results based on the linear propagation model, see Appendix D. The linear propagation model is used in the following parts of this chapter.

PARAMETERS FOR PROBABILISTIC ANALYSIS

Table 4.3 lists the probabilistic density functions and the input variables used in Monte Carlo simulations. The explanations of the used numerical values are shown as:

Initial crack depth a_0 : Initial cracks in welded connections may be caused in manufacture or fabrication. The size of initial crack can range from grain scale to several millimetres. The lognormal density function with the mean value $a_0 = 0.15$ mm and the Coefficient of Variation (CoV) of 0.66 recommended by the Joint Committee on Structural Safety (JCSS) [64] are adopted in the current study.

Initial crack shape a_0/c_0 : Various initial crack shapes with a_0/c_0 ranging from 0 to 1 are given in the literature [23, 24, 126, 188, 196] for welded connections with the specific value depending on structural detail, crack depth, and applied stress. The lognormal density function with the mean value $a_0/c_0 = 0.62$ and the CoV of 0.40 recommended by JCSS [64] are adopted in the current study.

Paris' equation parameters C , m , and ΔK_{th} : Material parameters C and m are often analysed using the lognormal function and the deterministic density function following [196], respectively. The value of m is set as 3.00 following [64, 126, 188, 196]. Parameter C is affected by microstructure of material and applied stress. Stress ratio can affect the value of C . High residual stress induced by welding may exist in weld. However, it is hard to be assumed without knowing the details of welding procedure and the constraint conditions during welding. Under the applied load ratio $R = -1$, the mean values of C in the current study are $C_a = 1.00 \times 10^{-13}$ (units in millimetre and Newton) and $C_c = 3.00 \times 10^{-13}$ (units in millimetre and Newton) following [126]. Due to the high residual stress at the surface of weld toe, C_c is thus larger than C_a . The CoVs are set as 0.98 based on the calibration against the experimental results. The threshold value ΔK_{th} follows the lognormal density function with the mean value of $170 \text{ N/mm}^{3/2}$ and the CoV of 0.20 [126].

Hot spot stress deviation ratio B_{hs} : The hot spot stress range, $\Delta\sigma_{hs}$, is obtained by linear extrapolating from the predefined locations to the hot spot (weld toe here) to consider the geometric stress concentration. The deviation between the actual and the calculated $\Delta\sigma_{hs}$ can be considered by $B_{hs} \cdot \Delta\sigma_{hs}$ where the ratio B_{hs} follows the lognormal density function. The mean value and the CoV are set as 1.0 and 0.05 following [197], respectively.

Fracture toughness K_{mat} : The resistance of material for fracture is controlled by its fracture toughness K_{mat} which finally affects the fatigue life. In road bridges, the value is treated as the Weibull density function [198]. The mean value of $2250.00 \text{ N/mm}^{3/2}$ and the CoV of 0.25 recommended in [198] are used in the current study.

Table 4.3: Variables and probabilistic functions for Monte Carlo simulations using the linear propagation model (units: mm and N).

Variable	Density function	Mean value	CoV
a_0	Lognormal	0.15 [64]	0.66 [64]
a_0/c_0	Lognormal	0.62 [64]	0.40 [64]
C_a	Lognormal	1.00×10^{-13} [126]	0.98
C_c	Lognormal	3.00×10^{-13} [126]	0.98
m	Deterministic	3.00 [126]	-
ΔK_{th}	Lognormal	170.00 [126]	0.20 [126]
B_{hs}	Lognormal	1.00 [197]	0.05 [197]
K_{mat}	Weibull	2250.00 [198]	0.25 [198]

Monte Carlo Simulation

Statistical analysis is carried out using either the generated samples by Monte Carlo simulations or the experimentally obtained results as shown in Figure 4.22. For each deck plate thickness, approximately 10^5 Monte Carlo samples are generated using the parameters introduced in the previous section. The calculation is based on a random combination of the eight variables. Fatigue life is then calculated using fracture mechanics in conjunction with the linear propagation model. The samples with $10^5 \leq N \leq 5 \times 10^6$ and final $a/c \leq 1$ are grouped in the statistical analysis. These samples are plotted as grey circles in Figure 4.22. The predicted stress ranges at 2 million cycles are 147 MPa and 193 MPa for $P_s = 95\%$ and $P_s = 50\%$, respectively. The differences between the Monte Carlo simulations and the experiments are within 5%. The predicted value of 255 MPa for $P_s = 5\%$ is 7% lower than the experimentally obtained value of 273 MPa. Note that the uni-axial stress assumption is adopted to calculate $\Delta\sigma_{hs}$ by the measured strain ranges in the experiments [107, 110, 127, 166], whereas the hand calculation model is developed by the FE models using the 400 mm (in the longitudinal weld direction) deck plate following [127]. The stresses in the FE models are extracted in the middle of specimen for the $\Delta\sigma_{hs}$ calculation, which are close to the plane strain state. $\Delta\sigma_{hs}$ in the hand calculation model is therefore transferred from the plane strain assumption to the plane stress assumption by multiplying $1 - \nu^2$ where $\nu = 0.3$ in Figure 4.22 to keep in line with the experiments.

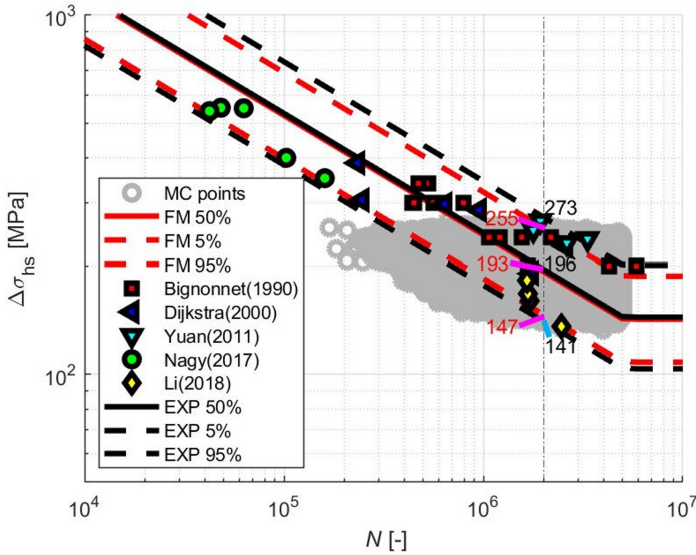


Figure 4.22: Statistical analysis for detail C1a based on the Monte Carlo generated samples and the experimental results, both with $t_d = 10, 12, 15, \text{ or } 19 \text{ mm}$ [107, 110, 121, 127, 166].

Figure 4.23 shows the predicted fatigue resistances with the deck plate thickness t_d ranging from 10 mm to 30 mm. Either plane strain or plane stress assumption is adopted in the analysis, whereas only the values with the plane stress assumption are further dis-

cussed. A reduction of the characteristic value at 2 million cycles with $P_s = 95\%$, $\Delta\sigma_{C,hs}$, is observed with the increase of t_d . The predicted $\Delta\sigma_{C,hs}$ are 141 MPa, 125 MPa, and 122 MPa for $t_d = 14$ mm, 26 mm and 30 mm, respectively. Detail category 140 MPa is therefore proposed for the deck plate with thicknesses ranging from 10 mm to 14 mm, whereas detail category 125 MPa is proposed for the deck plate thicker than 14 mm.

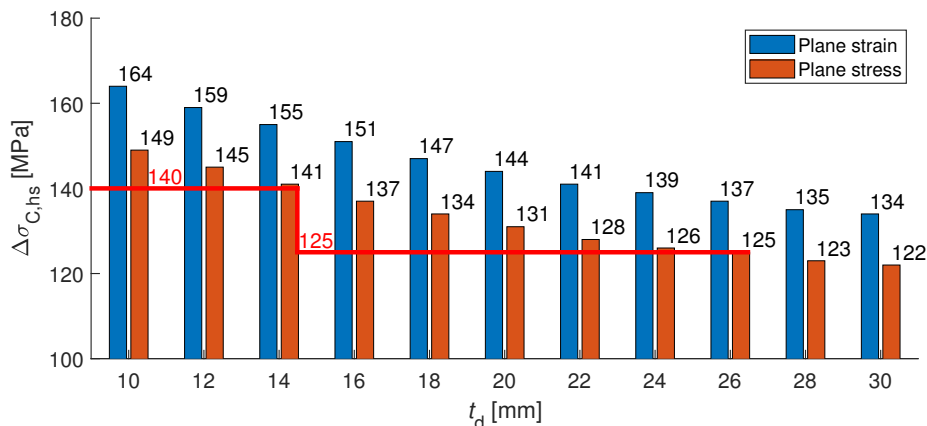
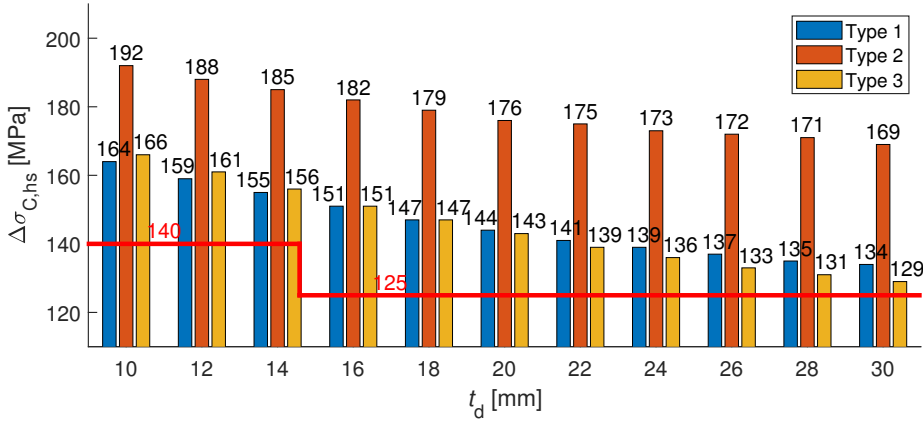


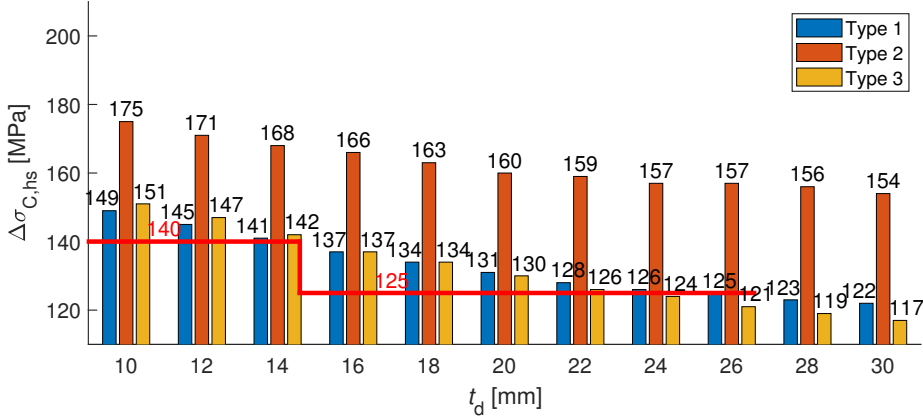
Figure 4.23: Fatigue resistances predicted by Monte Carlo simulations for t_d ranging from 10 mm to 30 mm.

EFFECT OF WELD PROFILE

Figure 4.24 compares the fatigue resistances of Type 1, Type 2, and Type 3 weld profiles predicted by Monte Carlo simulations. An approximately 20% to 25% increase of $\Delta\sigma_{C,hs}$ is observed when post-weld treatment is applied (Type 2) compared with the as-welded condition (Type 1). The difference between Type 1 and Type 3 weld profiles is within 1% until $t_d = 22$ mm. It further increases to 4% when $t_d = 30$ mm. It can be concluded that even the most unfavourable overhead welding situation (Type 3) may have limited effects on the fatigue resistance of detail C1a.



(a) plane strain



(b) plane stress

Figure 4.24: Comparison of the fatigue resistances of Type 1, Type 2, and Type 3 weld profiles predicted by Monte Carlo simulations.

4.5. SUMMARY OF THE CHAPTER

Fatigue resistance of the crack in stiffener-to-deck plate welded connection that initiates at the weld toe and propagates into the deck plate between crossbeams (designated as detail C1a in TS [27]) is studied using the experiments found in the literature and the fracture mechanics in this chapter. The geometric correction factors M_k , which are functions of the crack size and the plate thickness, are developed by the finite element method. M_k can be used in conjunction with the empirical equations (developed by Newman and Raju [192] for a semi-elliptical crack in a plate) for the semi-elliptical crack at the weld toe under bending (e.g. detail C1a) with three weld profiles. Fatigue resistances of detail C1a are accurately predicted by Monte Carlo simulations using the hand calculation (analytical) model with M_k implemented. The following conclusions and recommendations are drawn from the study of this chapter:

- A higher fatigue resistance for the hot spot stress compared with the standardised curve (with the characteristic value at 2 million cycles $\sigma_{C,hs} = 100$ MPa [16, 17, 23]) is obtained using the small-scale experiments in the literature, resulting in $\Delta\sigma_{C,hs} = 141$ MPa. This is mainly due to the high proportion of bending stress and negative load ratios.
- Multi-point extrapolation approach in conjunction with the Paris' equation (linear propagation model) can accurately predict the crack shapes and the corresponding load cycles when the first crack shape is accurately assigned based on the experimental measurement.
- Geometric correction factors M_k obtained by fitting the finite element calculations with numerical functions can be added to the empirical equations (developed by Newman and Raju [192]). Good agreement is found for the crack propagation in depth when comparing the calculated values with the experimentally obtained cracks in one specimen. For shallow cracks with shapes differ from the assumed semi-ellipse, the hand calculation using the semi-elliptical crack propagation model underestimates of the crack propagation in length when the intersection angles (between the crack front and the surface of plate) at edges are small.
- Monte Carlo simulations are successfully carried out using the M_k implemented hand calculation equations in conjunction with the linear crack propagation model. Excellent predictions of $\sigma_{C,hs}$ are obtained by the statistical analysis of the generated samples. The differences are within 5% compared with the experimentally obtained values. With the increase of deck plate thickness from 10 mm, to 14 mm, until 26 mm, the predicted $\Delta\sigma_{C,hs}$ is reduced from 149 MPa, to 141 MPa, until 125 MPa. Based on the validated probabilistic fracture mechanics calculations, the recommended characteristic fatigue resistances for detail C1a are:
 - **140 MPa** for deck plate thickness ≤ 14 mm,
 - **125 MPa** for deck plate thickness > 14 mm.
- Weld shape profiles affect the fatigue resistance of detail C1a, which was not considered by fracture mechanics before. A convex weld profile (Type 2) and a concave weld profile (Type 3) are analysed using the probabilistic fracture mechanics and compared with the standard case (Type 1) where the weld profile is assumed to be straight. Type 2 and Type 3 represent two extreme cases (*i.e.* the post-weld treatment and the overhead welding with limited operational space) that may occur in a welded connection, respectively. Type 2 weld profile gives higher $\sigma_{C,hs}$, up to 25%, than Type 1 weld profile. A 4% difference in $\sigma_{C,hs}$ is found between Type 1 and Type 3 weld profiles. From the fracture mechanics calculations in this study, the post-weld treatment is proven to be an effective method to improve the fatigue resistance, whereas the effects of overhead welding are limited.

5

WELD ROOT CRACK IN DECK PLATE OF STIFFENER-TO-DECK PLATE WELDED CONNECTION AT CROSSBEAM (C1c)

Fatigue behaviour of cracks in stiffener-to-deck plate welded connection between crossbeams has been discussed in Chapters 3 and 4. Another possible fatigue failure is the crack initiating from the weld root of stiffener-to-deck plate welded connection at crossbeam is studied in Chapter 5. The full-scale experimental investigations in this study use a 20 mm thick deck plate with a dimension of 9400 mm × 5100 mm, including three crossbeams, which represents modern designed OBDs. The experiments show an arrest of crack propagation with a final crack depth of approximately 75% of the deck plate thickness. On the contrary, through-thickness cracks develop in the 10 or 12 mm deck plates. Hot spot stress-based fatigue detail categories using various failure criteria are derived from the experiments. Analysis with the effective notch stress shows that the detail categories have low sensitivity to the amount of weld penetration. The results of the extended finite element method analyses, used to study the crack propagation path and crack arrest, are in line with the experiments of the current study and in the literature.

This chapter is based on the journal paper: *Fatigue behaviour of root crack in stiffener-to-deck plate weld at crossbeam of orthotropic bridge decks*, *Engineering Structures*, **306**, (2024).

5.1. INTRODUCTION

Stiffener-to-deck plate welded connection in OBDs is subjected to high stresses caused by wheel loading of crossing lorries. Fatigue cracks may initiate from the weld toe (C1a and C2a) or weld root (C1b, C1c, and C2b) of the connection and propagate into the deck plate (C1a, C1b, and C1c) or stiffener web (C2a) or weld throat (C2b). Crack types C2a and C2b, and type C1a have been discussed in Chapter 3, and Chapter 4, respectively.

This chapter studies a commonly seen crack in stiffener-to-deck plate welded connection: the root crack that propagates into the deck plate at the crossbeam intersection (designated as type C1c), as shown in Figure 5.1. It is known that root cracks normally have lower fatigue resistances than toe cracks [23]. Type C1c crack accounts for a large proportion of fatigue cracks inspected in existing OBDs [7, 11, 124]. Top surface breaking cracks (through-thickness cracks) were observed in both experiments carried out in the laboratory and on-site inspections [1, 14, 123, 124, 172]. The studies demonstrate that type C1c crack has a relatively long crack propagation period due to stress redundancy [123, 124], which is confirmed by fracture mechanics calculations [126, 199]. The nominal stress method is unsuitable for detail C1c because of local stress concentration. The hot spot stress consisting of the bending stress and the membrane stresses of the plate [23, 26, 200] is, alternatively, more suitable for this detail.

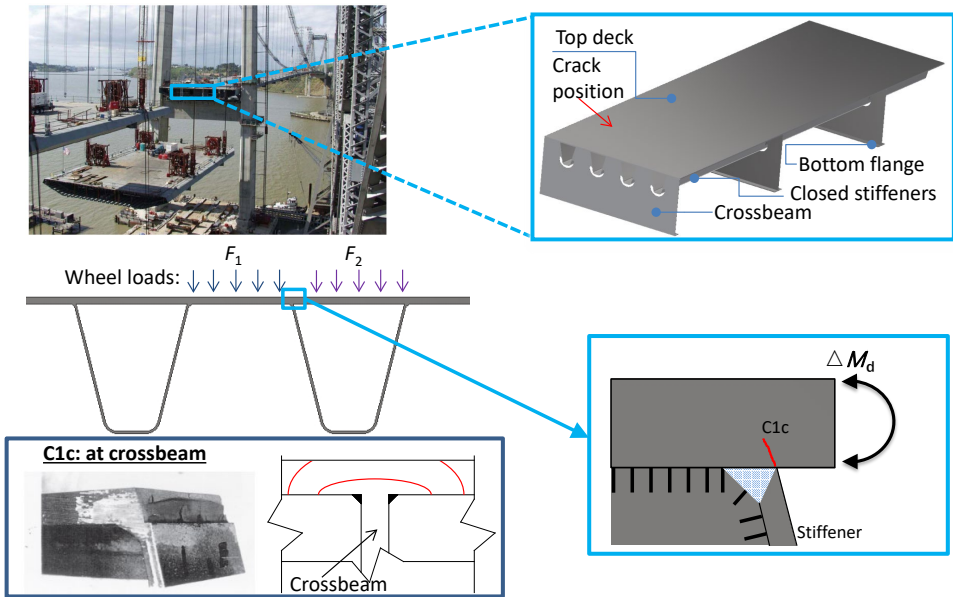


Figure 5.1: Illustration of type C1c crack in stiffener-to-deck plate welded connection at crossbeam (photo from [1]).

Detail C1c was included by neither in the first nor the second generation of (pr)EN 1993-1-9 [16, 17]. A Detail Category (DC) of 125 MPa for the hot spot stress was proposed by Kolstein [1] based on the conservative treatment of the experiments in [1, 14]. The US AASHTO LRFD Bridge Design Specifications [2, 19] gives Category C (equivalent to DC

90 MPa), the lowest DC for the hot spot stress, for detail C1c. Recent research shows that the fatigue resistance of detail C1c is higher than the recommended values, amongst others attributed to the generation of compressive stress under wheel loading at the crack initiation location.

With the increase in traffic volume, a need is identified to construct a new type of OBD with better fatigue performance. One easy and effective solution is to increase the deck plate thickness. Maljaars *et al.* [11] recommended a deck plate thickness ranging from 17 mm to 22 mm for “Cat. 1” type road bridges (with the heaviest traffic flow) with a target service life of 100 years based on a fracture mechanics assessment using the fatigue load measured with weigh in motion systems in the Netherlands. Modern OBDs commonly use a deck plate with the thickness ranging from 16 mm to 19 mm [2, 171, 172]. However, the recommended DC for detail C1c with these thick deck plates was not available.

The author, therefore, carries out fatigue experiments using a full-scale OBD specimen with a 20 mm thick deck plate. The specimen contains eight trapezoidal closed stiffeners that are loaded in fatigue individually. The cracks are inspected using the Time Of Flight Diffraction (TOFD), which is an ultrasonic non-destructive technique, and the naked eye inspection. Statistical analysis is carried out to obtain the fatigue life associated with the 10% or 25% change of the strain ranges measured on the top of the deck plate at the crossbeam intersection, or the first visible crack on the top of the deck plate. FE models are developed, and the effective notch stress method is used to study the effect of penetration ratio. The crack propagation path is analysed by fracture mechanics using the eXtended Finite Element Method (XFEM).

5.2. EXPERIMENTAL INVESTIGATION

5.2.1. SPECIMEN

GEOMETRIC CHARACTERISTIC

A full-scale 9400 mm long and 5100 mm wide OBD was constructed as the specimen for experiments. Figure 5.2 shows the geometric characteristics of the specimen. The specimen contains three crossbeams with a web thickness of 16 mm and a bottom flange of 200 mm wide and 12 mm thick. The 20 mm thick deck plate is supported by eight 6 mm thick trapezoidal stiffeners (sixteen C1c type welded details per crossbeam). The stiffeners are continuous at the crossbeams. The crossbeams are welded around at four of the stiffeners, whereas the others four have additional Haibach shape extended cut-outs in the crossbeams [1, 27]. Note that the cut-outs are not expected to influence the stress state of the detail C1c under consideration. The deck plate is composed of two steel plates connected with a butt weld, which is assumed not to affect the performance of detail C1c. The loaded details are designated as $S_x \underline{S}$ or $S_x \underline{N}$ for the South or North sides of the stiffeners, respectively (see designations $\underline{S}$ and $\underline{N}$ in Figure 5.2), where x denotes the stiffener number.

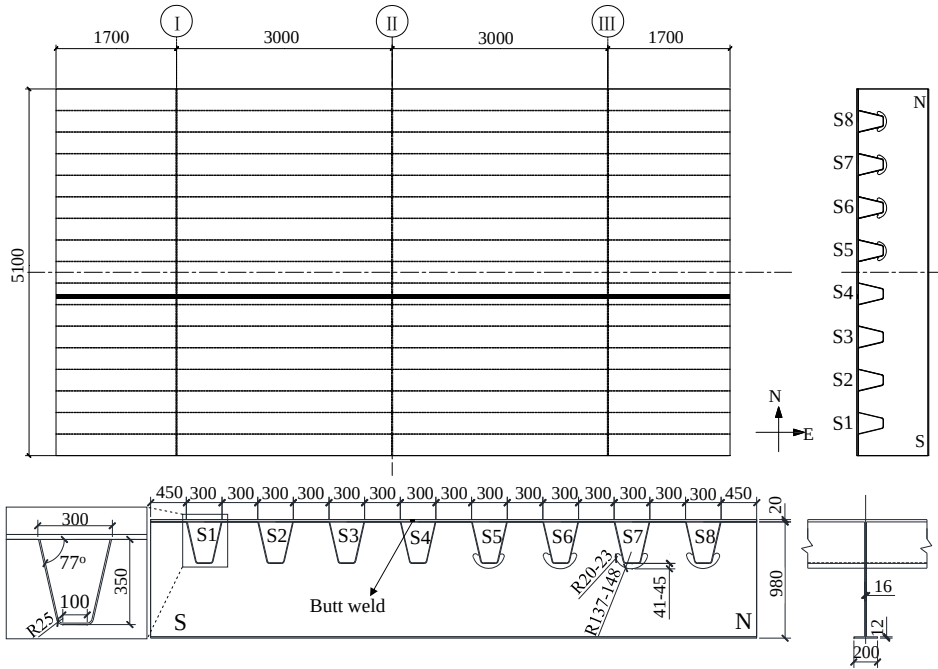


Figure 5.2: Overview of the full-scale specimen (unit: mm).

WELDING PROCEDURE

The specimen is made from the structural steel grade S355J2 with a nominal yield stress of 355 MPa [170]. It is known that the fatigue performance of welded connections may be affected by the welding procedure [201, 202].

Two-pass automatic metal arc welding is used to weld S1, S3, S5, and S7 to the deck plate (denoted as Weld 1, in short W1), and single-pass manual metal arc welding is used for the other stiffeners (denoted as Weld 2, in short W2) to study the effect of welding procedure on the fatigue performance of detail C1c, see Table 5.1 for the applied welding parameters and Figure 3.6 for the illustrations of the welding processes. Figure 5.3 illustrates the two welding procedures. The stiffener end is bevelled to a flat edge prior to welding with a minimum thickness of 3 mm for W1. A lack of penetration of a maximum of 1.5 mm is specified for both welding procedures following the requirements of TS [27]. Non-destructive testing is conducted and it is confirmed that the weld quality meets the acceptance criteria.

Figure 5.4 presents the resulting weld profiles at the representative cross sections. The geometry of weld toe transition is an important factor affecting the fatigue resistance of weld toe cracks [168, 175]. The flank angles of the automatic welding (θ'_A and θ''_A) are typically smaller than those of the manually welded ones (θ'_M and θ''_M). The weld surface of the automatic welding procedure appears smoother than the manual welding procedure.

Table 5.1: Welding parameters used in the fabrication of the stiffener-to-deck plate connections.

Type	W1	W1	W2
Pass	1	2	1
Process	FCAW	SAW	SMAW
Wire diameter [mm]	1.2	3	4
Current [A]	230-240	520-530	160-170
Voltage [V]	25-27	25-57	23-24
Current type	DC+	DC+	DC+
Welding speed [cm/min]	42-48	45-50	8-10
Heat input [kJ/mm]	0.7-1.0	1.6-1.9	2.3-2.8

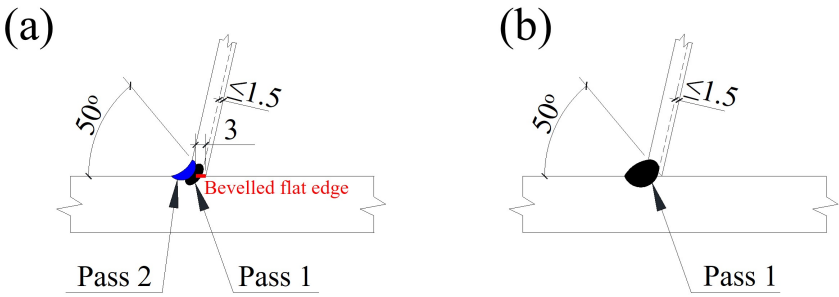


Figure 5.3: Welding procedure of: (a) W1; (b) W2 (unit: mm).

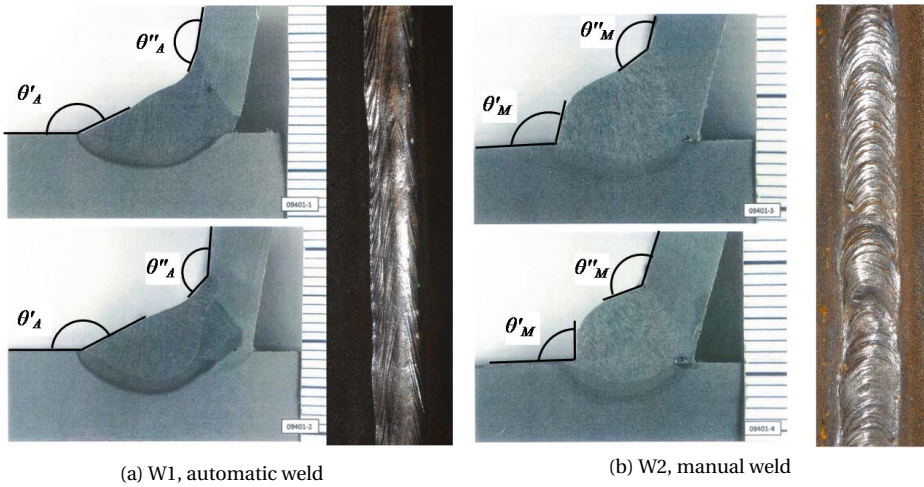


Figure 5.4: Representative cross sections of the stiffener-to-deck plate connections.

5.2.2. INSTRUMENTATION

Electric resistance strain gauges are used to obtain the strain distribution in the direction perpendicular to the weld line of the deck plate near the detail of interest (detail C1c). Three sets of strain gauges are symmetrically applied at both sides of the closed stiffeners, as shown in Figure 5.5. Set A (magenta) is applied at the top of the deck plate above every stiffener web. Set B (red) is applied at the top of the deck plate at S2 and S7. Set C (blue) is applied close to the weld root at the bottom of the deck plate at S6 and S7. Positions of strain gauges are selected sufficiently away from the weld to be unaffected by the local weld geometry and to measure global stress concentrations.

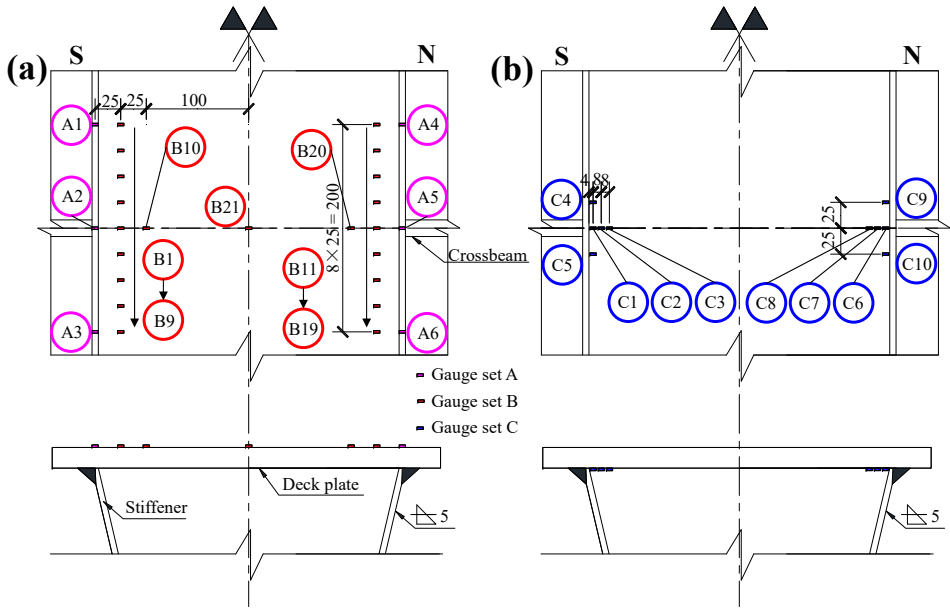
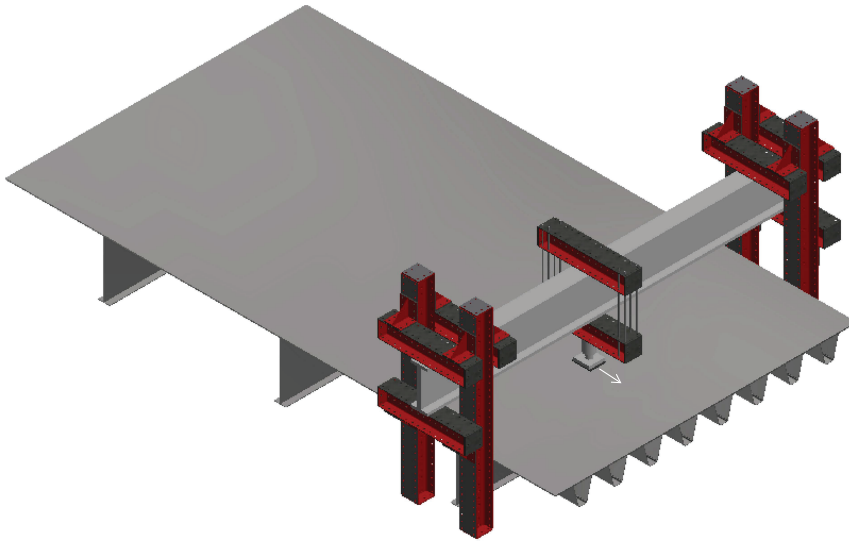


Figure 5.5: The arrangement of strain gauges (unit: mm), (a) top of the deck plate, (b) bottom of the deck plate.

5.2.3. FATIGUE LOADING PROGRAMME

Figure 5.6 shows a three-dimensional model of the setup and specimen. The bottom flanges of the crossbeams are continuously supported on the concrete floor (not presented). Fatigue load is applied through a hydraulic actuator that presses onto a 32 mm thick steel plate and three plates of rubber, 8 mm thick each, see Figure 5.7. The upper two rubber plates have a surface area of $270 \times 320 \text{ mm}^2$. The lowest plate has a surface area of $180 \times 320 \text{ mm}^2$, narrower than the other rubber plates, to allow the attachment of strain gauges. The load is applied centred over one stiffener and one of the outer crossbeams (Crossbeam III in Figure 5.2). The load is shifted in each experiment to the adjacent stiffener. The strains in the deck plate at adjacent stiffeners are negligible, checked by the strain gauge measurements. The load ratio, defined as the minimum load divided by the maximum load, is approximately 0.1. Each experiment contains two loading phases. The load is applied centred over Crossbeam III in Phase 1, with four dif-

ferent load ranges: 144 kN, 170 kN, 200 kN, or 240 kN to enable the statistical analysis of fatigue resistance, Table 5.2. During Phase 1, no surface crack is detected on the deck plate from the top. In Phase 2, the loading area is shifted 100 mm to the east direction. The load ranges in Phase 2 are nearly the same as in Phase 1. This load shift is considered relevant because it mimics the moving traffic loads that bridges are subjected to (axles crossing a bridge in the length direction). Section 5.3.1 provides the criterion to end Phase 1 and begin Phase 2.



(a) global view, with the arrow showing the load shift direction in Phase 2

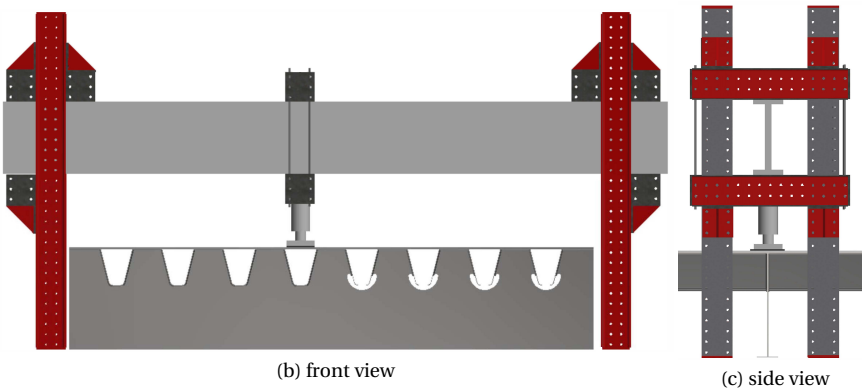


Figure 5.6: Sketch of the setup frame and actuator.

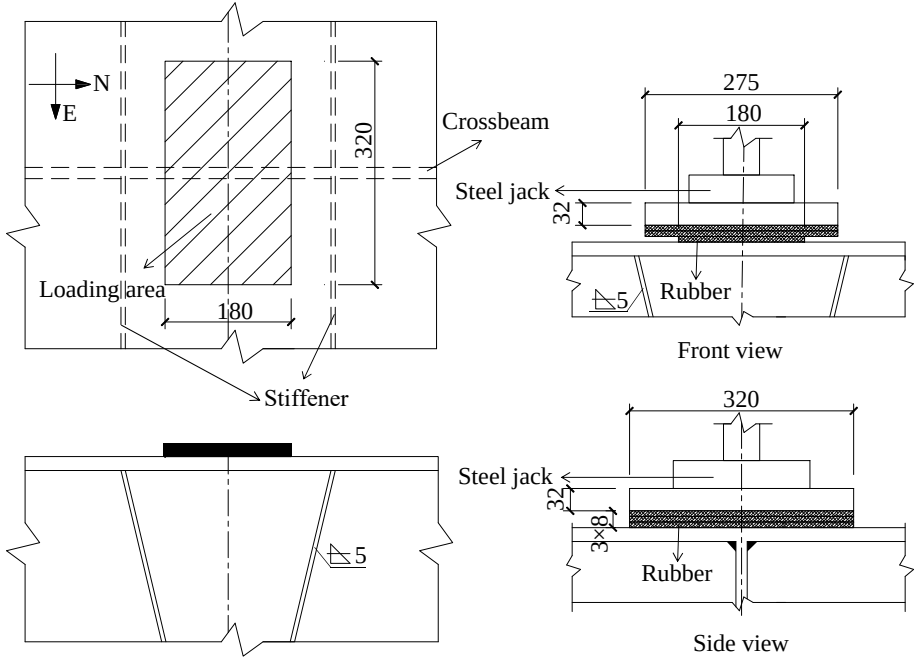


Figure 5.7: Illustration of the load position and the loading area in Phase 1 (unit: mm).

Table 5.2: Summary of the fatigue loading programme.

Stiffener		1	2	3	4	5	6	7	8
Phase 1	Maximum load [kN]	172	161	220	260	261	191	160	223
	Minimum load [kN]	2	16	20	24	24	27	16	23
	Load range [kN]	170	145	200	237	237	165	144	200
	Load cycles [million]	5.1	12.5	5.1	1.2	5.0	5.3	19.1	5.6
Phase 2	Maximum load [kN]	190	161	220	264	264	196	162	220
	Minimum load [kN]	12	15	20	24	24	29	16	20
	Load range [kN]	178	146	200	240	240	168	146	200
	Load cycles [million]	2.0	3.2	2.0	3.2	2.0	2.0	2.2	2.4

5.3. EXPERIMENTAL RESULTS

5.3.1. FATIGUE CRACK CHARACTERISTICS

Visual inspections by the naked eye are carried out at the end of Phase 1 and Phase 2 on the top of the deck plate for the top surface breaking crack detection. Cracks are not found on the top of the deck plate at the end of Phase 1. One top surface breaking crack is detected on the south side of S4 at the end of Phase 2 as shown in Figure 5.8. A 136 mm long arc-shaped crack is shown on S4 S. The blue dash lines represent centrelines of the crossbeam and the stiffener webs. Note that only a portion of the loading area in Phase 2 is displayed in Figure 5.8.

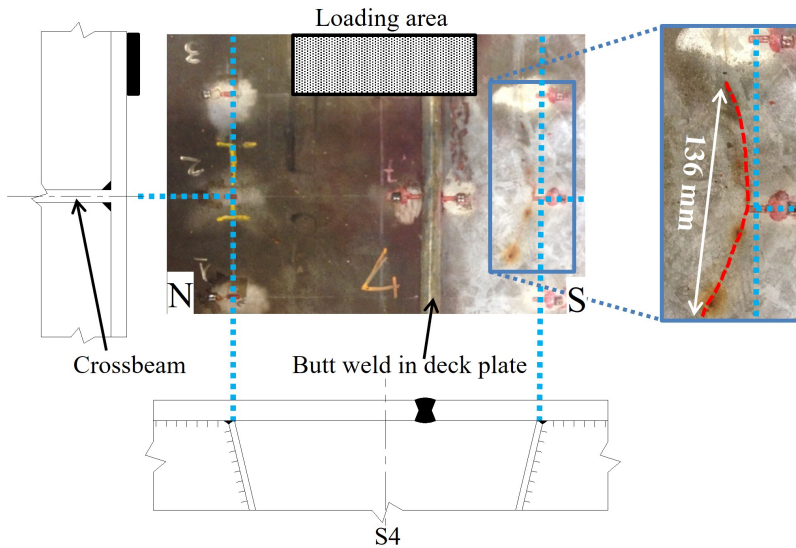


Figure 5.8: The visible cracks observed from the top of S4 at the end of Phase 2.

Figure 5.9a shows the inspected regions of the TOFD measurements. TOFD measurements are carried out within an approximate distance of 450 mm on either side of Crossbeam III using the dB-UT system [203]. Figure 5.10 shows an illustration of a TOFD measurement. The ultrasonic signal is sent and received from the top of the deck plate. Diffracted and reflected signals occur when the ultrasonic signals meet a crack. The signal receiver receives the diffracted signals. Details about the TOFD procedure are given, e.g. in [204]. The scan device moves stably from one side of the crossbeam to the other, with the location automatically stored in the system. The blue lines and the red lines (separated manually to improve visualisation) represent detected cracking areas at the end of Phases 1 and 2, respectively. Figure 5.9b gives a typical example of the signals in the presence of a crack. The centreline of the crossbeam is defined as the distance of 0, and negative numbers indicate the west direction.

Table 5.3 summarises the detected cracks from the TOFD measurements. Phase 1 is terminated (and Phase 2 starts) after reaching 5 million cycles of loading in most exper-

iments. The TOFD measurements reveal that cracks develop in an early stage in Phase 1 in most experiments, whereas these cracks hardly grow (or are arrested) during the remaining million cycles of Phase 1. Phase 1 is terminated after a much larger number of cycles in S2 and S7 to determine whether the cracks will pick up growth in the later stage. Phase 1 is terminated earlier in S4 to determine if this will affect the growth in Phase 2. The largest cracks arrest at the depth of approximately 14 mm to 15 mm (75% of the deck plate thickness) at the end of Phase 1. The start and end locations reveal that the crack may not propagate symmetrically at two sides of the crossbeam, which is not surprising given the random nature of fatigue process and the weld imperfection distribution. Besides the main crack, secondary cracks are detected at various details. The cracks grow in the length direction in Phase 2 (especially for S4 S-01, S6 N-02, and S7 S-01) but, with two exceptions, they hardly grow in the depth direction and none of the cracks grow through the thickness. Note that the start and end locations of the cracks may not be accurately located when the cracks are small, *e.g.* S3 S-01. A thorough visual inspection of the entire specimen is conducted at the end of Phase 2.

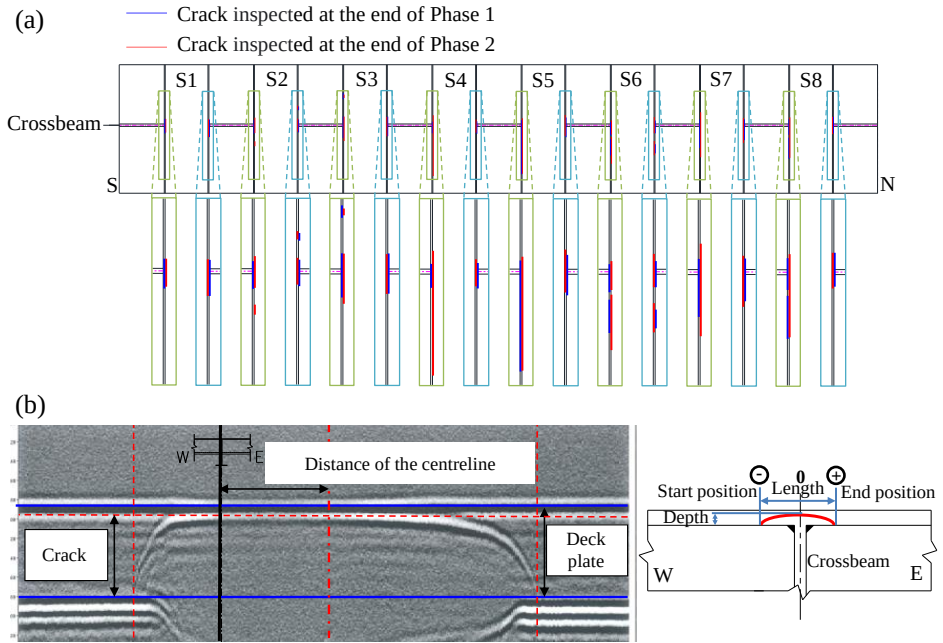


Figure 5.9: TOFD measurements for detail C1c, (a) inspection region, (b) an example at S7 S (ultrasonic signal waves on the left and illustration on the right) [203].

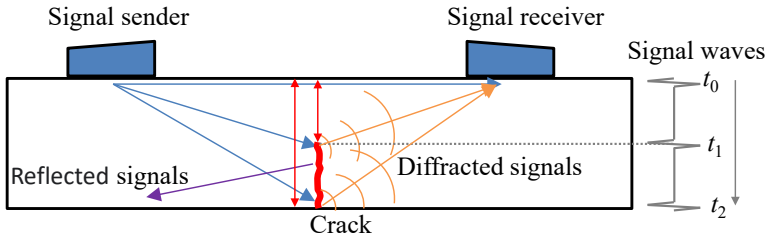


Figure 5.10: Illustration of ultrasonic signals in a TOFD measurement.

Table 5.3: Fatigue cracks locations of detail C1c at the end of Phases 1 and 2 as obtained by the TOFD measurements (unit: mm).

ID End of Phase	Start location		End location		Length		Depth	
	1	2	1	2	1	2	1	2
S1 N-01	-38	-40	78	78	116	118	14.5	15.1
S1 S-01	-42	-42	54	48	96	90	14.1	14.9
S2 N-01	-124	-130	-99	-103	25	27	2.1	2.2
S2 N-02	-38	-44	47	42	85	86	14.9	15.5
S2 S-01	-266	-260	-266	-260	0	0	2.4	2.3
S2 S-02	-34	-50	55	52	89	102	14.6	15.1
S2 S-03	-	106	-	138	-	32	-	4.3
S3 N-01	-57	-55	72	73	129	128	14.5	15.3
S3 S-01	-212	-202	-172	-180	40	22	6.2	6.4
S3 S-02	-58	-57	84	104	142	161	14.6	15
S4 N-01	-26	-38	56	49	82	87	14.7	14.8
S4 S-01	-24	-66	53	335	77	401	13.7	20
S5 N-01	-51	-68	79	70	130	138	13.7	14.5
S5 S-01	-34	-46	323	320	357	366	19.5	19.5
S6 N-01	-31	-54	53	53	84	107	14	14.7
S6 N-02	125	104	185	198	60	94	3.9	11.3
S6 S-01	-22	-32	68	61	90	93	13.6	14.4
S6 S-02	92	76	199	254	107	178	13.8	14.4
S7 N-01	-39	-48	98	116	137	164	14.3	14.6
S7 S-01	-37	-86	149	211	186	297	14.3	15.8
S8 N-01	-56	-56	53	59	109	115	13.7	13.9
S8 S-01	-39	-52	63	58	102	110	13.8	13.8
S8 S-02	82	58	220	215	138	157	13.8	14.3

The appearance of a crack can be shown by a change in recorded strain ranges during the experiments. An evaluation criterion often applied in the fatigue experiments of OBD details is a certain change of strain ranges as a function of applied number of load cycles.

Figure 5.11 shows the strain ranges of the strain gauges at the top and the bottom of the deck plate around the crossbeam, as a function of the logarithm of the number of cycles at S6 and S7 with the markers indicating the 10% (first symbol) and 25% (second symbol) changes of each curve. The strain ranges are approximately constant up to 0.1 million cycles, indicating that there is no significant settlement effect. The drop in strain range, in some cases preceded by an increase in strain range, is attributed to the development of crack(s). An immediate drop in reading may indicate a crack very close to the strain gauge, whereas an initial increase may indicate a crack initiating at a distance from the strain gauge. Figure 5.12 shows the strain ranges at the top of the deck plate at S2, at and within a distance of 100 mm from the centreline of the crossbeam. The change in strain range takes place over a large number of cycles. It indicates that a fast initiation followed by a relatively slow propagation in line with the observations in thinner deck plates [1, 14, 123, 124]. This finding based on the strain ranges agrees well with the TOFD measurements [205] and fracture mechanics calculations [126], where a slow propagation is also observed. Note that three other types of cracks are detected in addition to C1c, see Appendix C.1 for details.

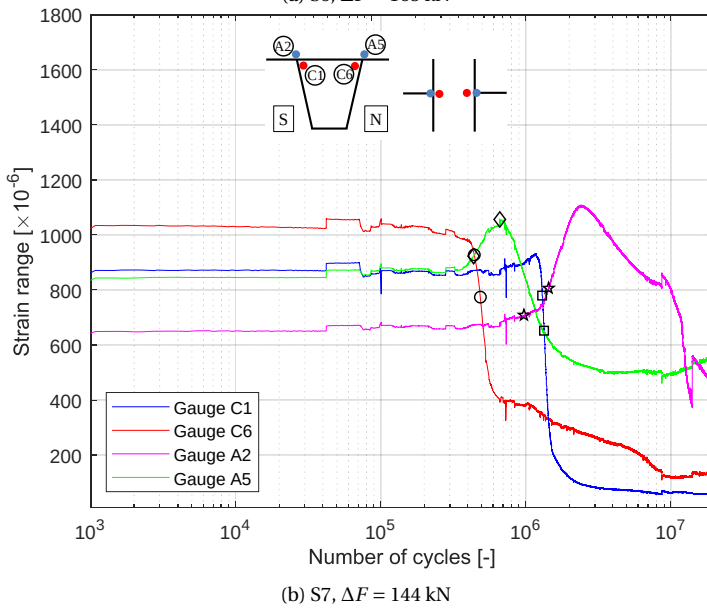
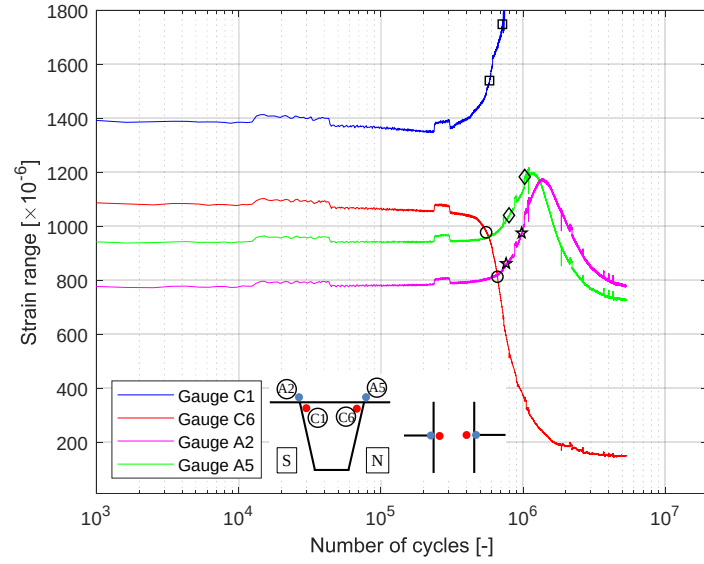


Figure 5.11: Change of the selected strain ranges as a function of load cycles, measured by the strain gauges at S6 and S7.

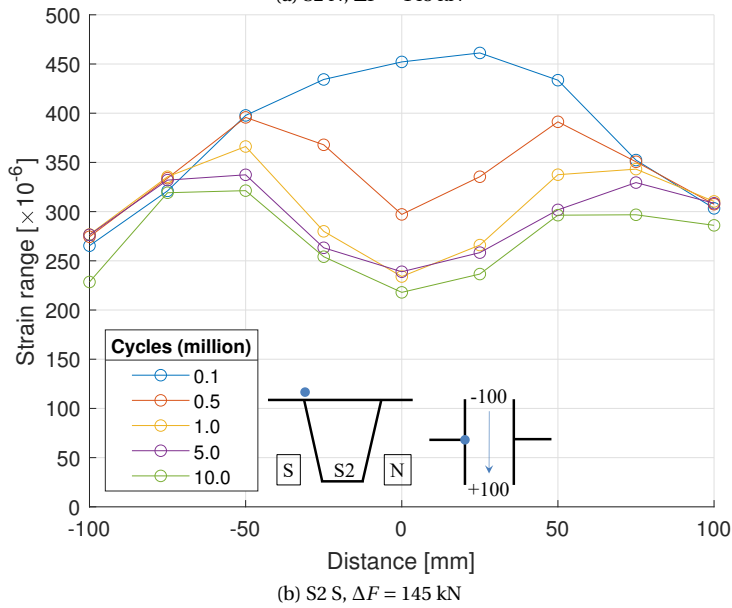
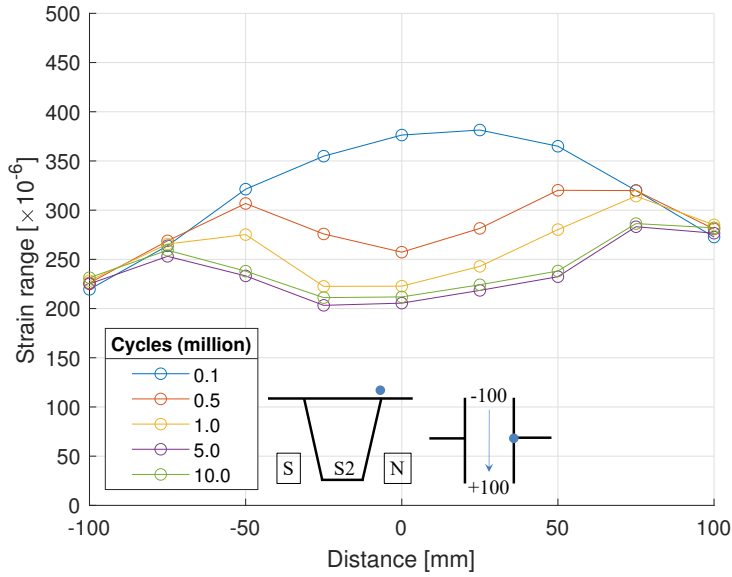


Figure 5.12: Distribution of strain ranges at the top of the deck plate at S2 for selected cycle counts.

5.3.2. (STRUCTURAL) HOT SPOT STRESS AT THE INITIAL LOADING STAGE

High stress concentrations appear in many details of OBDs, including detail C1c. Due to large stress gradients, the nominal stress is not simply defined for detail C1c. The hot spot stress is an attractive alternative as it contains both bending stress and membrane stress at the location of interest. De Jong [14] proposed a simplified hot spot stress cal-

calculation model for detail C1c. It considers a two-dimensional representation of the deck plate, which is modelled as a beam fully clamped at the intersections with the crossbeam, and it calculates the stress at the bottom of the beam, see Figure 5.13. The “wheel load” with a contact area of $L_q \times W_q$ is modelled with a uniformly distributed load q . L_d is the distance between the stiffener webs (*i.e.* the unsupported span of the deck plate). A slightly modified version of this hand calculation model is used in the Dutch National Annex to NEN-EN 1993-2+C1/NB: 2011 [206]. It is assumed that the load acts at the deck plate centre with a spread angle of 45° . Thus, q can be determined by Equation (5.1) [14] where ΔF and t_d are the wheel load and the thickness of deck plate, respectively. The hot spot stress range $\Delta\sigma_{hs}$ is determined as the nominal stress from the simplified beam model (shown in Figure 5.13), $\Delta\sigma_{nom}$, multiplied by a Stress Concentration Factor (SCF), see Equations (5.2) to (5.4) [14, 206]. $\Delta\sigma_{nom}$ is calculated using the beam theory. The SCF is based on a curve fit of parametric three-dimensional analyses using the Finite Element (FE) method, accounting for the difference in the effect of crossbeam between the three-dimensional reality and the two-dimensional beam model. The following dimensions are applied to the full-scale specimen presented in the current study: $t_d = 20$ mm, $L_q = 180$ mm, $W_q = 320$ mm, and $L_d = 300$ mm. This results in $\Delta\sigma_{hs} = 156.42$ MPa for the load range of 100 kN. Note that the model may not be accurate in special cases with significant shear deformation of crossbeam, which is different from the current setup with continuously supported crossbeams.

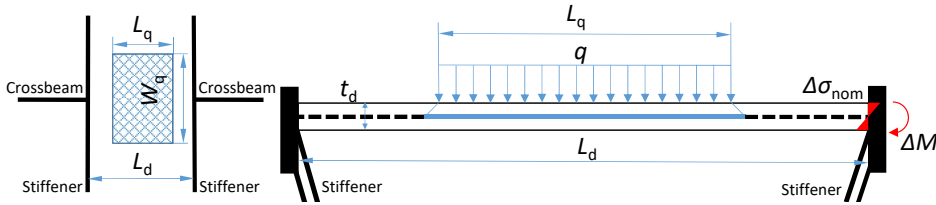


Figure 5.13: Illustration of the hand calculation model for $\Delta\sigma_{hs}$ in NEN-EN 1993-2+C1/NB: 2011 [206].

$$q = \frac{\Delta F}{(W_q + t_d) \times (L_q + t_d)} \quad (5.1)$$

$$\Delta\sigma_{nom} = \frac{\Delta M \times 0.5 t_d}{I} \quad (5.2)$$

where ΔM is the bending moment range and the second moment of area $I = t_d^3/12$, both in unit width.

$$\Delta\sigma_{hs} = SCF \cdot \Delta\sigma_{nom} \quad (5.3)$$

$$SCF = \begin{cases} 1.4 & \text{for pavement} \geq 50 \text{ mm,} \\ 1.2957 - 0.00938 t_d & \text{for pavement} < 50 \text{ mm.} \end{cases} \quad (5.4)$$

5.3.3. EVALUATION OF THE EXPERIMENTAL RESULTS

FATIGUE LIFE EVALUATION CRITERIA

Various fatigue life evaluation criteria were proposed in the literature for detail C1c. Some authors used the number of cycles at a certain change of the strain range at the top of deck plate as the evaluation criterion: 10% (Kolstein [1]), 15% (Dung [98]), 20% (Konda [123]), and 25% (Kolstein [1]). Among them, the 10% and 25% failure criteria are used in the current study denoted as N_1 and N_2 , respectively. The changes of the strain ranges measured by the strain gauges A2 and A5 are used to detect N_1 and N_2 , see Figure 5.5 for the strain gauge positions and Figure 5.11 for the detections of S6 and S7. Another failure criterion is the number of cycles at the earliest detection (with the naked eye) of a through-thickness crack at the top of deck plate, denoted here as N_3 [1, 14, 205]. This failure criterion is widely adopted for OBDs because of the relatively slow crack propagation and the ability to still carry the load with a limited surface breaking length. N_3 is more practical for on-site inspections.

CURRENT EXPERIMENTS WITH THE 20 MM THICK DECK PLATE

Table 5.4 presents the experimental results of Phase 1. The number of cycles corresponding to the failure criterion N_1 or N_2 , and the total load cycles N_t , are listed. The ratio N_2/N_1 evaluated from the strain gauges at the top of the deck plate ranges between 1.3 and 2.9, except for S5 S. A slow crack propagation is observed as N_t/N_1 varies from 3.2 to 59.1. The wide range in numbers may be caused by the crack sizes (shown in Table 5.3) at the end of loading. S5 S (marked) is excluded from the statistical analysis. This is because the strain range change was first observed at the location 100 mm away from the centreline of the crossbeam. The crack may not initiate at the crossbeam location for S5 S.

Table 5.4: Experimental results of Phase 1.

ID	ΔF [kN]	$\Delta\sigma_{hs}$ [MPa]	N_1	N_2	N_t	N_2/N_1	N_t/N_1
S1 N	170	252	203000	489797	5102664	2.5	25.1
S1 S	170	252	438837	729878	5102664	1.7	11.6
S3 N	200	296	127137	176034	5088787	1.4	40.0
S3 S	200	296	236629	339841	5088787	1.4	21.5
S5 N	237	351	158904	233896	5000001	1.5	31.5
S5 S*	237	351	1295773	5000001	5000001	3.9	3.9
S7 N	144	213	438119	663105	19122751	1.5	43.7
S7 S	144	213	974245	1440261	19122751	1.5	19.6
S2 N	145	215	211877	337779	12514488	1.6	59.1
S2 S	145	215	235858	686318	12514488	2.9	53.1
S4 N	237	351	382160	612537	1225001	1.6	3.2
S4 S	237	351	142185	341964	1225001	2.4	8.6
S6 N	165	244	797706	1029209	5287737	1.3	6.6
S6S	165	244	757172	978800	5287737	1.3	7.0
S8 N	200	296	126363	188983	5603231	1.5	44.3
S8 S	200	296	169902	328062	5603231	1.9	33.0

Statistical analysis is carried out to evaluate the fatigue performance of the current experiments and the results collected from the literature using the procedure described in [177, 178]. The slope parameter m is fixed to 3.00 (fully informative prior), in line with (pr)EN 1993-1-9 [16, 17].

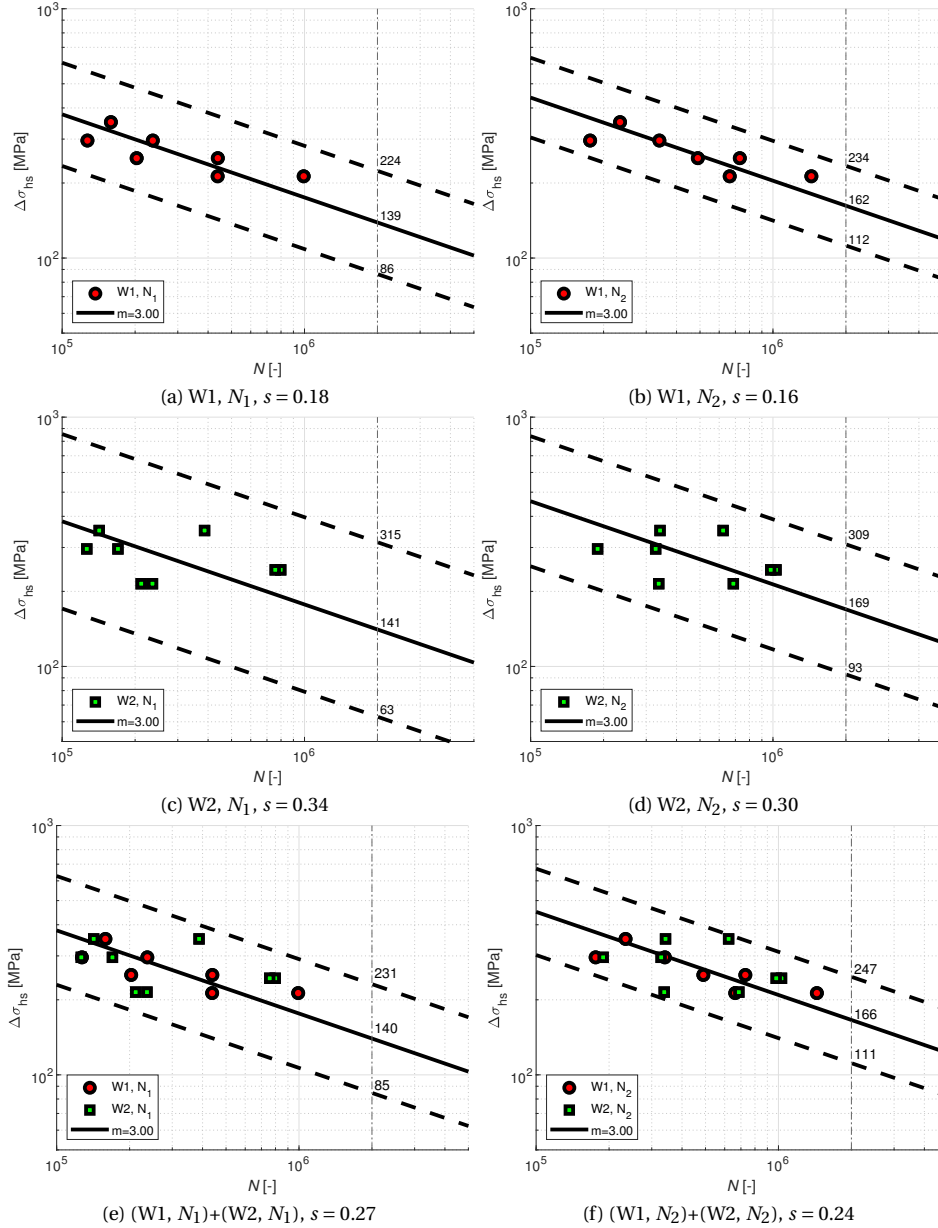


Figure 5.14: Statistical analysis for the current experiments with the 20 mm thick deck plate.

Figure 5.14 shows the statistical analysis for the evaluation criterion N_1 or N_2 . The numbers in the graphs present the fatigue resistances at 2 million cycles with 5%, 50%, and 95% exceedance fractions. The fatigue resistances with the 50% exceedance fraction of W1 and W2 are approximately equal, whereas the standard deviations (s) of W1 (0.16 and 0.18) is smaller than W2 (0.30 and 0.34), resulting in a lower $\Delta\sigma_{C,hs}$ for W2. Combining W1 and W2, $\Delta\sigma_{C,hs}$ are 85 MPa and 111 MPa with $s = 0.27$ and $s = 0.24$ for N_1 and N_2 , respectively.

EXPERIMENTS WITH LONG LOAD PATCHES

Figure 5.15 combines the current experiments with the 20 mm thick deck plate and the results in the literature with 10 or 12 mm thick deck plates. All data are obtained with the load patches with $W_q = 320$ mm (dedicated as long load patches).

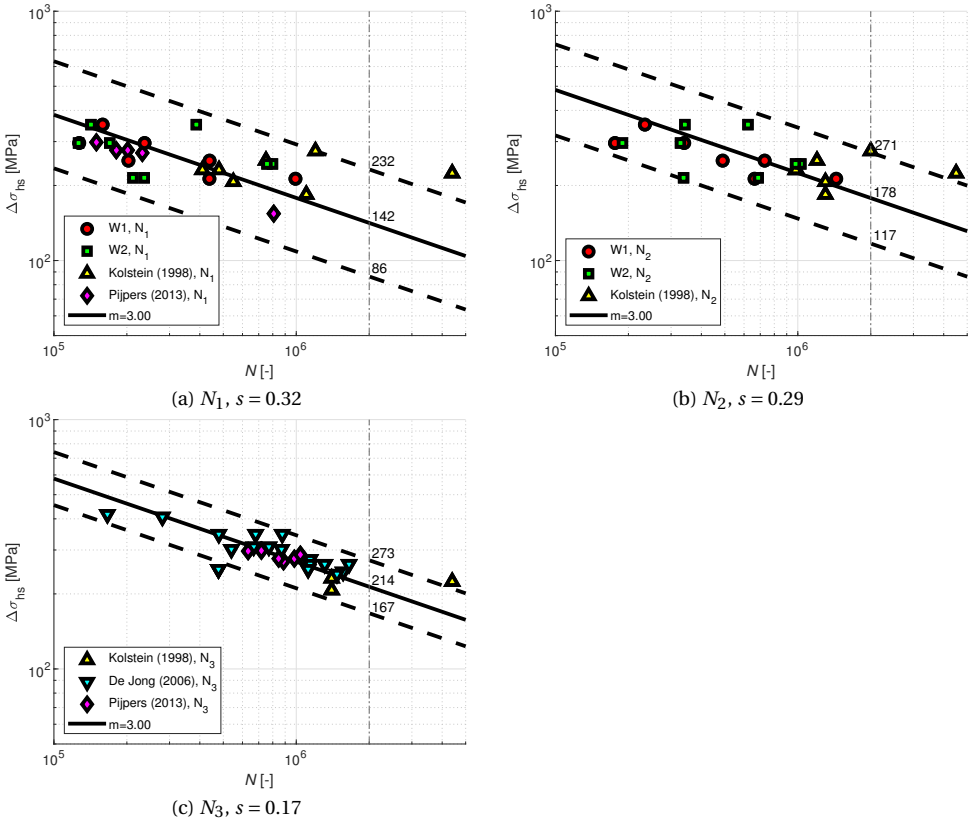


Figure 5.15: Statistical analysis for the joint database using long load patches [1, 14, 205].

Fatigue resistances with 50% and 5% exceedance fractions of the joint dataset are approximately equal to those of the 20 mm deck plate sub-dataset for N_1 and they are slightly higher for N_2 . The characteristic resistances $\Delta\sigma_{C,hs}$ related to N_1 and N_2 , are 86 MPa and 117 MPa, respectively. They are (almost) equal to the characteristic resistances given in the standards AASHTO LRFD Bridge Design Specifications: 2020 [19] (Category C, equal to $\Delta\sigma_C = 90$ MPa) and the Dutch National Annex to EN 1993-2: 2011 [206] ($\Delta\sigma_C = 125$ MPa), respectively. $\Delta\sigma_{C,hs}$ for N_3 is 167 MPa, obtained from the sub-dataset with thin deck plates only because (almost) no top surface breaking cracks are observed for thick deck plates. Note that $\Delta\sigma_{C,hs}$ for N_3 is much higher than the fatigue resistances for recommended by IIW [23] and in (pr)EN 1993-1-9 [16, 17]. $\Delta\sigma_{C,hs} = 100$ MPa is recommended by both documents for weld toe cracks at non-loaded fillet welded connections. Maljaars *et al.* [126] gave possible reasons for the relatively high resistance of detail C1c:

- Whereas fatigue cracks in welded connections usually either initiate from the plate at the weld toe and propagate in the base metal or initiate from the weld root and propagate in the weld throat, the crack in detail C1c initiates from the weld root and propagates in the base metal. It is, hence, a different type of crack.
- The fatigue resistance in the standards is (conservatively) given for tension-tension loading, whereas detail C1c is subject to the bending of deck plate.
- A load shedding effect occurs for detail C1c, where the stress reduces with the distance from the crossbeam. This gives a lower propagation rate as compared with usual details.

EXPERIMENTS WITH SHORT LOAD PATCHES

In [123, 124], experimental investigations were carried out but using shorter load patches with a size of $260 \times 200 \text{ mm}^2$ with 12 mm thick deck plates [124], and $200 \times 200 \text{ mm}^2$ with 12 or 16 mm thick deck plates [123], respectively. Surface cracks were found only in the specimens with 12 mm thick deck plates. Figure 5.16 shows the statistical analysis for the specimens loaded using short load patches. The hot spot stress ranges are obtained by FE calculations (validated against the strain gauge measurements) using the surface extrapolation approach designated in TS [27]. This is because the specimens in [123, 124] are approximately 300 mm wide (in the longitudinal weld direction) and the hand calculation model is not verified for this type of specimen. High fatigue resistance, $\Delta\sigma_{C,hs} = 277 \text{ MPa}$, is observed, which is even higher than the values with long load patches. The FE calculation in Section 5.4 explains the difference: The maximum principal stress (in absolute value) decays faster with the distance from the centrelines of the crossbeams for short load patches.

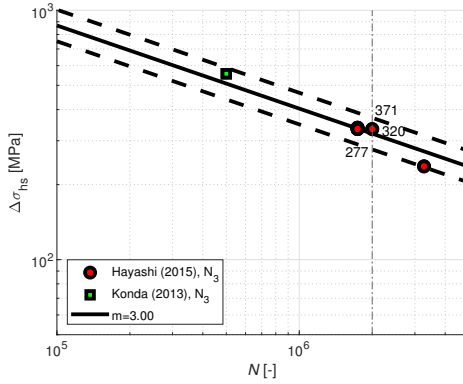


Figure 5.16: Statistical analysis for the specimens loaded using short load patches (N_3 , $s = 0.09$) [123, 124].

5.4. FINITE ELEMENT ANALYSIS

5.4.1. DESCRIPTION OF THE FE MODEL

Figure 5.17 gives an overview of the FE model generated and analysed using the commercial software Abaqus 2019 [179], dedicated to simulating Phase 1. A symmetrical model is built to reduce the computational cost. Symmetry in the Z-axis direction is not used to study Phase 2, which is not further discussed here. The FE model is 1300 mm long, and it contains one-and-a-half closed stiffeners. A “Static” load step is used with the implicit calculation method [179]. The bottom side of the lower flange is constrained in the vertical displacement direction ($U_Y = 0$). The symmetry boundary conditions with $U_X = 0$ for Sides 1 and 2, and $U_Z = 0$ for Sides 3 and 4. Note that a sensitivity study was carried out using different boundary conditions for Sides 1, 3, and 4, varying these boundaries from free of constraints to fully constrained in all directions with no noticeable effect on the local stress at the hot spot. The results given below are based on the symmetry boundary conditions at the four side planes. A uniform pressure load is applied at the tyre contact surface of Phase 1. The linear elastic material is applied in the model with the elastic modulus $E = 210$ GPa and the Poisson's ratio $\nu = 0.3$. 20-node quadratic brick elements with reduced integration (type C3D20R [179]) are used with a locally refined mesh (see Figure 5.18 for the nodal locations shown as markers).

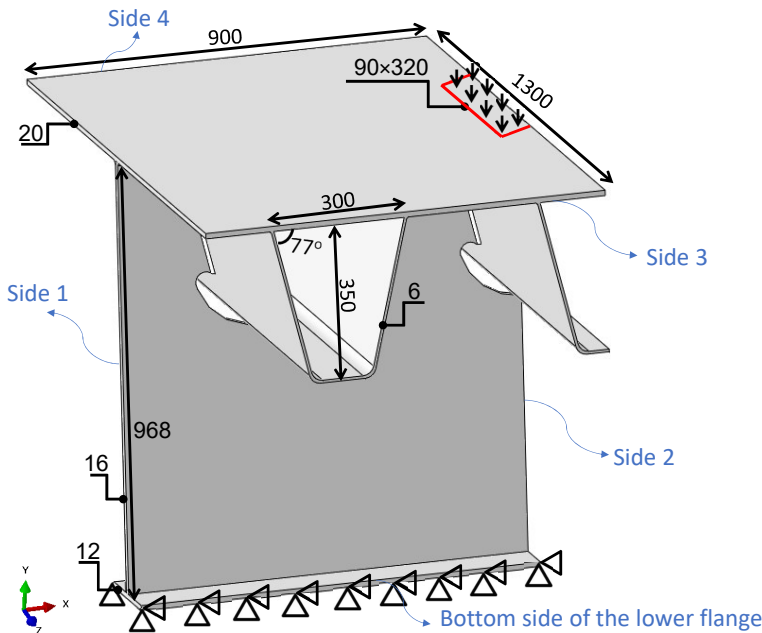


Figure 5.17: Overview of the FE model for detail C1c under Phase 1 loading (unit: mm).

5.4.2. MODEL VALIDATION

Figure 5.18 compares the strain ranges of the experimental measurements and the FE calculation. The values are normalised to the load range of 1 kN. The strain ranges at the top of the deck plate increase from 2×10^{-6} to 3×10^{-6} when approaching the crossbeam, Figure 5.18b. The results of the FE calculation are close to the average values from the measurements. Overall, the FE model gives a reasonable agreement with the experimental measurements both qualitatively and quantitatively.

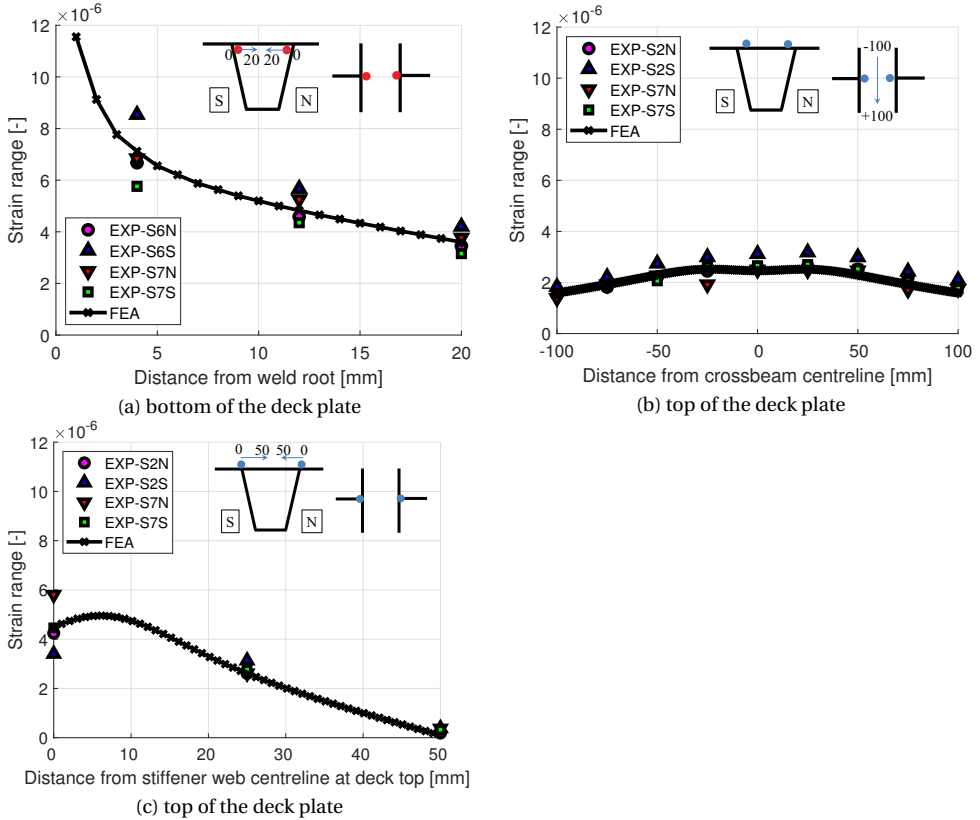


Figure 5.18: Comparison of the experimental measurements and the FE calculation under the normalised load range of 1 kN.

5.4.3. LOCAL STRESS STATE

The effective notch stress [17, 23] is applied to study the local stress state at the weld root. Figure 5.19 shows the FE model same as the one in Figure 5.17, but with two denser meshed sub-regions (Sub 1 and Sub 2). Tie constraints are used at the interfaces [179] between the sub-regions and the global region. Approximate element sizes in the global, Sub 1, and Sub 2 regions are $50 \times 50 \times 50 \text{ mm}^3$, $8 \times 8 \times 8 \text{ mm}^3$, and $0.5 \times 0.5 \times 0.5 \text{ mm}^3$, respectively. Four models are constructed with a lack of penetration adopted in the Sub 2 region of 0.0 mm, 0.5 mm, 1.0 mm, and 1.5 mm. The effective notch stress at the weld

root is modelled with an arch with the radius of 1 mm following the IIW recommendations [23]. Figure 5.20 shows the principal stresses (in absolute value) and the von Mises stresses around the notches at the location in conjunction with the crossbeam under the normalised load of 1 kN. The minimum principal and maximum von Mises effective notch stress values range from -7.8 to -7.6 MPa (4.9 to 5.0 times $\Delta\sigma_{hs}$ in absolute value) and from 6.8 to 7.0 MPa (4.3 to 4.5 times $\Delta\sigma_{hs}$), respectively. This indicates that a lack of penetration ranging between 0 and 1.5 mm has no significant effect on the effective notch stress, similar to the findings in [98] with a 12 mm thick deck plate. The low sensitivity of the resistance to local weld geometry, as determined by the effective notch stress, may also explain the low sensitivity to the welding procedure observed in the experiments. Figure 5.21 compares the principal stress (in absolute value) distribution loaded with different patch sizes. A long load patch ($90 \times 320 \text{ mm}^2$) results in a more uniform stress distribution (smaller peak stress at the weld root at the crossbeam location) than a short load patch ($90 \times 200 \text{ mm}^2$). See Figure 5.22 for the principal stress (in absolute value) of the two selected paths, Path 1 or 2 at the bottom of the deck plate with the direction along the longitudinal weld with the distance 2.2 mm or 4.4 mm from the weld root, and the ratio of principal stress between the short patch ($90 \times 200 \text{ mm}^2$) and the long patch ($90 \times 320 \text{ mm}^2$).

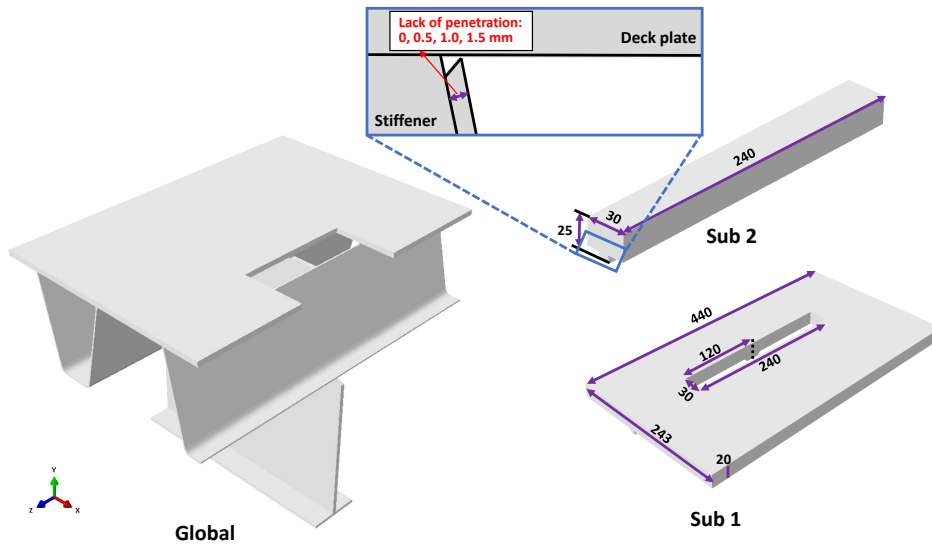


Figure 5.19: Illustration of the FE model with the global, Sub 1, and Sub 2 regions (unit: mm).

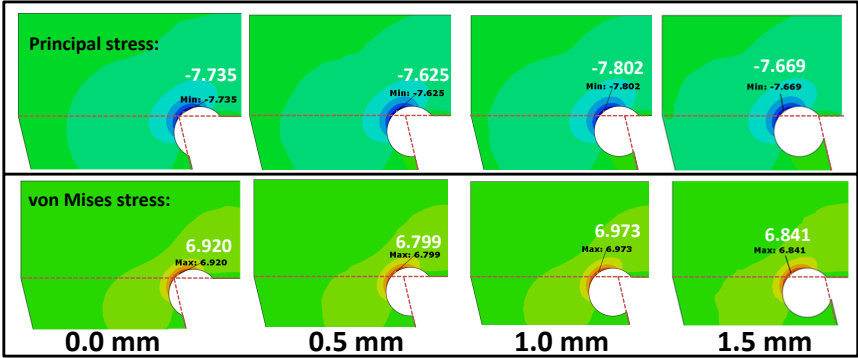


Figure 5.20: Effective notch stress at the weld root located in conjunction with the crossbeam with different lack of penetrations under the normalised load of 1 kN.

5

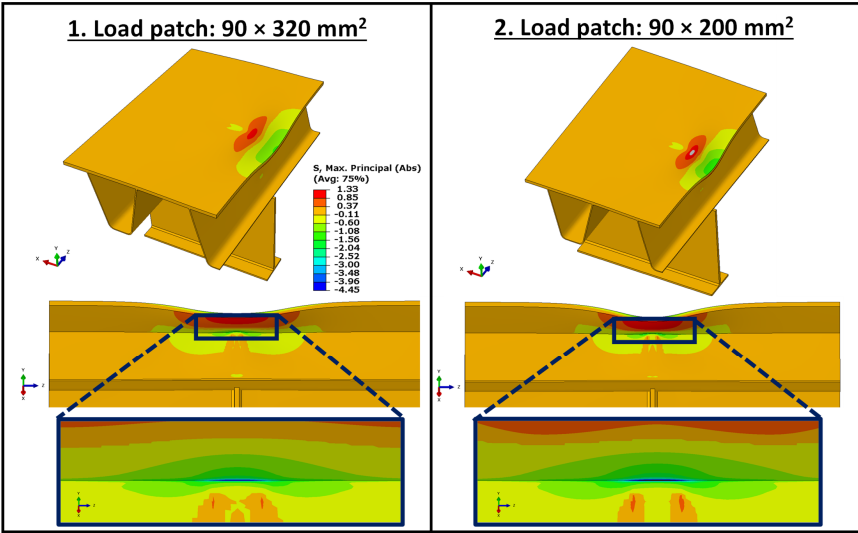


Figure 5.21: Principal stress distribution under the normalised load of 1 kN, loaded with different patch sizes (deformation scale: 20000).

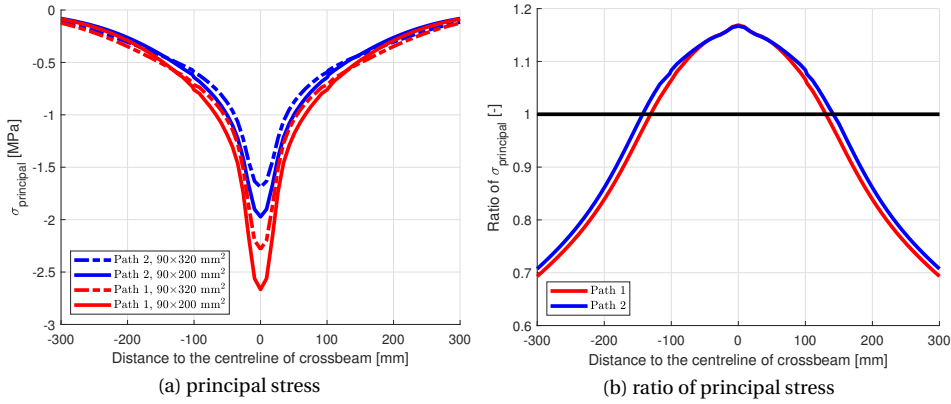


Figure 5.22: Principal stress distribution along the selected paths in Figure 5.21.

FRACTURE MECHANICS ANALYSES OF CRACK SHAPES

5

Further FE analyses are conducted to evaluate crack arrest and crack propagation paths. The XFEM is used to study crack propagation in conjunction with the Paris' equation. Four models with 10 mm, 12 mm, 16 mm, or 20 mm thick deck plates are created as presented in Figure 5.19. Here, the global and Sub 2 regions are meshed using the 8-node linear brick elements with reduced integration (C3D8R) and the Sub 2 region is activated as the enriched region for the XFEM [179]. Two load patches with sizes of $270 \times 320 \text{ mm}^2$ (for the 12 mm deck plate) and $180 \times 320 \text{ mm}^2$ (for other deck plate thicknesses) are applied. A semi-circular-shaped crack is inserted at the weld root with a horizontal shift of 0.3 mm from the weld root to avoid singularity, see Figure 5.23. The depth and the semi-length of the initial crack are $a = 0.5 \text{ mm}$ and $c = 0.5 \text{ mm}$, based on the detectability of flaws [34], respectively. The initial angle assumed between the crack plane and the vertical direction, θ , is 0° for the 10 mm and 12 mm thick deck plates [126], and 30° for the 16 mm and 20 mm thick deck plates [199] following the observed crack shapes. The Virtual Crack Closure Technique is used to calculate the strain energy release rate [179]. The element breaks at the maximum value location in each increment with the propagation direction following the Maximum Tangential Stress criterion, see Equation (5.5) where $\hat{\theta}$ is the crack propagation angle [179]. Cyclic loading is applied in a “low-cycle fatigue analysis” step. Fatigue crack propagation is calculated by the modified Paris' equation using the range of strain energy release rate, see Section 3.4.3 for more information. The material properties are set as $m = 3.00$ (dimensionless), $C = 1 \times 10^{-13}$ (units in millimetre and Newton), and $\Delta K_{th} = 63$ (units in millimetre and Newton) following [126]. Tensile residual stresses are assumed at the weld root of stiffener-to-deck plate connection, and the crack is assumed to remain open during the entire load cycle [126]. This study focuses only on the analysis of the fatigue crack propagation shapes and the crack arrest phenomenon using the LEFM. The applied forces are high enough ($\Delta\sigma_{hs}$ between 200 MPa and 250 MPa) to propagate the crack at the initial state, whereas the fracture toughness is not considered. Local plastic failure is excluded from the analysis. A fatigue life calculation can be done using the XFEM [207], which is left outside the scope of the

current study.

$$\hat{\theta} = \arccos \left(\frac{3K_{II}^2 + \sqrt{K_I^4 + 8K_I^2 K_{II}^2}}{K_I^2 + 9K_{II}^2} \right) \quad (5.5)$$

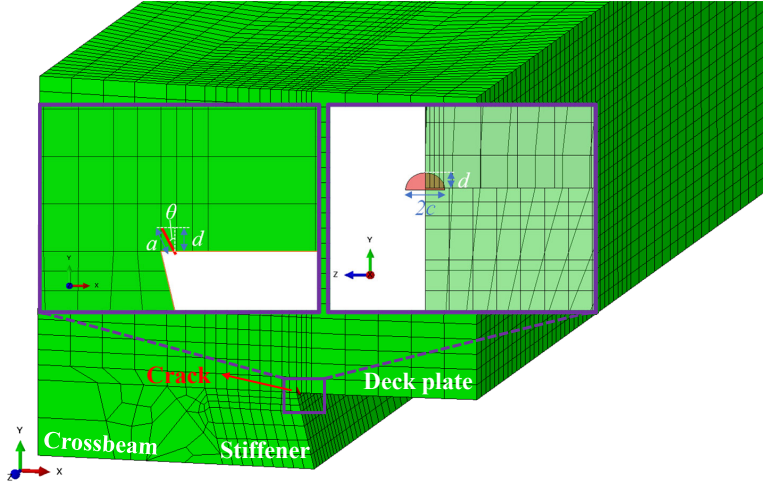


Figure 5.23: Illustration of the XFEM model for detail C1c.

Figure 5.24 illustrates the crack shapes of the four simulations. A three-dimensional crack with a shape close to the semi-ellipse is observed for all deck plate thicknesses. The crack propagates in the depth and the length directions until it breaks through the top surface of the deck plate in the 10 mm thick deck plate simulation (defined as Failure type A). Crack arrest is predicted for the 12 mm thick deck plate simulation but very close to the top surface (defined as Failure type B). It then propagates further in the length direction. Crack arrest in the depth direction occurs after the crack depth reaches approximately 75% of the deck plate thickness with the 16 mm and 20 mm thick deck plates, whereas the crack propagates further in the length direction (defined as Failure type C), see the lengths at the top ($2c_{top}$) and the bottom ($2c_{bottom}$) of the deck plates in Figure 5.24. In the simulations, the cracks incline towards the horizontal direction. The results are in qualitative agreement with the experimental observations in [14, 98, 123, 124, 205].

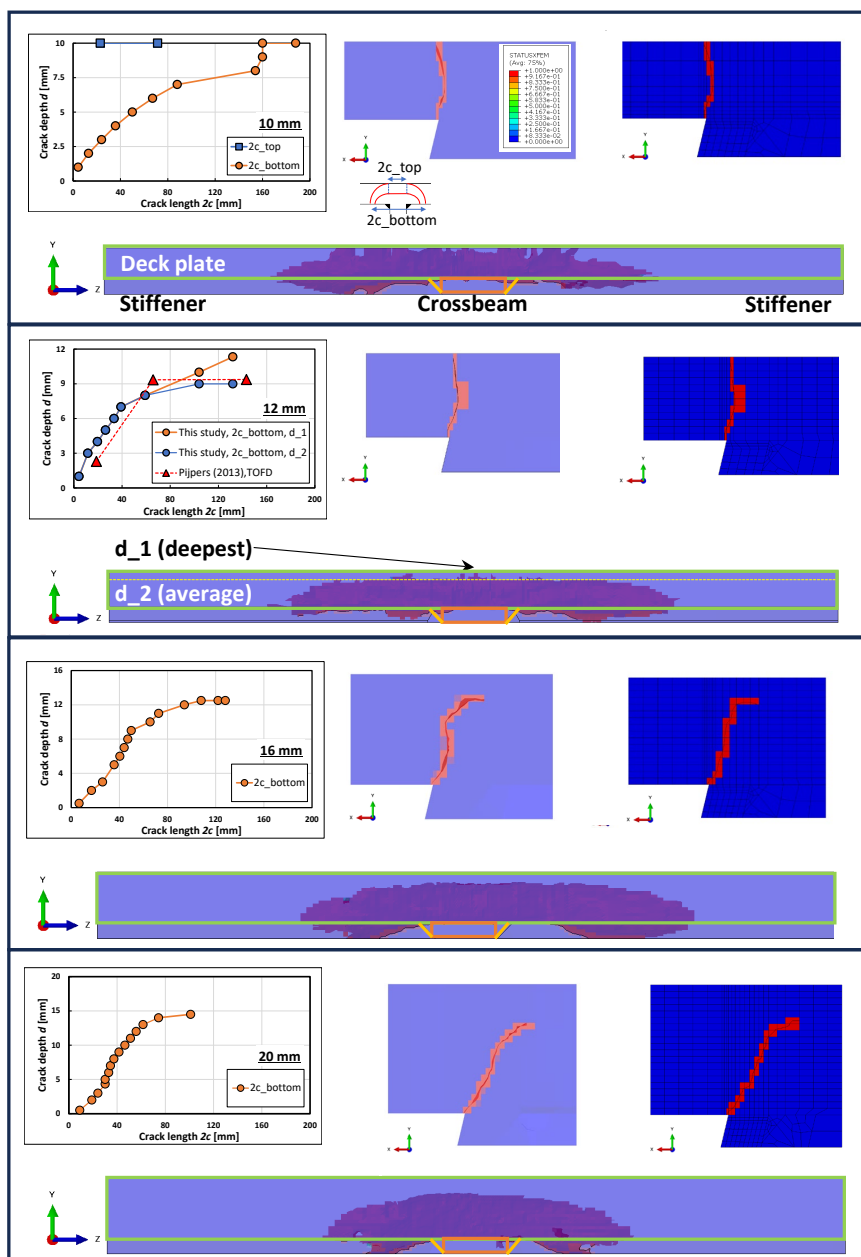


Figure 5.24: Crack shapes from the XFEM for the 10 mm, 12 mm, 16 mm, and 20 mm thick deck plates (the load patch size is $270 \times 320 \text{ mm}^2$ for the 12 mm thick deck plate, and it is $180 \times 320 \text{ mm}^2$ for the rest) [205].

5.5. SUMMARY OF THE CHAPTER

Experimental investigations and finite element analyses are used for the fatigue crack of stiffener-to-deck plate welded connection at crossbeam (detail C1c). Unlike previous publications, the deck plate is relatively thick in the current study, thereby representing a new generation of OBDs. One full-scale specimen with a 20 mm thick deck plate is loaded, resulting in sixteen C1c type cracks. The stress and strain distribution, the crack propagation, the fatigue resistances for the hot spot stress, and the effect of the lack of weld penetration are studied. Additionally, fatigue crack propagation paths are predicted using the extended finite element method for different plate thicknesses. The main conclusions are:

1. The maximum stress in stiffener-to-deck plate welded connection at crossbeam occurs at the weld root. Fatigue cracks that initiate from the weld root at crossbeam and propagate into deck plate are observed in the experiments. With the 20 mm thick deck plate, the crack arrest occurs at an approximate depth between 14 mm and 15 mm. This is approximately 75% of the deck plate thickness.
2. Using the author's own experiments and the results in the literature, the recommended characteristic fatigue resistances for the hot spot stress are:
 - **112 MPa** for early cracking according to the measurement of strain gauges.
 - **170 MPa** for the earliest visible crack detection criterion from the top of deck plate with the deck plate thickness of 10 or 12 mm.
3. A lack of penetration of up to 1.5 mm, Partial Joint Penetration (PJP) = 75% with 6 mm stiffener web, does not significantly influence the effective notch stress.
4. The crack propagation path predicted by the extended finite element method in conjunction with the Paris' equation is similar to the experimental results for 10 mm, 12 mm, 16 mm, and 20 mm thick deck plates. After reaching 75% of total deck plate thickness, the predicted crack stops propagating in the thickness direction for deck plates thicker than 12 mm. The through-thickness crack is predicted in the 10 mm thick deck plate. For the 12 mm thick deck plate, the final crack is predicted to be very close to the top surface of the deck plate and can break through at high load levels. Moreover, cracks are predicted in the weld throat and the stiffener at the junction with the crossbeam, in line with experimental observations.

6

WELD ROOT CRACK IN CROSSBEAM-TO-STIFFENER WELDED CONNECTION WITHOUT CUT-OUT (C4D)

Fatigue behaviour of crossbeam-to-stiffener welded connection without cut-out is studied in Chapter 6. Out-of-plane cyclic loading is applied on the free edge side of the unloaded (Crossbeam I) region with the crossbeams fully supported using the full-scale specimen introduced in Chapter 5. Fatigue cracks are first observed at the weld surfaces and then propagate to the crossbeam, indicating weld root cracks. Fatigue crack initiation is obtained by either a certain percentage change of the strain range in the vicinity of the crack initiation location or the naked eye inspection. The propagation of the cracks is monitored during the experiments. Relatively slow propagation rates are observed for the surface cracks that grow in the crossbeam. Finite element models are built to study the stress distribution in the vicinity of the details. The finite element models are validated by comparing the calculated strains with the measured values. Afterwards, the nominal weld stress, structural weld stress, and effective notch stress at the weld root are calculated. Fatigue assessment of the experimental results is carried out with five fatigue failure criteria using the three stress methods. Finally, one detail category based on the nominal weld stress with two fatigue resistances aiming for different applications is established using the author's experiments and the results in the literature.

6.1. INTRODUCTION

Crossbeam-to-stiffener welds connect longitudinal stiffeners and crossbeams. The stiffeners can either be continuous or discontinuous as shown in Figure 6.1. The connection welded all around the stiffener without cut-out is a commonly used type in design [10]. Under traffic loading, fatigue cracks may initiate at the bottom of stiffener from the weld toe (C3c, C4c, C3d, and C4e) or the weld root (C4d, and C3e) either at the stiffener or the crossbeam side. The cracks then propagate into the stiffener (C3c and C3d) or crossbeam (C4c and C4e), or the weld throat (C4d and C3e).

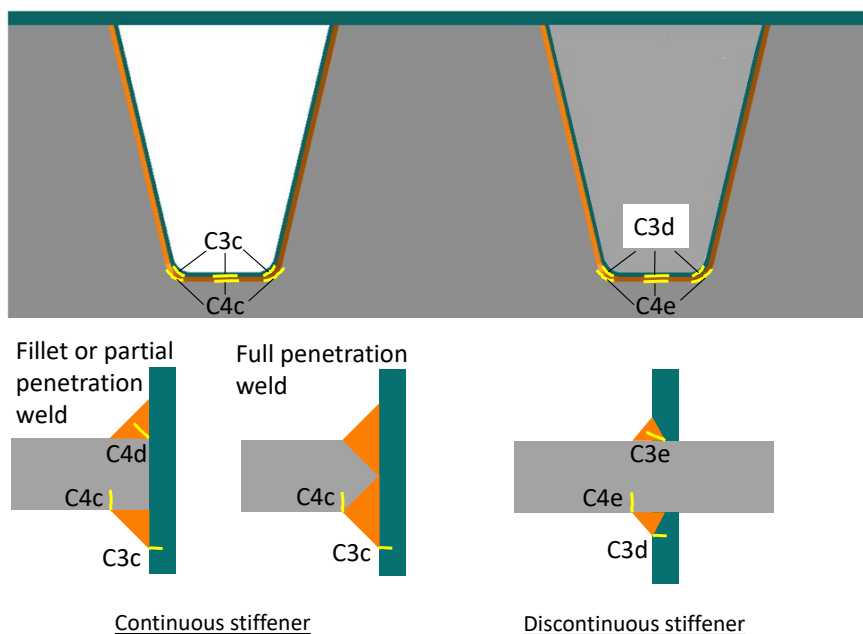


Figure 6.1: Illustration of possible fatigue cracks at the bottom of stiffener in crossbeam-to-stiffener welded connections without cut-out (continuous stiffener on the left and discontinuous stiffener on the right).

Fatigue behaviour of type C4d crack is studied in this chapter using experimental investigations and Finite Element (FE) calculations. The nominal weld stress, structural weld stress, and effective notch stress are applied to the verification of the studied details. Fatigue assessment of the experimental results is carried out with five fatigue failure criteria using the three stress methods. A comparison of the root cracking in fillet welded connections between the full-scale OBD specimen used in the author's own experiments and the results of load-carrying welded plates found in the literature is given using the nominal weld stress. One detail category with two fatigue resistances is established using the author's experiments and the results in the literature aiming for different applications. The detail was not covered in EN 1993-1-9: 2005 [16].

6.2. SPECIMEN, SETUP AND LOADING PROGRAMME

The full-scale OBD specimen consists of a $9400 \times 5100 \text{ mm}^2$ full-scale deck plate stiffened by eight trapezoidal stiffeners and three crossbeams, as introduced in Chapter 5. The Crossbeam III region of the specimen has been loaded in fatigue, as described in Chapter 5, to study the fatigue behaviour of detail C1c. In the current chapter, the experiments are carried out on eight details in the unloaded (Crossbeam I) region. Extended cut-outs with the Haibach-type shape are used on the north side of the specimen, whereas the crossbeam is welded around the entire perimeter of the stiffener (without cut-out) on the south side, see Figures 5.2 and 6.2. The latter is studied in the current chapter. The crossbeam-to-stiffener connections are automatically welded by one pass. The Flux-Cored Arc Welding (FCAW) is used with DC+ current. Fillet welds with a predefined effective thickness of 5 mm are created. The welding parameters are as follows: a 1.2 mm wire diameter, a current of 270 to 300 amperes, a voltage of 28 to 30 volts, a speed of 23 to 31 cm/min, and a heat input ranging from 1.5 to 2.4 kJ/mm. The welds meet the quality level B of ISO 5817 [173].

The load is moved to the free edge side of Crossbeam I as shown in Figure 6.2. The load patch with an area of $270 \times 320 \text{ mm}^2$, same as the Type C contact patch designated by the Fatigue Load Model 4 (FLM4) in EN 1991-2 [80], is centred between the stiffener webs with a distance of 600 mm from the middle of the Crossbeam I thickness. Cyclic load is applied through a hydraulic actuator that presses onto a 32 mm thick steel plate and two plates of rubber, 8 mm thick each. The load is shifted to the adjacent stiffener in each experiment. The strains at adjacent stiffeners are measured by strain gauges with negligible strains. Thus, two details at the corners of one stiffener are loaded at each load case. In total, eight details are loaded. A summary of the loading programme is shown in Table 6.1. The applied load ranges with a load ratio of 0.1 are between 160 kN and 220 kN, approximately 3.6 to 4.9 times 45 kN defined by the FLM4 in EN 1991-2 [80]. The load frequency is set to 3 Hz for S1, and 2 Hz for S2, S3 and S4.

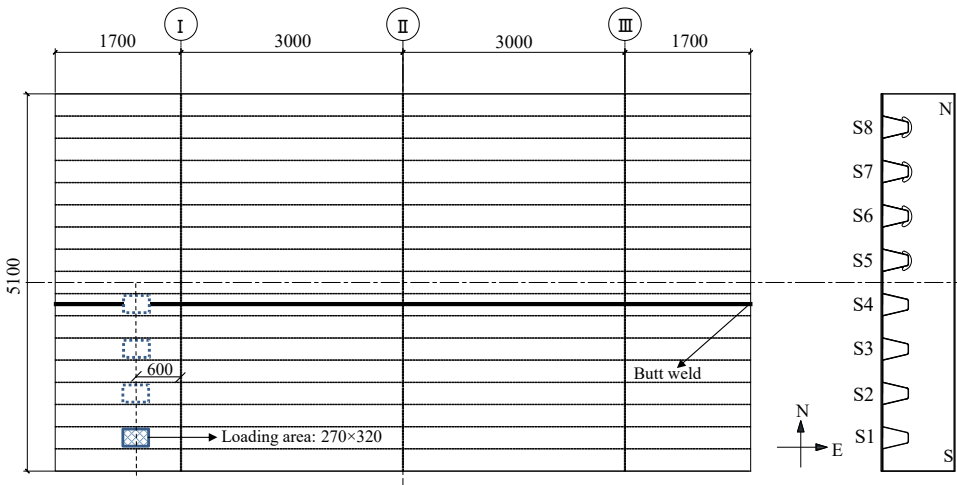


Figure 6.2: Overview of the loading positions for detail C4d (unit: mm).

Table 6.1: Summary of the loading programme for detail C4d.

Loaded detail	S1 N&S	S2 N&S	S3 N&S	S4 N&S
Load range [kN]	160	220	196	220
Frequency [Hz]	3	2	2	2
Load ratio	0.1	0.1	0.1	0.1
Cycles [million]	12.06	2.61	8.45	4.76
Run-out		S2 S:×		

6.3. STRAIN AND STRESS DISTRIBUTION

6.3.1. ARRANGEMENT OF STRAIN GAUGES

Strain gauges are attached at the bottom area of S1 and S4, see Figure 6.3 for the arrangement and the predefined positions. The directions of strain measurement are perpendicular to the weld toe, as shown by arrows. These strain gauges are used to measure the strain distribution of the studied details and monitor the strain ranges during the experiments. At the free edge side of the stiffeners, chain strain gauges (5 strain gauges intermediate spaced at 2 mm in a chain) are attached with the first strain gauge 4 mm away from the weld toe, followed by two strain gauges 25 mm and 50 mm away from the weld toe, at two corners (N and S) and at the middle (M). At the in span side of the stiffeners, strain gauges are only applied to S1 at two corners. At the free edge side of the crossbeam, similar arrangement is applied at two corners: the chain strain gauges are attached with the first strain gauge 4 mm away from the weld toe, followed by a single strain gauge 50 mm away from the weld toe. The number of strain gauges is reduced to two at the middle M, with distances 25 mm and 50 mm away from the weld toe. No strain gauges are positioned at the in span side of the crossbeam. For S2 and S3, strain gauges are positioned at the free edge side only. Same arrangement as S1 and S4 is used at the stiffener, whereas only one strain gauge 10 mm away from the weld toe is positioned at each corner of the crossbeam. The reduction of strain gauges is conducted as the strain distribution is expected to be similar for the four stiffeners. The numerical values are presented in Section 6.3.3.

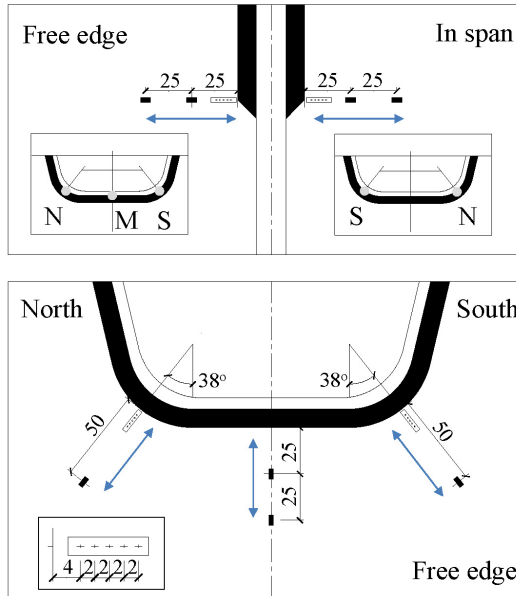


Figure 6.3: Overview of the strain gauge arrangement for S1 and S4 (unit: mm).

6.3.2. INTRODUCTION OF THE FE MODEL

A FE model is built using the commercial software package Abaqus 2019 [179]. Figure 6.4 gives an overview of the FE model with the loading and boundary conditions. The model is built following the geometry characteristics of the specimen described in Figure 5.2. Linear elastic material is used in the FE model with the elastic modulus of 210 GPa and the Poisson's ratio of 0.3. Uniform pressure is applied at the load patch, see Figure 6.2 for the loading positions. Displacement constraints are applied along the middle of the crossbeam thickness at the bottom flanges in three directions ($U_X = U_Y = U_Z = 0$).

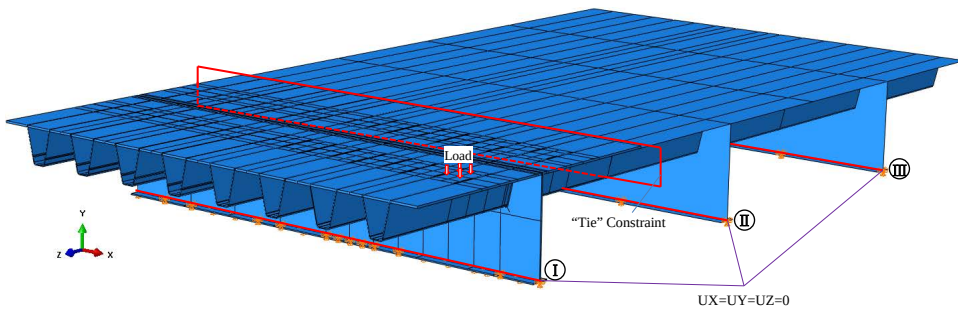


Figure 6.4: Overview of the FE model for detail C4d.

Figure 6.5 shows the mesh and details in the FE model. 8-node linear brick elements with reduced integration (type C3D8R) [179] are assigned to the Crossbeam I region, and

8-node quadratic shell elements with reduced integration (type S8R) [179] are assigned to the Crossbeams II and III regions. “Tie” constraints [179] are created at the intersects shown as the red rectangular in Figure 6.4 and the white dot line in Figure 6.5. The nodes of a “tie” constrained interface have the same displacements and rotations. The pre-defined effective weld thickness is 5 mm with weld legs equal to 7.07 mm. “Seam” is applied at the interfaces between the crossbeam and the stiffeners, which allows the departure of the nodes at the selected seam faces [179]. Four layers of elements are created in the thicknesses of the stiffeners and Crossbeam I. The brick elements have an approximately size of $25 \times 25 \times 25 \text{ mm}^3$ with local refined mesh at the bottom of the crossbeam-to-stiffener welded connections. Shell elements with an approximate size of $100 \times 100 \times 100 \text{ mm}^3$ are created in the Crossbeams II and III regions with refined mesh of $25 \times 25 \times 25 \text{ mm}^3$ at the intersects.

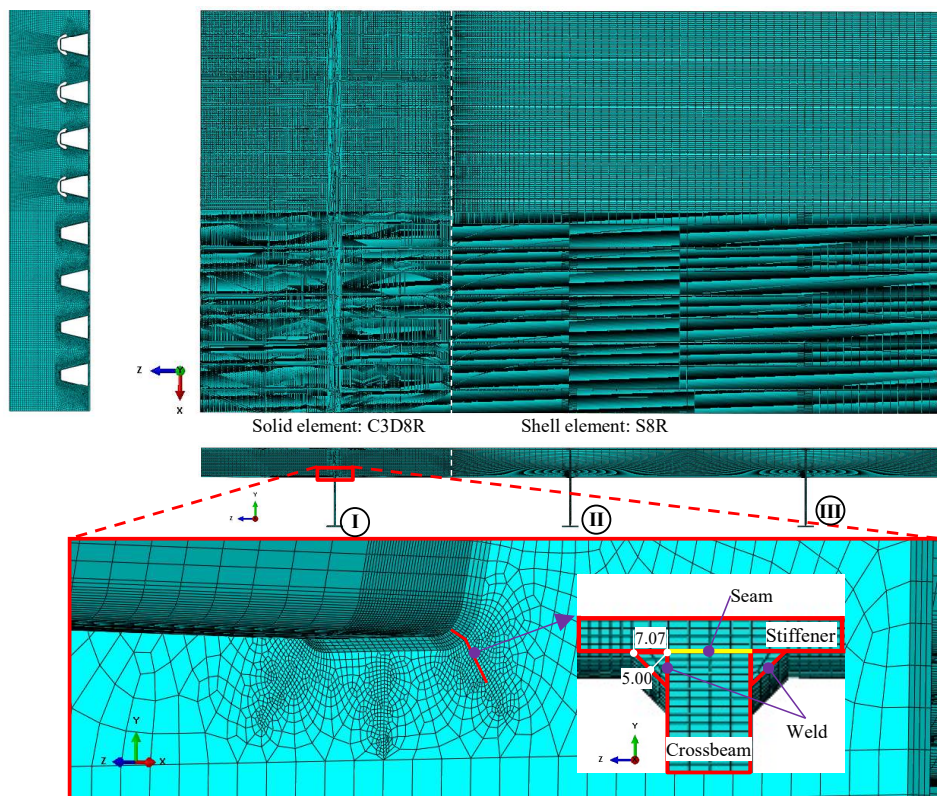


Figure 6.5: Mesh and details of the FE model for detail C4d (unit: mm).

6.3.3. STRAIN RANGE DISTRIBUTION

Figures 6.6 and 6.7 show the distributions of the strain ranges obtained either by Experiment measurement (EXP) or FE Analysis (FEA), under the normalised load range $\Delta F = 1 \text{ kN}$. The absolute strains in Figures 6.6 and 6.7 are all negative, whereas ranges

are always positive. The FEA shows a good agreement with the EXP in the studied details. At S1 N&S, the strain ranges at the free edge side are higher than the values at the in span side. At the corners of the stiffeners, concentrations are observed with steep strain gradients when approaching the weld toe. The slope becomes gentle with the distance approximately exceeding 30 mm. Higher strain ranges are observed in S1, which can be explained by the weaker constraints at edge of the specimen. At the crossbeam side, strain ranges are significantly smaller than the values at the stiffener side. The results are logical as the crossbeam is 16 mm thick, whereas the stiffeners are only 6 mm thick. Note that the strain ranges of the FEA in S2, S3 and S4 are very close and the curves are overlapped. Some markers of the EXP are also overlapped at: the stiffener side of S2 N; the stiffener side of S2 S and S3 S; the stiffener side of S2 M and S3 M; the stiffener side of S1 S (in span side); the crossbeam side of S1 N; the crossbeam side of S1 S and S3 S; and the crossbeam side of S1 M.

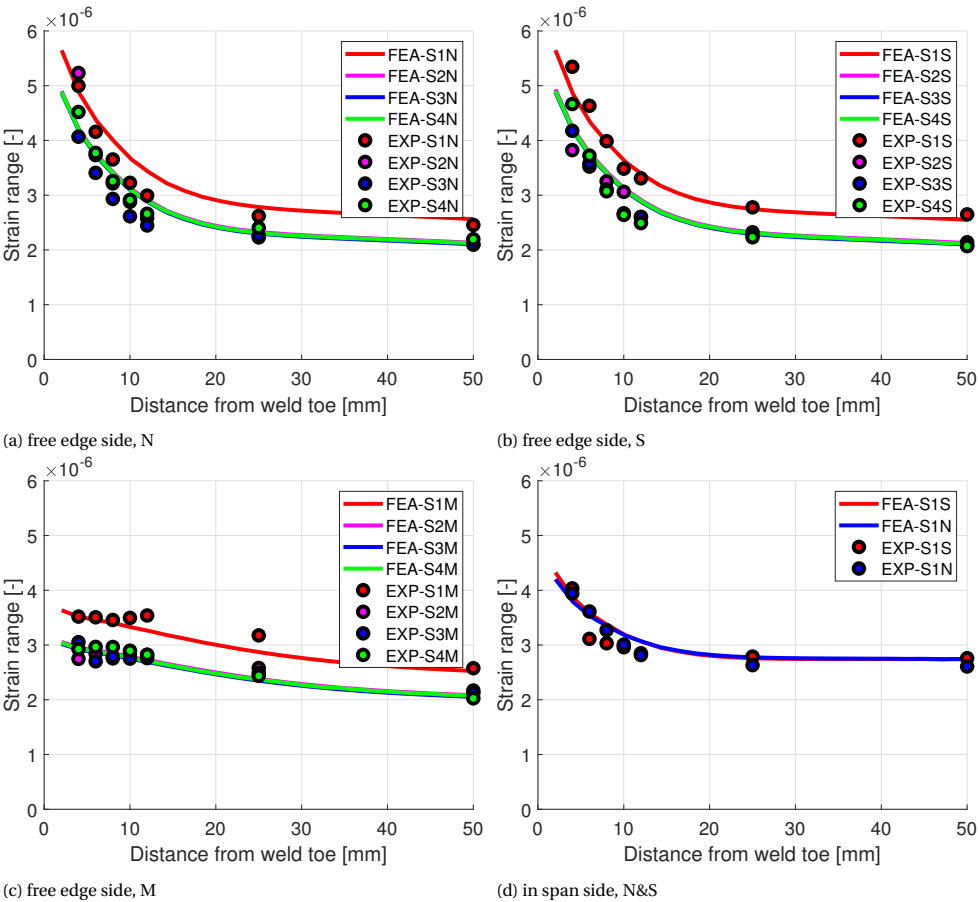


Figure 6.6: EXP and FEA obtained strain ranges at the stiffener side, $\Delta F = 1$ kN.

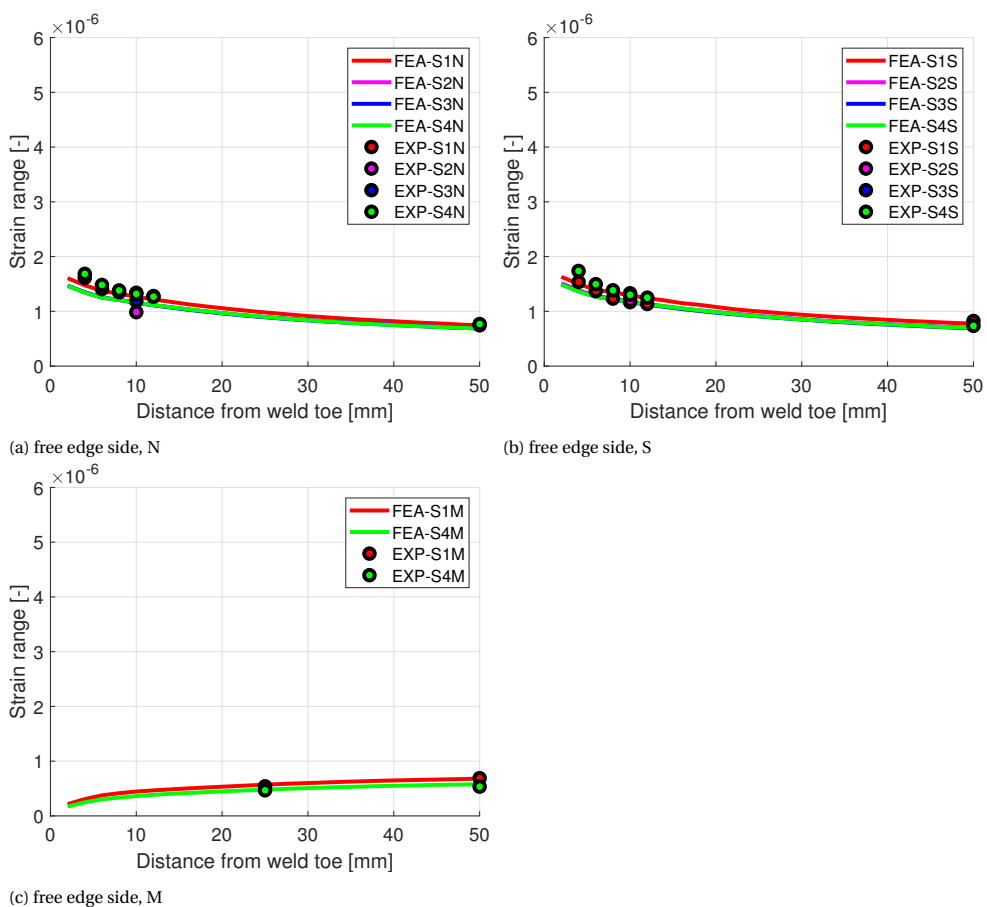


Figure 6.7: EXP and FEA obtained strain ranges at the crossbeam side, $\Delta F = 1$ kN.

6.4. STRESS EVALUATION USING THE FE METHOD

This section evaluates detail C4d based on the FE model validated in Section 6.3.3. The nominal weld stress, structural weld stress, and effective notch stress are used for the evaluation.

6.4.1. STRESS IN THE WELD THROAT

The stress in the weld throat is analysed for type C4d crack that initiates from the weld root and propagates into the weld throat. Figure 6.8 shows the stress state in the weld throat of a double-sided fillet welded connection under the axial force and the bending moment. The nominal weld stress, $\sigma_{w,n}$, can be calculated using Equations (6.1) and (6.2). First, the nominal bending stress ($\sigma_{b,n}$) and the nominal membrane stress ($\sigma_{m,n}$) need to be calculated. Second, $\sigma_{w,n}$ is obtained by the superposition of $\sigma_{b,n}$ and $\sigma_{m,n}$. The structural weld stress, $\sigma_{w,hs}$, consists of the bending stress ($\sigma_{w,b}$) and the membrane stress ($\sigma_{w,m}$) in the weld throat. $\sigma_{w,hs}$ can be calculated by linearly extrapolating the stresses at predefined locations to the hot spot location, which is the weld root in this case. In addition to the normal stress (perpendicular to the weld surface), the principal stress is used in [49] for the calculation of $\sigma_{w,hs}$.

$$\sigma_{b,n} = \frac{M_{CB}}{(t_w + t_{CB}) t_w b} \quad (6.1)$$

$$\sigma_{m,n} = \frac{F_{CB}}{2 t_w b} \quad (6.2)$$

where t_w , t_{CB} and b are the effective weld thickness, the crossbeam thickness and the width of detail used in calculation, respectively.

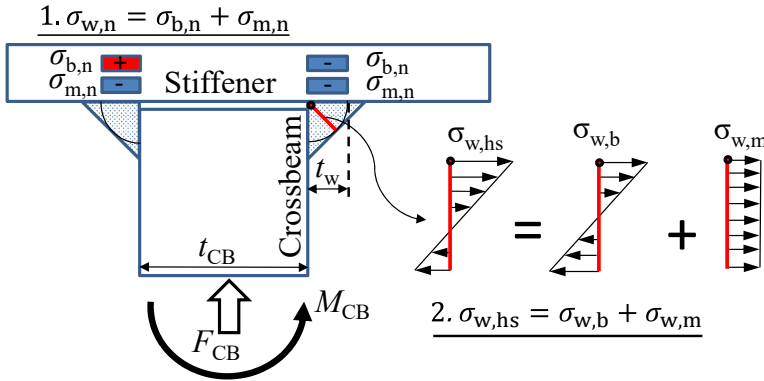


Figure 6.8: Illustration of the nominal weld stress and the structural weld stress in a double-sided fillet welded connection.

Figure 6.9 illustrates the calculation of $\sigma_{w,n}$ in the FE model. The critical plane (red line) is located in the middle of the corner curve [1, 208]. The angle to the vertical black line is 38° for the details in the full-scale specimen. The normal stress (perpendicular to the weld toe at the crossbeam side) in the cross sections of the crossbeam at the critical plane with distances of 0.0 mm, 2.1 mm, 4.3 mm, and 6.4 mm from the weld toe is evaluated. The cross section in which the stress is (approximately) linearly distributed is used to calculate the bending moment M_{CB} and the axial force F_{CB} . The stress in the selected cross section is linearly extrapolated and the stress is substituted into the bending stress $\sigma_{b,CB}$ and the membrane stress $\sigma_{m,CB}$. The Degree Of Bending (DOB) is designated by Equation (6.3).

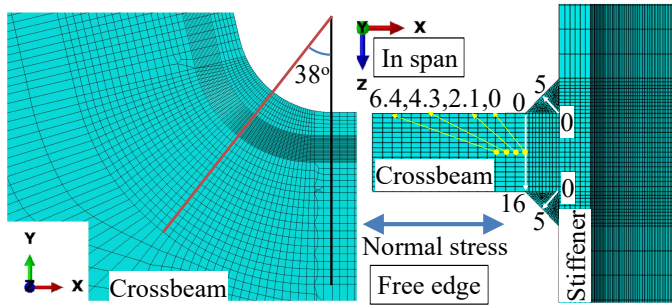


Figure 6.9: Illustration of the calculation of $\sigma_{w,n}$ in the FE model (unit: mm).

$$DOB = |\sigma_{b,CB}| / (|\sigma_{b,CB}| + |\sigma_{m,CB}|) \quad (6.3)$$

Using the obtained $\sigma_{b,CB}$ and $\sigma_{m,CB}$, M_{CB} and F_{CB} are calculated using Equations (6.4) and (6.5). The nominal bending stress $\sigma_{b,n}$ and the nominal membrane stress $\sigma_{m,n}$ can therefore be obtained using Equations (6.1) and (6.2). The superposition of both results is the nominal weld stress $\sigma_{w,n}$.

$$M_{CB} = \sigma_{b,CB} \frac{t_{CB}^2 b}{6} \quad (6.4)$$

$$F_{CB} = \sigma_{m,CB} (t_{CB} b) \quad (6.5)$$

NOMINAL WELD STRESS

Figure 6.10 plots the normal stress along the crossbeam thickness in Paths 1, 2, and 3. Largest absolute stress occurs at Path 1 with the non-linear distribution close (with the distance 0.0 mm or 2.1 mm) to the weld toe. Note that the element sizes in Path 3 are not the same as the ones in Paths 1 and 2, resulting in a slightly different distribution of the stress extraction points, see Figure 6.10c. Linear fit of the extracted stresses in the cross section 6.4 mm away from the weld toe with distances from the in span side ranging from 2 mm to 14 mm is shown in Figure 6.11. The solid lines and the dash-dotted lines represent the fitted lines ($\sigma_{b,CB} + \sigma_{m,CB}$) and the membrane stress components ($\sigma_{m,CB}$), respectively. In Path 1 (critical path), $\sigma_{b,CB} + \sigma_{m,CB} = -0.34$ MPa and $\sigma_{m,CB} = -0.18$ MPa. The DOB is approximately 0.5 and $\sigma_{w,n}$ is -0.35 MPa.

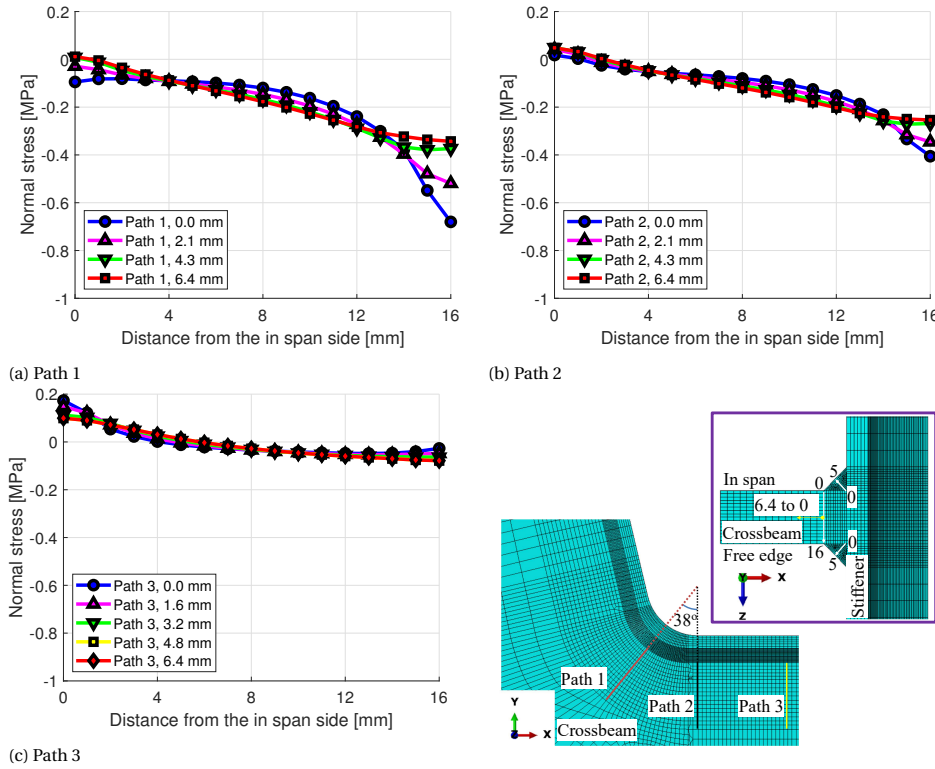


Figure 6.10: Normal stress in the four or five cross sections close to the weld toe in the three paths, S2 under $F = 1$ kN (unit: mm).

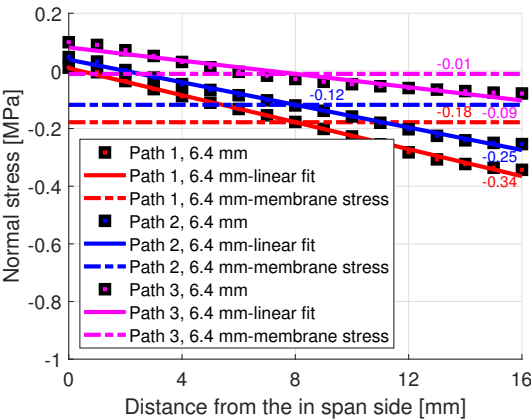


Figure 6.11: Linear fit of the normal stress and the substituted membrane stress in the cross section 6.4 mm away from the weld toe in the three paths, S2 under $F = 1$ kN (unit: mm).

STRUCTURAL WELD STRESS

Figure 6.12 shows the normal stress and the principal stress along the weld throat in Paths 1, 2, and 3. Among them, Path 1 shows largest values (in absolute) and is the critical path, which is in line with the results of Figures 6.10 and 6.11. At the weld root, the extracted nodal stresses are averaged by the connected elements and are smaller than the values at location 0.5 mm away from the weld root. The absolute stresses at the free edge side are larger than the values at the in span side, whereas the slope is more gentle. In this study, $\sigma_{w,hs}$ is calculated by linearly extrapolating the normal stresses at locations 1.25 mm ($0.25t_w$) and 3.75 mm ($0.75t_w$) to the weld root following reference [49], see Table 6.2 for numerical values.

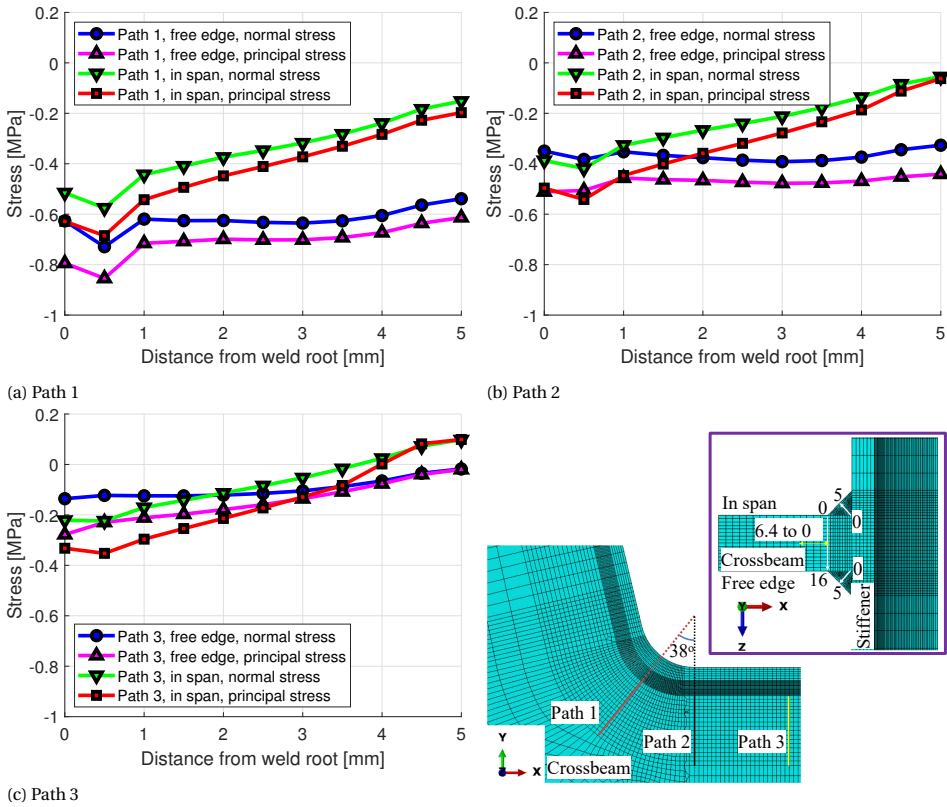


Figure 6.12: Normal stress and principal stress in the weld throat of the three paths, S2 under $F = 1$ kN (unit: mm).

EFFECTIVE NOTCH STRESS

A sub model is built and meshed by 20-node quadratic brick elements with reduced integration (type C3D20R) [179] with an approximate size of $0.75 \times 0.75 \times 0.75$ mm³. The sub-modelling method in Abaqus [179] bridges the difference in scale between the local geometric characteristics (sub model) and the overall specimen (global model). Figure 6.13 shows the sub model with a dimension of 51.5 mm and 41 mm in the Y-axis and

Z-axis directions, respectively. The distance between the crossbeam and the stiffener interacted surfaces is assumed to be 0.01 mm. The displacements at the intersections of the sub model are constrained to match the displacements of the global model as highlighted in red. Effective notches with $R_{\text{ref}} = 1$ mm are created at the weld roots with local refined mesh (see Figure 6.14) following the IIW recommendations [23].

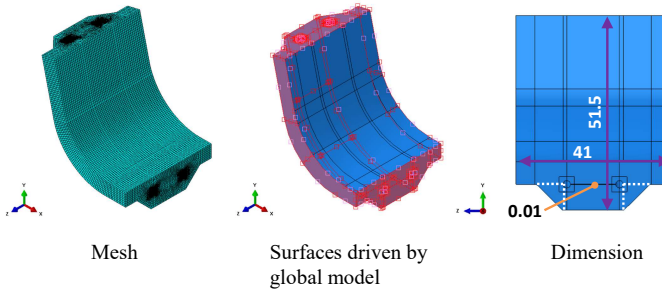
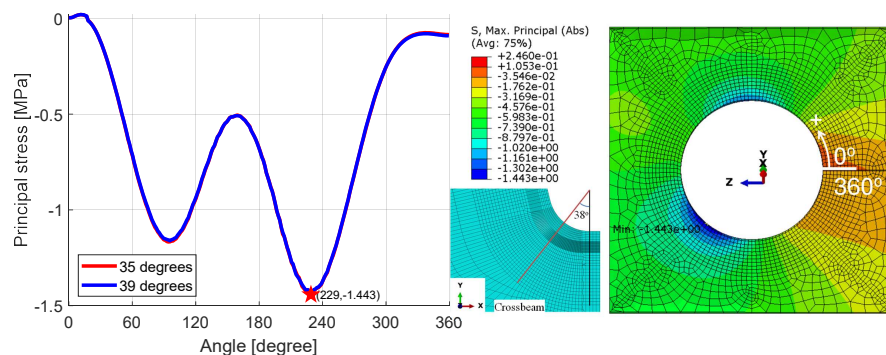
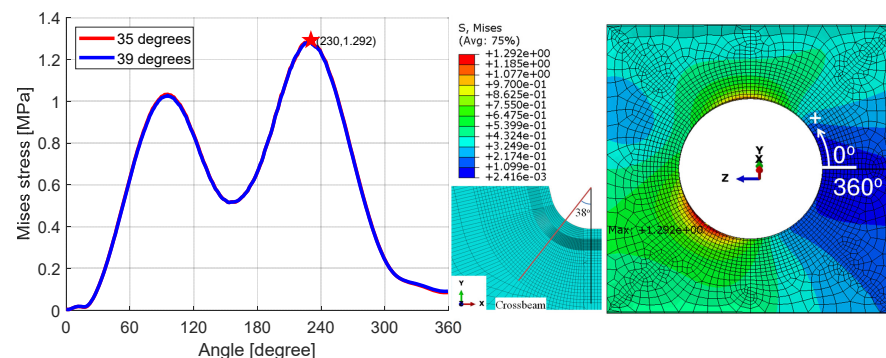


Figure 6.13: Illustration of the sub model for the effective notch stress (unit: mm).

Figure 6.14 shows the principal stress and the Mises stress along the notch circles at the free edge side at two critical planes with angles of 35° and 39° to the Y-axis direction. The most critical plane, 35° to the Y-axis direction, has the maximum absolute values of -1.443 MPa and 1.292 MPa for the principal stress criterion and the equivalent (Mises) stress criterion, respectively. Both values locate on the notch with an approximate angle of 230° , calculated clockwise from the weld root. The stress at the plane with an angle of 39° is plotted as comparison. Both principal stress and Mises stress are very close at the two planes. It confirms that Path 1 (38°) can be treated as the critical plane. The numerical values of the effective notch stress at the critical locations for S1, S2, S3, and S4 are summarised in Table 6.2.



(a) principal stress, notch at the free edge side



(b) Mises stress, notch at the free edge side

Figure 6.14: Effective notch stress at the planes with angles 35° and 39° to the Y-axis direction, S2 under $F = 1$ kN.

6.5. FATIGUE CRACK CHARACTERISTICS

6.5.1. STRAIN GAUGE DETECTION

Figure 6.15 plots the strain ranges at the lower corners (the critical locations with an angle of 38°) of the stiffeners measured by the strain gauges 4 mm away from the weld toe at the stiffener side. The strain ranges keep constant in the early stage. With the initiation and development of the cracks, the strain ranges start to fall. S2 S is a run-out in which the strain range (almost) keeps constant until the end of loading, see Figure 6.15b.

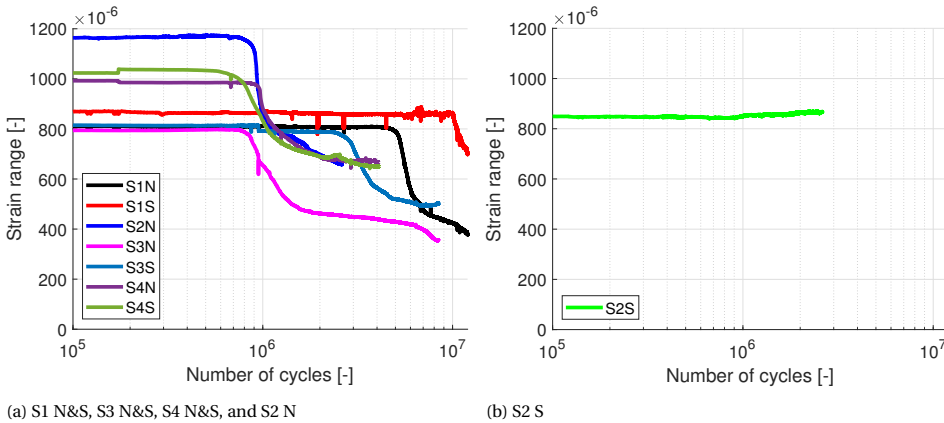


Figure 6.15: Strain range versus load cycles at the lower corners of the stiffeners with a distance of 4 mm from the weld toe at the stiffener side.

6.5.2. NAKED EYE INSPECTION

Surface cracks are first inspected by the naked eye. Afterwards, a digital microscopic camera is used to identify the crack tip locations. Multiple cracks are observed which gradually merge into large cracks during the propagation. Surface crack locations are marked and measured using the clipper during the experiments. Figure 6.16 shows the cracks in S1 after the experiment. Fatigue cracks are first inspected at the free edge side and then at the in span side. At each location, the cracks are first detected at the weld surfaces at the lower corners, afterwards propagate towards two sides at the weld surfaces, and finally grow to the crossbeam.

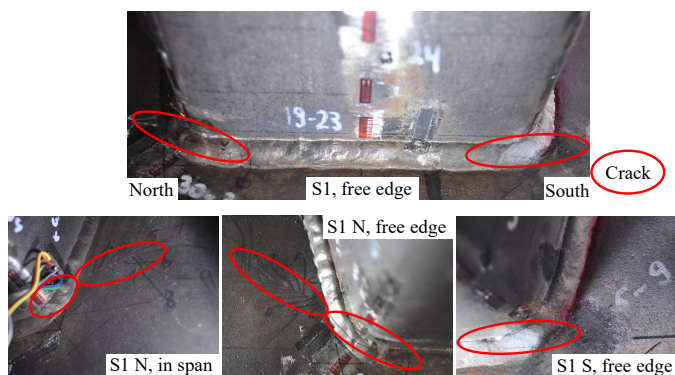


Figure 6.16: Fatigue crack inspection for S1 after the experiment.

Figure 6.17 illustrates the fatigue crack propagation process of type C4d crack. In total four stages are designated: The cracks initiate from the weld root and propagate into the weld throat (Stage 1 in green). Short cracks propagate on the weld surface and merge into long cracks (Stage 2 in purple). Afterwards, the main crack grows to the crossbeam and propagates as a surface crack (Stage 3 in yellow). Finally, the crack grows through the crossbeam thickness and propagates as a through-thickness crack (Stage 4 in red).

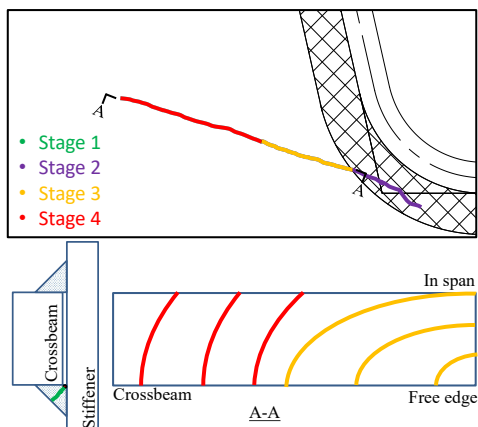
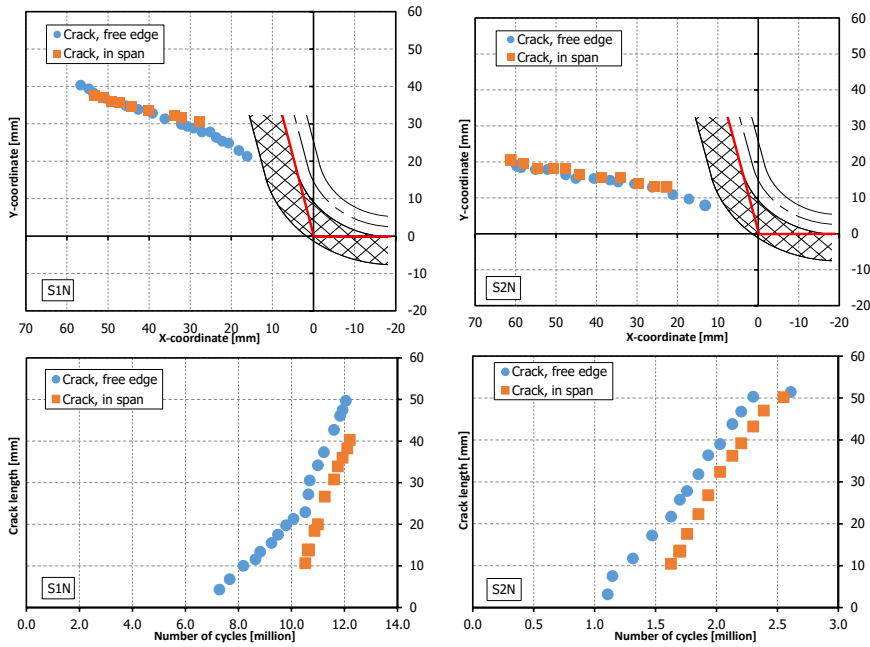


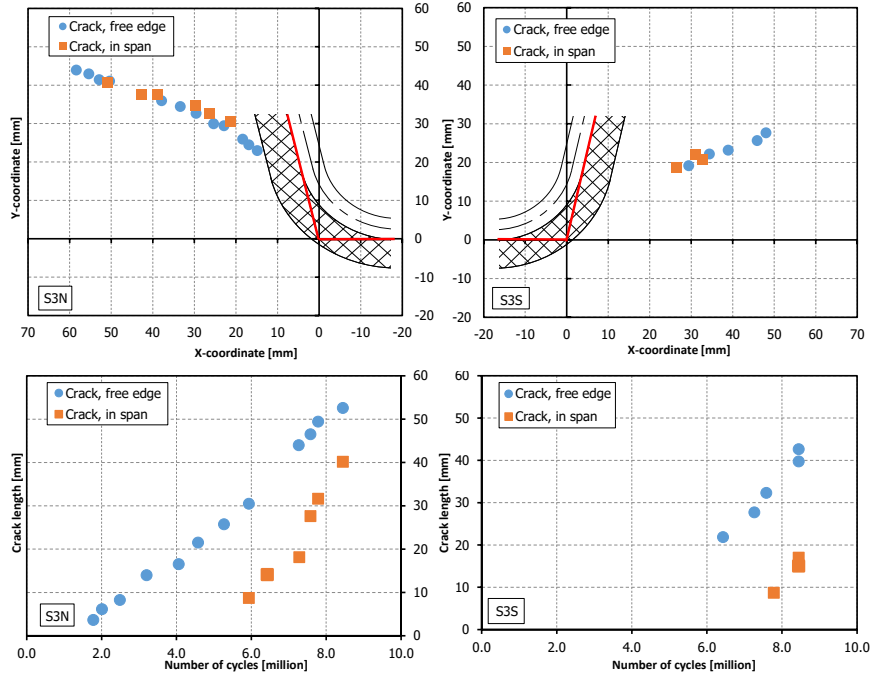
Figure 6.17: Illustration of the fatigue crack propagation process of type C4d crack.

Figure 6.18 shows the patterns of the crack propagation and the crack length versus number of cycles for S1, S2, S3, and S4. The cracks touch the crossbeam at the corners, and propagate towards the upper-outside direction. At the free edge and in span sides the propagation paths match well, except for S4 N. Linear crack propagations with relatively slow rates are observed in all studied cracks.



(a) S1 N

(b) S2 N



(c) S3

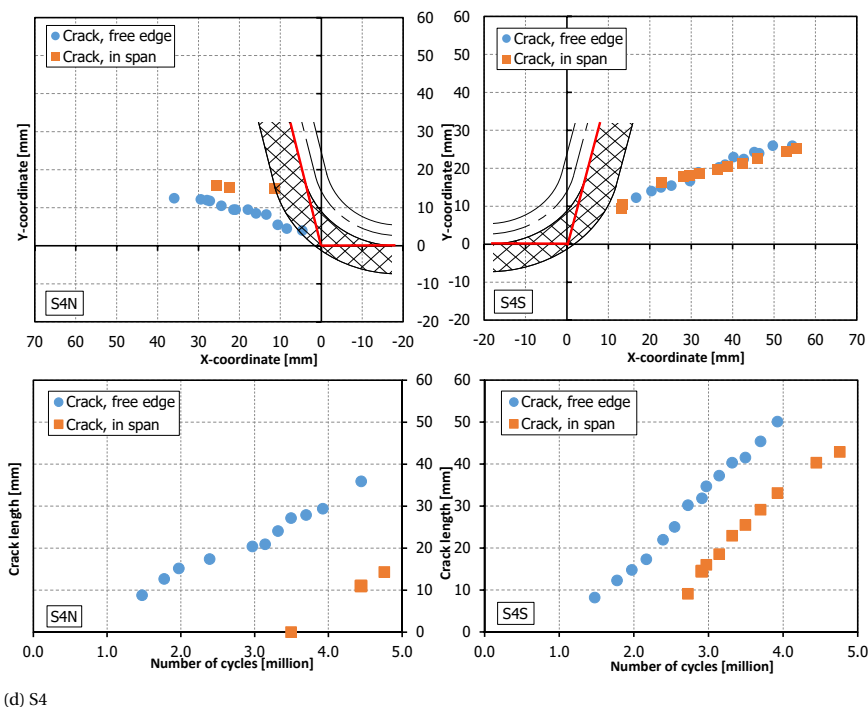
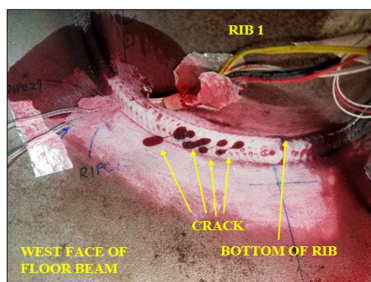
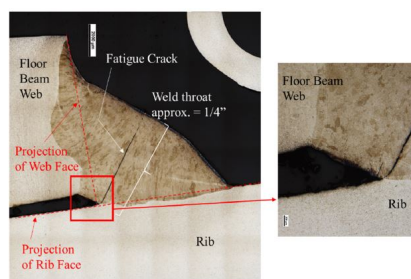


Figure 6.18: Patterns of the crack propagation and the crack length versus number of cycles for S1, S2, S3, and S4.



(a) weld root crack (Figure 17 in [209])



(b) optical observation (Figure 18 in [209]).

Figure 6.19: Report of the weld root crack in a U shape rib-to-crossbeam (or crossbeam-to-stiffener as designated in this study) detail by Lehigh University in 2019, adapted from [209].

To the best of the author's knowledge, this type of weld root cracking has been rarely reported for crossbeam-to-stiffener welded connection without cut-out at the time of writing. A similar type of crack was reported by Lehigh University in 2019 (almost the same time when the current experiment was carried out) in U-shape automatically welded details in a full-scale specimen after the in-plane cyclic loading [209], see Figure 6.19. Surface cracks were found at the weld throat, indicating a crack initiating from the weld

root and propagating through the weld throat. The hypothesis is verified by the optical observation for the cut part from the tested specimen as shown in Figure 6.19b. Note that in [209], this type of root crack was observed when the weld toe (at the stiffener side) experiences both tensile and compressive mechanical-induced stress. In the author's experiments, only compressive mechanical-induced stress occurs.

6.6. FATIGUE LIFE EVALUATION

Either early detection in the laboratory or practical on-site inspection can be used as the fatigue life criterion for OBDs. The former may be determined by a certain percentage change of the strain range close to the weld or by the naked eye inspection at a close distance. However, short cracks are not easy to inspect on-site in practice. Therefore, a surface crack with a predefined length is commonly adopted as the practical evaluation criterion. Five failure criteria are used for detail C4d in this section, as described in the following:

- N_1 : 10% change of the strain range at location 4 mm away from the weld toe at the stiffener side.
- N_2 : 25% change of the strain range at location 4 mm away from the weld toe at the stiffener side.
- N_3 : First Visible Crack (FVC) inspected by the naked eye.
- N_4 : Approximately 25 mm long surface crack on the crossbeam.
- N_5 : Approximately 50 mm long surface crack on the crossbeam.

Table 6.2 summarises the experimental measurements and the FE calculations for detail C4d. The nominal weld stress ($\Delta\sigma_{w,n}^a$ or $\Delta\sigma_{w,n}$), structural weld stress ($\Delta\sigma_{w,hs}$), and effective notch stress ($\Delta\sigma_{w,en}$) are obtained based on the FE calculations. In all cases, a constant amplitude fatigue limit, N_D , is defined at 10×10^6 cycles. The corresponding number of cycles is shown in bold when it is within the constant amplitude fatigue limit (failure A), $N \leq 10^7$. It is defined as failure B when $N > 10^7$. Note that $\Delta\sigma_{w,n}^a$ is calculated by the measured weld throat thickness, whereas the rest are based on the predefined 5 mm weld throat.

Table 6.2: Summary of the experimental measurements and the FE calculations for detail C4d.

Location	ΔF kN	$\Delta\sigma_{w,n}^a$ MPa	$\Delta\sigma_{w,n}$ MPa	$\Delta\sigma_{w,hs}$ MPa	$\Delta\sigma_{w,en}$ MPa	N_1	N_2	N_3	N_4	N_5	N_t
S1 N	160	68	72	114	253	5367636	5786237	6410000	10525000	11003000	12056000
S1 S	160	73	75	118	273	10777000	×	11100000	×	×	12056000
S2 N	220	68	83	138	317	926254	999428	938000	1758564	2610244	2617247
S2 S	220	79	83	138	317	×	×	×	×	×	2617247
S3 N	196	73	74	123	283	924350	1131383	846000	5269540	7784751	8446804
S3 S	196	68	74	123	283	2959338	3535631	6430000	7581532	×	8446804
S4 N	220	95	83	138	317	992909	1455782	971000	3698135	×	4758840
S4 S	220	91	83	138	317	871702	1141907	857000	2546937	3698135	4758840

Note:

- a : $\Delta\sigma_{w,n}^a$ is calculated using measured weld throat thickness.
- N_t : the number of total cycles until the terminal of experiment.
- ×: run-out.

6.7. FATIGUE RESISTANCE OF DETAIL C4D

6.7.1. FATIGUE EVALUATION USING VARIOUS FAILURE CRITERIA AND ASSESSMENT METHODS

Statistical analysis is carried out following the procedure described in [177, 178] with a fixed slope parameter $m = 3.00$ (fully informative prior), in line with (pr)EN 1993-1-9 [16, 17]. Figure 6.20 shows the statistical analysis for the results in Table 6.2. $\Delta\sigma_{50\%}$ is defined as the stress range at 2 million cycles with a 50% exceedance fraction. Various assessment methods ($\Delta\sigma_{w,n}^a$, $\Delta\sigma_{w,n}$, $\Delta\sigma_{w,hs}$, and $\Delta\sigma_{w,en}$) are used with five failure criteria for each.

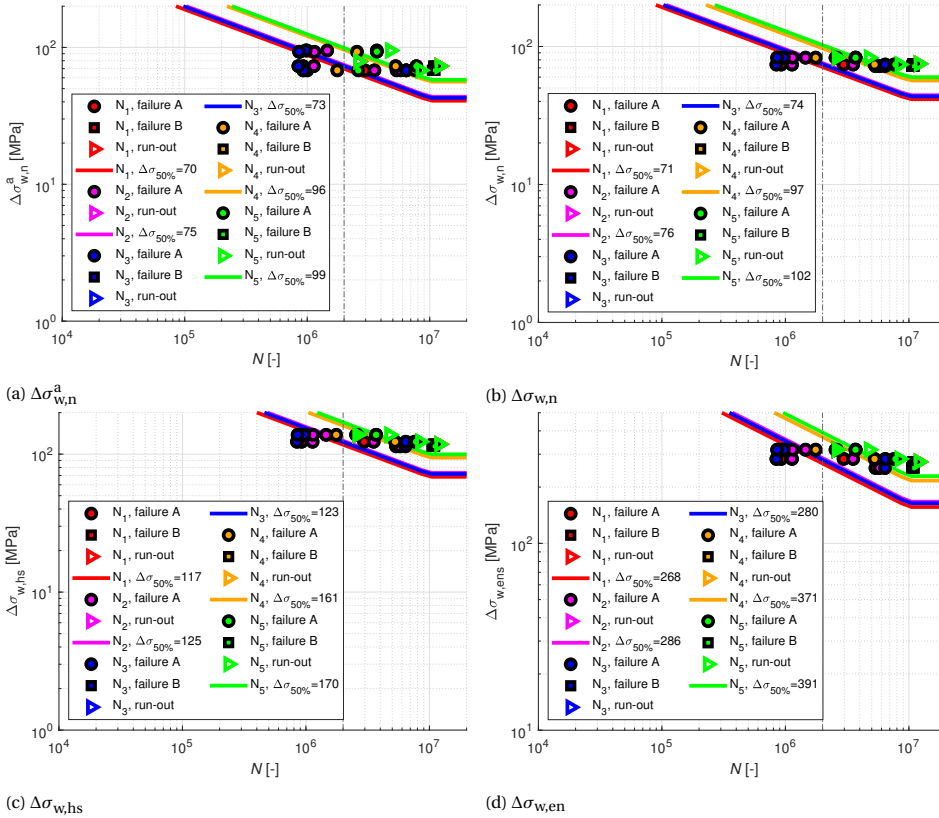


Figure 6.20: Evaluation of $\Delta\sigma_{50\%}$ using $\Delta\sigma_{w,n}^a$, $\Delta\sigma_{w,n}$, $\Delta\sigma_{w,hs}$, and $\Delta\sigma_{w,en}$ with the five failure criteria for each ($m = 3.00$).

In the studied two types of the failure criteria, the early detection criteria (N_1 , N_2 , and N_3) and the on-site detectable criteria (N_4 , and N_5), close values are found for $\Delta\sigma_{50\%}$ in each type. The latter is approximately 1.4 of the former. This is in line with the relatively slow crack propagation rates observed in Figure 6.18. The difference of $\Delta\sigma_{50\%}$ between

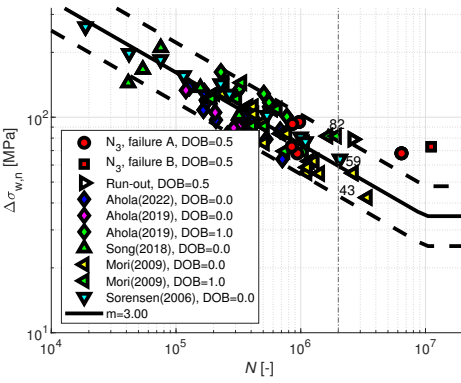
cases $\Delta\sigma_{w,n}^a$ and $\Delta\sigma_{w,n}$ is within 1 MPa, except for N_5 where the difference is 3 MPa. When $\Delta\sigma_{w,hs}$ is used, $\Delta\sigma_{50\%}$ increases from 73 MPa to 123 MPa, and from 99 MPa to 170 MPa with the failure criteria of N_3 and N_5 , respectively. The values further increase to 280 MPa, and 391 MPa when using $\Delta\sigma_{w,en}$ instead of $\Delta\sigma_{w,n}^a$.

6.7.2. STATISTICAL ANALYSIS INCLUDING THE RESULTS IN THE LITERATURE

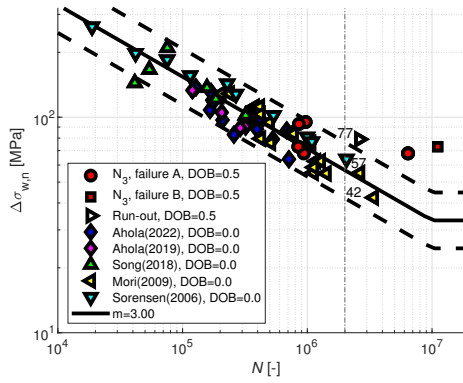
Load carrying fillet or partial penetration welded plates were studied under either axial force (DOB = 0.0) or bending moment (DOB = 1.0) in [49, 210–213]. Experimental investigations were carried out for either mild strength steel, *e.g.* S355, or (ultra) high strength steel, *e.g.* S960. The root failures in [49, 210–213] are selected and grouped into the statistical analysis in this section.

Figure 6.21 shows the statistical analysis using the joint dataset, including the author's own experiments and the results found in the literature, for $\Delta\sigma_{w,n}^a$ with the failure criterion of N_3 . Both $\Delta\sigma_{50\%}$ and standard deviation (s) are very close between the case with DOB = 0.0, 0.5 or 1.0 (Figures 6.21a), and the case with DOB = 0.0 or 0.5 (Figures 6.21b). The characteristic fatigue resistances (95% exceedance fraction at 2 million cycles), $\Delta\sigma_C$, are 43 MPa for DOB = 0.0, 0.5 or 1.0, and 42 MPa for DOB = 0.0 or 0.5, respectively.

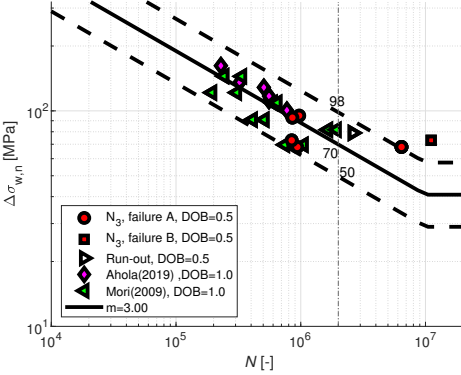
Figure 6.21c shows the results with DOB = 0.5 or 1.0. Higher fatigue resistance is observed, $\Delta\sigma_C = 50$ MPa, whereas the standard deviation ($s = 0.25$) keeps almost the same as the values in Figures 6.21a and b.



(a) DOB = 0.0, 0.5 or 1.0, $s = 0.24$



(b) DOB = 0.0 or 0.5, $s = 0.23$



(c) DOB = 0.5 or 1.0, $s = 0.25$

Figure 6.21: Statistical analysis for for detail C4d using the author's own experiments ($\Delta\sigma_{w,n}^a$ and N_3) and the results in the literature [49, 210–213].

6.8. SUMMARY OF THE CHAPTER

Fatigue behaviour of crossbeam-to-stiffener welded connection without cut-out is studied in Chapter 6 using one full-scale OBD specimen. Fillet welds without penetration are used to connect the crossbeams and the trapezoidal stiffeners. Cyclic loading is applied on four trapezoidal stiffeners (two details in each) over the centre of stiffener webs with a distance of 600 mm from the middle of one outer crossbeam thickness on the free edge side. Root cracks are systematically studied in the crossbeam-to-stiffener welded connection without cut-out. The cracks are first observed at the weld surface at the lower corners and then propagate to the crossbeam. This detail is designated as C4d in TS [27]. The main conclusions based on this study are:

1. The lower corners of trapezoidal stiffeners have stress concentrations where cracks are first observed at the middle of corner curves (38° to the vertical). The crack propagation process of detail C4d consists of four stages: Fatigue cracks initiate from the weld root and propagate in the weld throat (Stage 1). The cracks grow at the weld surface with short cracks merging into long cracks (Stage 2). The main crack grows to the crossbeam and propagate as a surface crack (Stage 3). A through-thickness crack is formed which keeps propagating in the crossbeam (Stage 4).
2. The fatigue propagation rate on the crossbeam is relatively slow and shows linear rates until the end of the experiments (with a length of approximately 50 mm surface crack on the crossbeam). The nominal weld stress, structural weld stress, and effective notch stress are used for fatigue life assessment. The stress ranges at 2 million cycles with a 50% exceedance fraction and the failure criterion of a length of approximately 50 mm surface crack on crossbeam are around 1.4 times the values with the failure criterion of visual control (the first visible crack inspected by the naked eye) for the three assessment methods.
3. The Degree Of Bending (DOB) affects the fatigue performance of root cracking in load carrying welded connections. The statistical analysis using the author's experiments and the results in the literature shows that higher fatigue resistance is obtained when $DOB \geq 0.5$. Detail categories for the nominal weld stress (designated in TS) proposed based on this study are:
 - **50 MPa** for fatigue design considering possible large DOBs (≥ 0.5) and the relatively slow crack propagation on crossbeam.
 - **40 MPa** for conservative evaluation in case that neither DOB is available nor surface cracks are allowed.

7

WELD TOE CRACK IN STIFFENER AT THE LOWER END OF CROSSBEAM-TO-STIFFENER WELDED CONNECTION WITH CUT-OUT (C3A)

Fatigue behaviour of the weld toe crack at the lower end of crossbeam-to-stiffener connection with cut-out is studied in Chapter 7. The detail category for the hot spot stress is established based on eight experiments reported in the current study and the results found in the literature. Two fatigue resistances are proposed for new bridge decks and existing bridge decks, respectively. The effect of load position on the hot spot stress is analysed. In addition, the effective notch stress is used to evaluate the fatigue resistance considering different weld profiles. The welding residual stress is simulated by the finite element method. The regions affected by high temperature in welding are identified in the model. The residual stress is implemented into the mechanical model in which cyclic elastic-plastic material properties are assigned to the welded connection. The redistribution of residual stress under cyclic loading is studied and the crack initiation is predicted using the notch strain method. Further, the cyclic J-integral is calculated for the short crack propagation using the elastic-plastic fracture mechanics with the redistributed residual stress implemented in the model. The linear elastic fracture mechanics is adopted for the long crack propagation using the extended finite element method with “in-house” Python code. Finally, a 2⁺ stage model is proposed for the full stage fatigue analysis starting from the crack initiation stage to the long crack propagation stage.

7.1. INTRODUCTION

Fatigue crack at the lower end of the weld between stiffener web and crossbeam with cut-out is studied in Chapter 7. The crack initiates from the weld toe at the stiffener side, and propagates outside of the vicinity of the weld (designated as type C3a), see Figure 7.1. Stress ranges ($\Delta\sigma$) are introduced due to the local distortion of the stiffener below detail C3a in combination with the in-plane and out-of-plane deformation of crossbeam, causing stress concentrations at the weld toe. This type of crack has been observed in bridges in Europe [1, 138], Japan [91], the United States [214], and China [88]. Two types of cut-out shapes, Haibach-type or regular-type, are commonly used in Europe as illustrated in Figure 7.2.

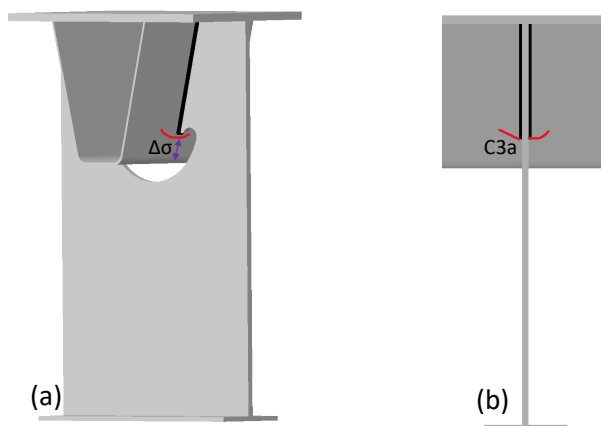


Figure 7.1: Illustration of the weld toe crack at the end of a crossbeam-to-stiffener connection with cut-out.

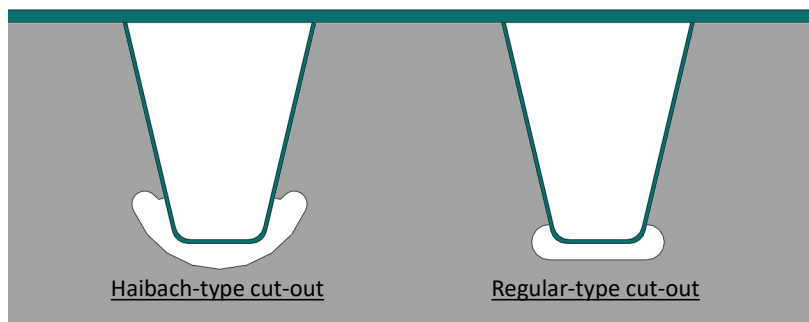


Figure 7.2: Sketch of the cut-out shapes commonly used in Europe.

The experimental investigation, hot spot stress, effective notch stress, statistical analysis, and effect of load position parts of this chapter are based on the journal paper: *Fatigue behaviour of toe crack at the lower end of crossbeam-to-stiffener weld with cut-outs in OBDs*, *Journal of Constructional Steel Research*, **222**, (2024). Section 7.8 is modified based on the master's thesis: *Fracture Mechanics, Application on Orthotropic Steel Decks*, master's thesis of Delft University of Technology, (2020) with the permission of the author D.H. Malschaert.

Detail C3a has been studied by researchers using experiments and FE simulations [1, 88, 100, 148, 149, 154, 158]. A relatively slow crack propagation rate for this detail is attributed to the crack growing away from the crossbeam weld with its stress-raising effect. The failure criterion, *i.e.* the crack size, at which the fatigue resistance is determined becomes relevant. Some of the mentioned studies used a certain surface crack length as a criterion. Others used the first time a crack was observed by the naked eye (“first visible crack”), which is accompanied by some scatter related to detectable crack length and inspection interval. As an early indicator of cracking, many authors considered a certain deviation in strain range, measured by strain gauges, concerning the initial strain range of the intact specimen. However, a well-defined and widely accepted failure definition is currently missing. The detail category was not included in either the first or the second generation of (pr)EN 1993-1-9 [16, 17].

This chapter experimentally studies the fatigue behaviour of detail C3a at eight locations in one full-scale specimen. The strain distribution is obtained by strain gauges, and surface cracks are tracked during the experiments. Subsequently, a FE model is described, which is validated against the measured strains. The hot spot stress is calculated by the FE model. Afterwards, the results obtained in the current experiments and selected from the literature are grouped to derive a fatigue resistance using the hot spot stress method. An additional S-N curve is proposed for existing bridge decks. The effect of load position is numerically investigated for the hot spot stress of detail C3a. The effective notch stress method is adopted to study the effect of weld profile.

Welding simulation is conducted for the studied detail using the commercial software Abaqus 2019 [179]. The Welding-induced Residual Stress (WRS) after cooling down in the as-welded condition and the mechanical behaviour under cyclic loading at the critical location are analysed. The cyclic constitutive models for the heat affected zone and the base metal are used with the region identified by the welding simulation in the cyclic loading. The redistribution of the WRS is numerically studied. The strain ranges at the critical locations are used for the crack initiation prediction using the notch strain method. The elastic-plastic fracture mechanics is adopted to study the states of short cracks using the cyclic J-integral, ΔJ , with the redistributed WRS implemented in the FE models. Finally, the eXtended Finite Element Method (XFEM) (available in Abaqus 2019 [179]) with “in-house” Python code is used to predict the long crack propagation using the linear elastic fracture mechanics. A 2⁺ stage model is proposed based on the methods applied to the studied detail for the full stage fatigue analysis of welded connection starting from the crack initiation stage to the long crack propagation stage with WRS properly taken into consideration.

7.2. EXPERIMENTAL INVESTIGATION AND FE MODEL VALIDATION

7.2.1. SETUP AND LOADING PROGRAMME

The specimen that has been introduced in Chapters 5 and 6 is used to study the fatigue behaviour of detail C3a, see Sections 5.2.1 and 6.2 for more information. The Crossbeam III region and the south side of the Crossbeam I region have been loaded in fatigue to study the fatigue behaviours of detail C1c and detail C4d, respectively. This chapter uses the unloaded part (on the north side) of the (Crossbeam I) region to examine the fatigue behaviours of eight C3a type details.

Two series of load cases, LC A and LC B, are applied in the experiments, see Figure 7.3. In LC A, the cyclic load is applied on the free edge side of Crossbeam I with the loading patch centred above the stiffeners. In LC B, cyclic loads F1 and F2 are simultaneously applied on the free edge and in span sides, with the load centred above a stiffener web. The loading area is $270 \times 320 \text{ mm}^2$, corresponding to the tyre patch of Axle Type C in EN 1991-2 [80] with two layers of 8 mm thick rubber plates between the top deck and the steel load jack at each load application position. Naked eye inspections are carried out to detect surface cracks during the experiments. Table 7.1 gives a summary of the loading programme. LC A is first applied on stiffeners 7 (S7) and 8 (S8) with no cracks observed. Thereafter, LC B is applied, resulting in fatigue cracks at the detail of study. Note that S8 N has been loaded first in LC A, and then two strengthening plates (approximately 50 mm thick steel plates) are placed on the deck plate at the load application positions with a size of approximately $500 \times 500 \text{ mm}^2$ before LC B is applied. This strengthening is applied due to the observed fatigue cracks in the stiffener-to-deck plate welded connections after S7 N&S are loaded in LC A. As the stress levels of the stiffener-to-deck plate welded connection in LC B are significantly smaller than LC A, strengthening is not needed in LC B. Therefore, they are not used for S5 and S6. The following part of the chapter focuses on the description of LC B. To obtain a conservative characteristic resistance, $\Delta\sigma_{C,hs}$, the cycles loaded in LC A are not counted in the current study.

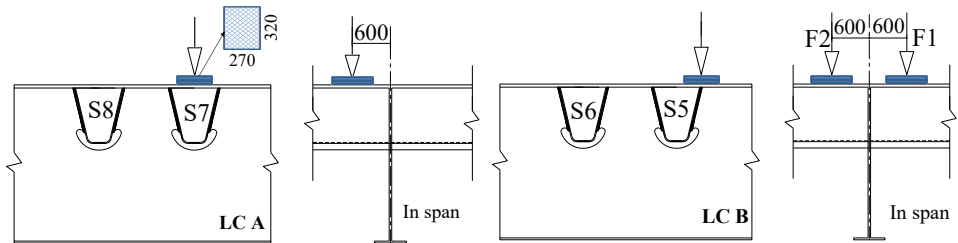


Figure 7.3: Sketch of the out-of-plane loading positions for the crossbeam-to-stiffener welded connections with Haibach-type cut-outs (unit: mm).

Table 7.1: Summary of the loading programme.

	Loaded detail	7 N&S	7 N&S	7 N&S*	8 N&S*	
LC A	Load range [kN]	1 × 275	1 × 360	1 × 360	1 × 320	
	Frequency [Hz]	2	2	2	2	
	Load ratio	0.1	0.1	0.1	0.1	
	Cycles [million]	7.40	0.67	1.73	9.91	
	Run-out	×	×	×	×	
	Loaded detail	8 N*	5 S	6 N	6 S	5 N
LC B	Load range [kN]	2 × 170	2 × 250	2 × 250	2 × 150	2 × 150
	Frequency [Hz]	2	2	2	2	2
	Load ratio	0.1	0.1	0.1	0.1	0.1
	Cycles [million]	0.33	0.31	0.55	1.67	10.00
	Run-out					×

Note:

“*”: the case in which strengthening plates are used.

“×”: run-out.

7.2.2. STRAIN MEASUREMENT

Figure 7.4 gives an overview of the strain gauges attached to the stiffeners of S5, S6, S7, and S8. The strains in the horizontal directions and the direction perpendicular to the weld toe at the lower end are referred to as “Stiffener H1 (free edge)”, “Stiffener H2 (in span)”, and “Stiffener V”, respectively. In each of these directions, a chain of 5 strain gauges with an intermediate distance of 2 mm is attached to cover the local stress field, with the first strain gauge intended to be positioned 4 mm from the weld toe. Stress gradients are obtained by the chain strain gauges, which can be used to calculate hot spot stress using the measured values [23] or validate the calculation of FE model. In addition, two separate gauges intended at 25 mm and 50 mm distances from the weld toe are attached to cover the global strain field.

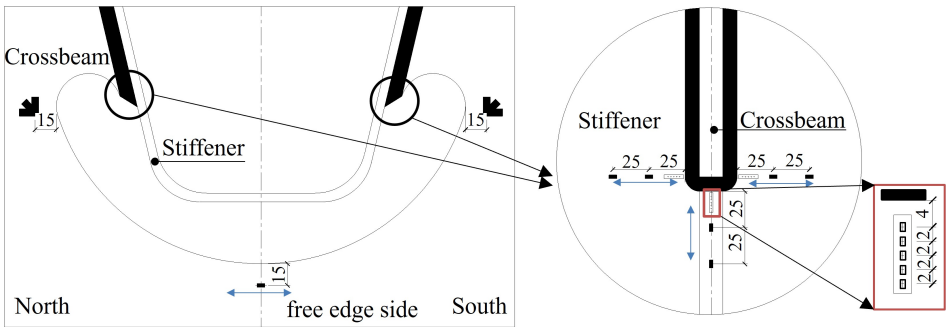
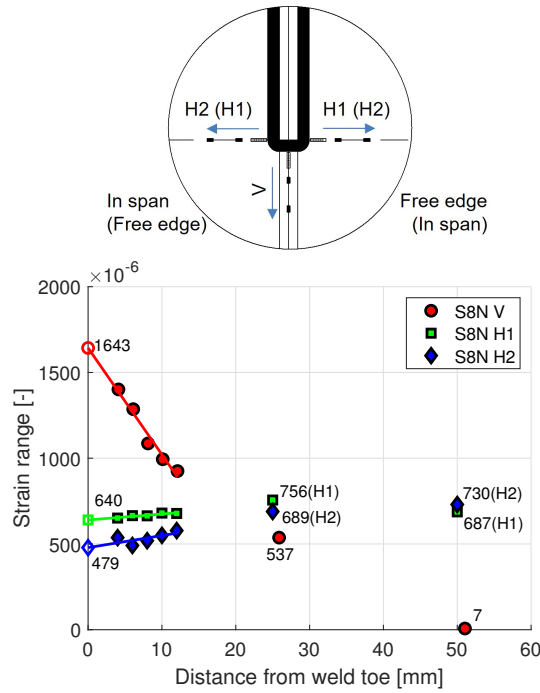
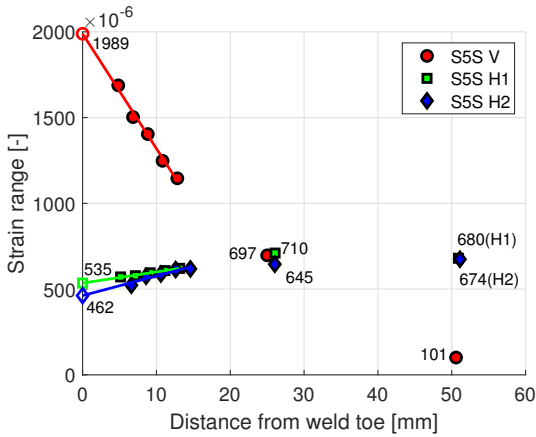


Figure 7.4: Overview of the strain gauge positions at S5, S6, S7, and S8 (unit: mm).

Figure 7.5 presents the measured strain ranges close to the weld toe in a stable cycle of loading after the settling of the specimen. The positions of the strain gauges are measured after the experiments and used in Figure 7.5. One exception is S8 N, where the strain gauges are removed before executing measurements, the intended distances are therefore used for S8 N. The strains in Figure 7.5 are positive in V direction and negative in H1 and H2 directions at the applied loads, although strain ranges are always positive. The highest strain ranges are observed at the lower end (V direction). The strain values in H1 and H2 directions are smaller than those in V direction. The ranges at the free edge side (H1) are slightly larger than those at the in span (H2) side. High strain ranges and concentrations are observed in V direction. The linear extrapolation is carried out using the 5 strain measurements close to the weld toe. The extrapolated strain ranges in H1 and H2 directions are between 2 and 4 times lower than the values in V direction.



(a) S8 N: 2×170 kN



(b) S5 S: 2×250 kN

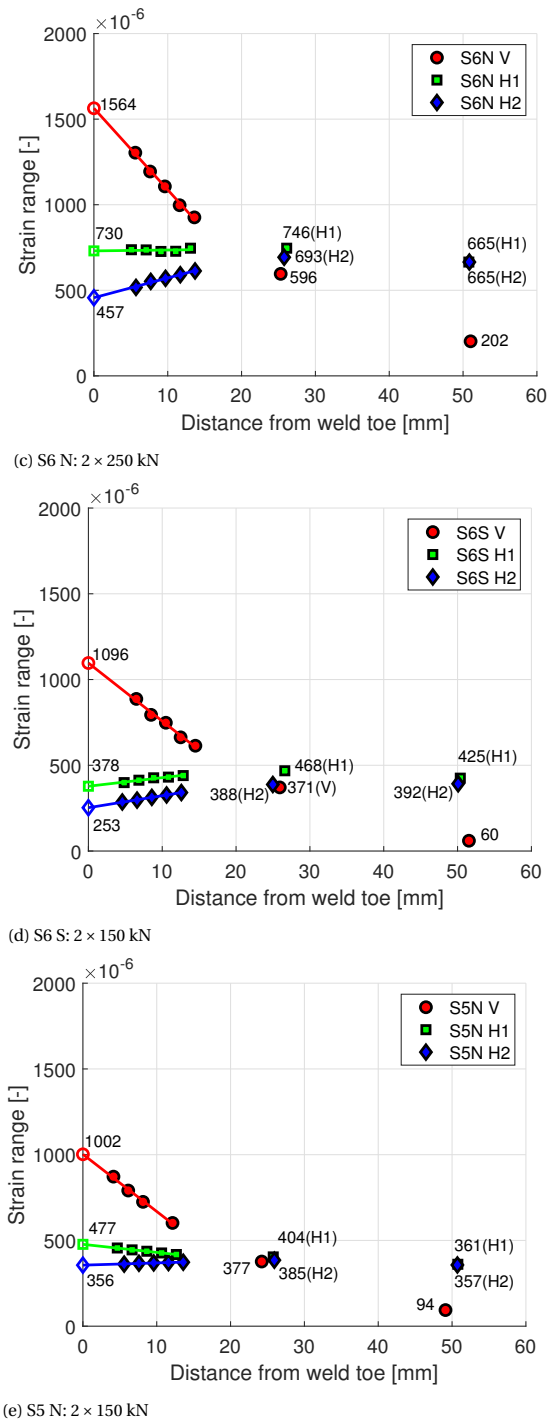


Figure 7.5: Strain ranges close to the weld toe, measured in LC B.

7.2.3. INTRODUCTION OF THE FE MODEL

Figure 7.6 shows the FE model constructed using Abaqus 2019 [179]. Displacement constraints are applied along at the bottom of the lower flanges along the middle of cross-beam thickness in three dimensions ($UX = UY = UZ = 0$). A uniform distributed pressure is applied to the load patches. Linear elastic material is applied in the model with the elastic modulus of 210 GPa and the Poisson's ratio of 0.3. Figure 7.7a shows the FE model of the specimen, where the global distinction between the use of brick and shell elements is made. The FE mesh consisting of brick elements at the lower end of crossbeam-to-stiffener welded connection is shown in Figure 7.7b and Figure 7.7c. 8-node linear brick elements with reduced integration (C3D8R) and 8-node quadratic shell elements with reduced integration (S8R) are used at and near Crossbeam I, and Crossbeams II and III, respectively.

A fillet weld with an effective weld throat thickness of 5.00 mm is modelled. A "seam", see Figure 7.7c, that duplicates the overlapping nodes along the selected region [179] is applied at the interface between the crossbeam web and the stiffener. The mesh close to the studied details is refined with an element size of approximately $2 \times 2 \times 2 \text{ mm}^3$. The element size gradually increases to approximately $50 \times 50 \times 50 \text{ mm}^3$ and $100 \times 100 \times 100 \text{ mm}^3$ for the parts meshed with element types C3D8R and S8R, respectively. "Tie" constraints [179] connect the brick and shell element parts, see the position of the red rectangle in Figure 7.6 and the corresponding interface in Figure 7.7a. The ties are applied in all directions. The nodes of a "tie" constrained interface have the same displacements and rotations [179].

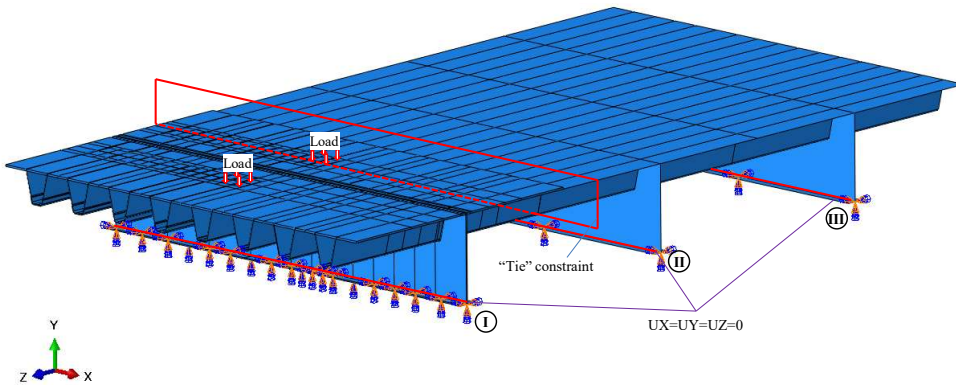


Figure 7.6: Load and boundary conditions of the FE model.

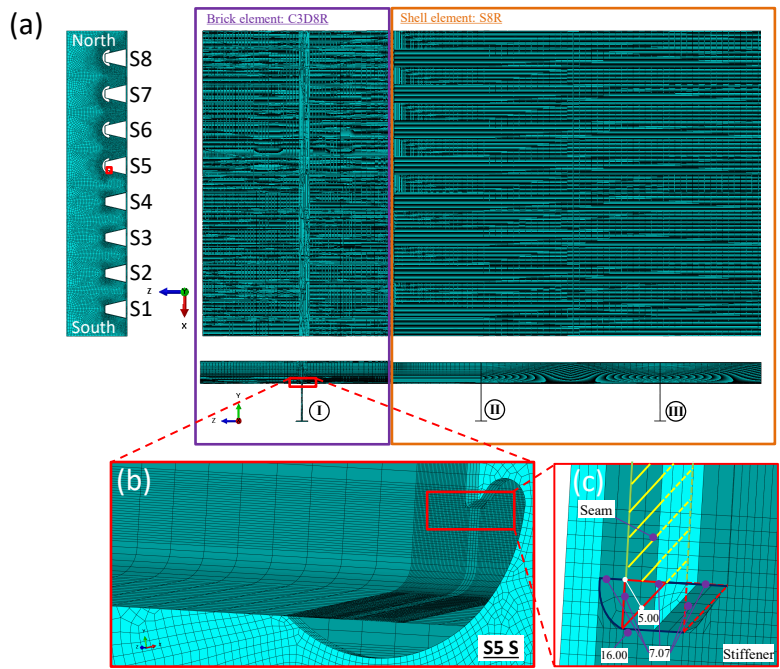


Figure 7.7: FE model of the specimen and the lower end of the crossbeam-to-stiffener welded connection in S5 S (unit: mm).

7.2.4. VALIDATION OF THE FE CALCULATION

The strain ranges measured from the experiments (EXP) and calculated by the finite element (FEA) are normalised by a load level of 2×1 kN. The results are presented in Figure 7.8. A good agreement is found between the FEA and the EXP in LC B. The stresses calculated by the FEA are used in Section 7.3 for fatigue resistance analysis.

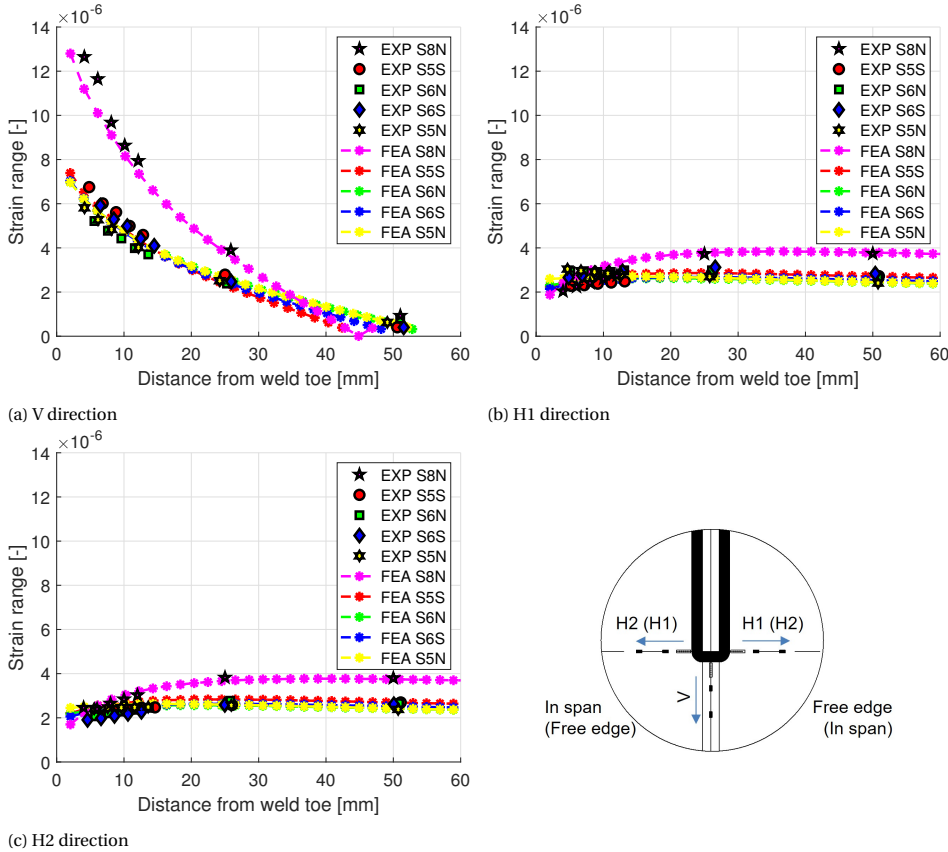


Figure 7.8: Comparison of the EXP and the FEA results in LC B under the load range of 2×1 kN.

7.2.5. FATIGUE CRACK INSPECTION

Stable strain range distributions are observed under the constant amplitude loading in the initial stage. After a certain number of cycles, deviations in the strain ranges are observed, caused by the local stiffness deviation due to the development of a fatigue crack. This deviation of strain range is first observed at the location 4 mm from the lower end of the weld toe, which is measured by the first strain gauge in the chain (G01) in V direction. Figure 7.9 shows the loading history of the strain ranges at “V G01” in four stiffeners. Note that the first two sharp drops in S5 S, at 2.1×10^4 cycles and at 2.8×10^4 cycles, which may be attributed to “the settling of the specimen/measurements”. The strain gauge is checked and works properly afterwards. The reference value of the strain

range deviation for S5 S is set after these two drops. S5 N has stable strain ranges until the end of loading. It indicates a run-out without fatigue cracking detected.

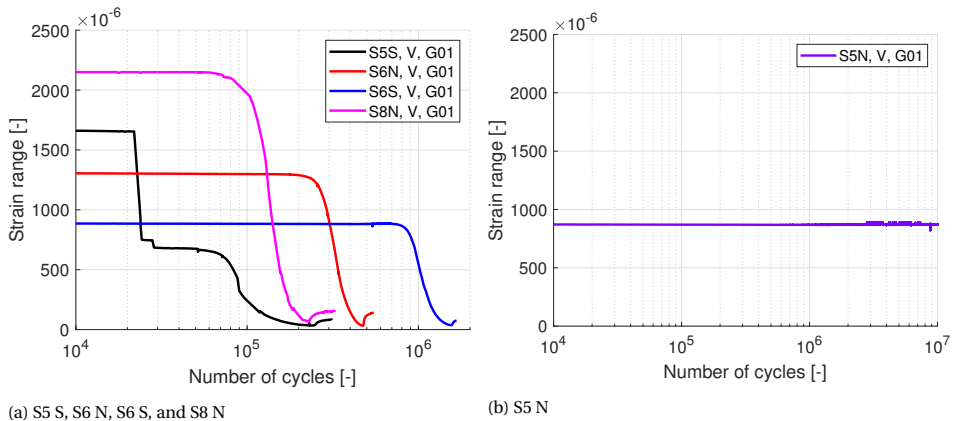


Figure 7.9: Strain range versus load cycles from the strain gauge “Stiffener V G01”.

Figure 7.10 shows a fatigue crack of S5 S after 0.31 million cycles of loading in LC B. The crack initiates at the lower end of the weld toe in the stiffener, and then propagates towards the free edge and in span sides of the stiffener.

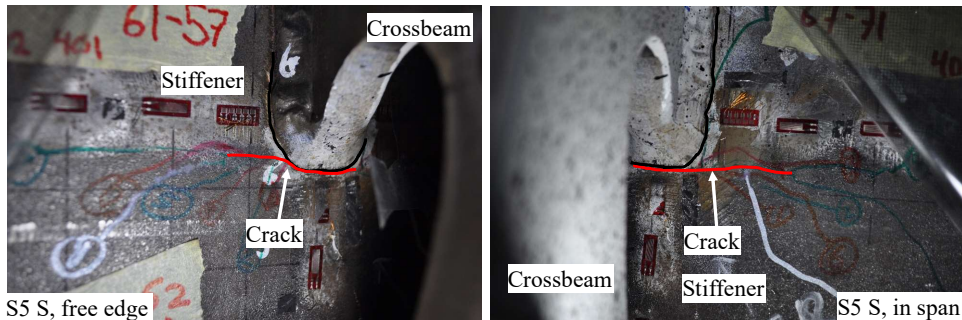


Figure 7.10: Fatigue crack of S5 S after 0.31 million cycles of loading in LC B, 2×250 kN load range.

Figure 7.11 gives an overview of the crack propagation of the four fatigue cracks observed. The cracks grow approximately linearly with the number of cycles for the crack lengths between the length at first detection of 7 to 18 mm and the end of the experiments with final crack lengths between 50 to 57 mm. The visible crack in S6 S is observed later and grows at a lower rate as compared with the other results. This agrees with the difference in the strain ranges in Figure 7.5. Figure 7.12 shows the marks of the crack edges as measured during the experiments using a clipper with naked eye inspections. The mark numbers in Figure 7.11 correspond to the sequential dots in Figure 7.12, except S8 N for which the longest of the two detected cracks is shown. The cracks are first

observed at the lower end of the weld toe, with some shift to the in span side for S6 N and S6 S.

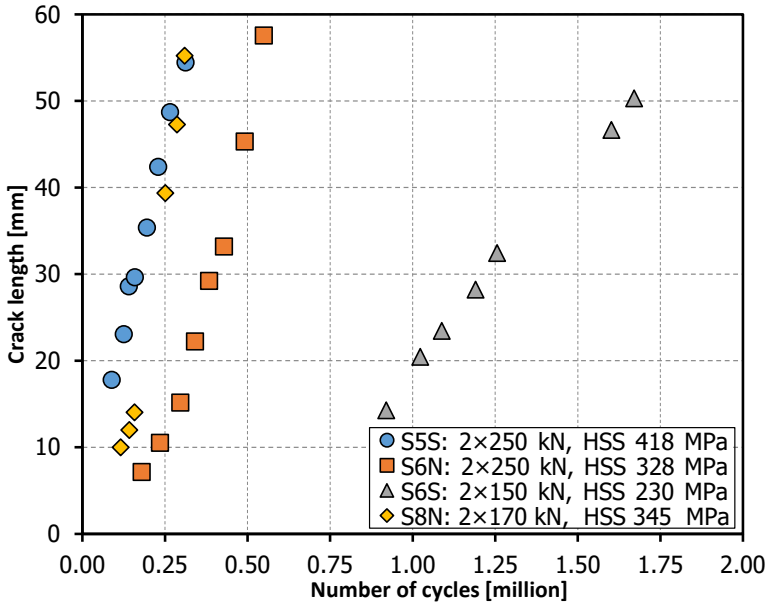


Figure 7.11: Crack length versus number of cycles.

Table 7.2 gives the numerical values of the load ranges, hot spot stresses, fatigue lives, and final crack lengths. In line with other fatigue studies of OBDs [1, 14, 215], the fatigue criteria used in the current study are designated as:

- N_1 : Number of cycles corresponding to 10% strain range deviation observed at 4 mm from the lower end of the weld toe.
- N_2 : Number of cycles corresponding to 25% strain range deviation observed at 4 mm from the lower end of the weld toe.
- N_3 : Number of cycles corresponding to the First Visible Crack (FVC) observed by the naked eye.
- N_4 : Number of cycles corresponding to an approximately 25 mm long surface crack.
- N_5 : Number of cycles corresponding to an approximately 50 mm long surface crack.

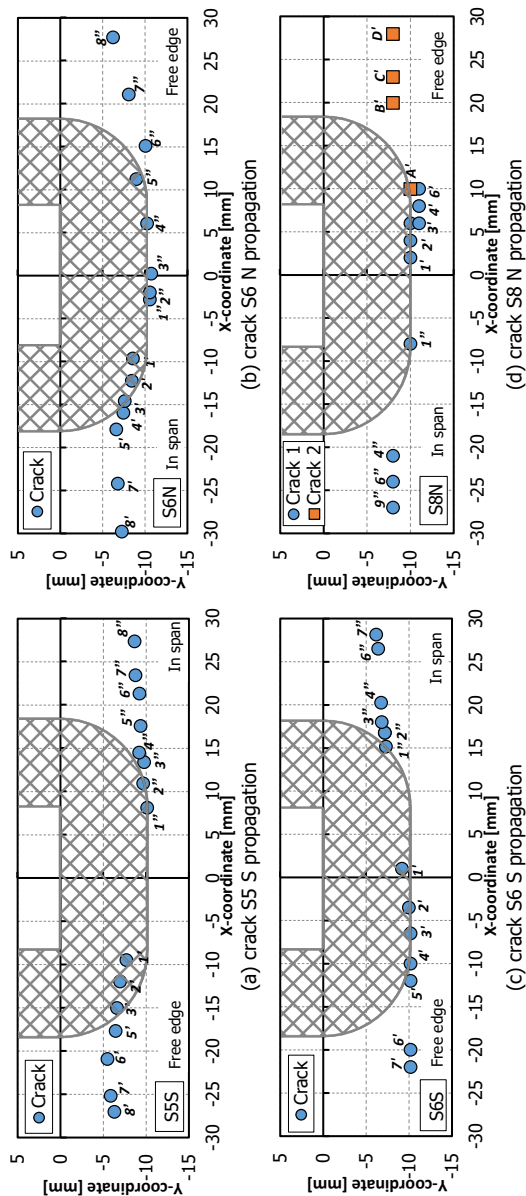


Figure 7.12: Patterns of crack propagation (grey hatched area represents the weld leg).

Table 7.2: Experimental results of the studied details using criteria N_i to define failure criteria of structural details in LC B.

Location	ΔF [kN]	$\Delta \sigma_{hs}$ [MPa]	N_1	N_2	N_3	N_4	N_5	Experiment terminated cycles	Final crack length [mm]
S5 S	2×250	466	71743	85076	88900	126000	266000	313569	55
S6 N	2×250	418	265822	294173	179918	341000	492000	548912	57
S6 S	2×150	254	885473	956493	920353	1190606	1670000	1670000	50
S5 N	2×150	248	$\times$	$\times$	$\times$	$\times$	$\times$	10000001	$\times$
S8 N	2×170	544	99362	107462	115739	251119	286810	327702	56

7.3. FATIGUE RESISTANCE

This section presents the evaluation of the fatigue resistances of detail C3a according to [27] using the author's own experiments and the results in the literature. The hot spot stress range used in the experiments from the literature is defined either by measurements or by validated FE models. It means that the results of the FE models used in the hot spot stress assessment agree well with the reported measurements. $\Delta\sigma_{hs}$ is calculated using the TS methodology [27]. The fatigue criteria N_1 to N_5 introduced in Section 7.2.5 are used as five possible criteria for fatigue evaluation.

Figure 7.13 provides the experimental results in S-N diagrams. The lines in the diagrams represent the exceedance fractions of 5%, 50%, 95% following the Basquin relation with a fixed slope $m = 3.00$, where the statistical procedure of Annex D of EN 1990 is applied [177, 178]. In all cases, a constant amplitude fatigue limit, N_D , is defined at 5×10^6 cycles. From Figure 7.13a to Figure 7.13f, one of the five criteria is used in each. Note that the criteria designation used in the references is adjusted to match the criteria given in Section 7.2.5. The year of the report, 1991, in [1] containing the experiments included in the current statistical analysis is used when citing the comprehensive reference. Moreover, not all criteria are reported for all the experiments in the literature. A loosened N_4 with a surface length of 30 mm is used for [148] in Figure 7.13f as N_5 is not available. The failure criteria of N_1 and N_2 lead to $\Delta\sigma_{C,hs}$ of 76 MPa and 94 MPa, respectively. The failure criterion of N_3 leads to $\Delta\sigma_{C,hs}$ of 117 MPa with a standard deviation (s) of 0.27. A clear bias is demonstrated between the experiments of Suganuma (2007) [154] and the other experiments. This is because the magnetic particle method, instead of visual control (the naked eye inspection), was used for crack detection in [154]. In the experiments of Huang (2019) [149], progressively increased load levels with an interval of 0.5 million cycles were applied and the visibility of crack in the initiation stage was not reported. The results in these two references are excluded from the statistical analysis. As the standard deviations of the experiments for the other criteria are smaller, the standard deviation of N_3 is attributed to differences in inspection regime. The bias between series for the other criteria is limited and the standard deviations of $\log(N)$ are in line with that of other welded details [169], despite the large differences in specimen geometry. This demonstrates that the hot spot stress is rather independent of geometry and therefore appropriate as a basis for the design of this detail. $\Delta\sigma_{C,hs}$ is rounded down to the nearest standardised value of 112 MPa.

The relatively large difference between the failure criteria of N_3 and N_5 demonstrates the relatively slow crack growth of the detail. The author considers the failure criterion of N_5 , with an approximate surface crack length of 50 mm, appropriate for the assessment of existing structures because it corresponds to a reasonable probability of the detection of surface crack with the naked eye [125]. The corresponding $\Delta\sigma_{C,hs}$ is rounded down to the nearest standardised value of 160 MPa. This relatively high resistance implies that most existing bridge decks will pass the assessment for this detail.

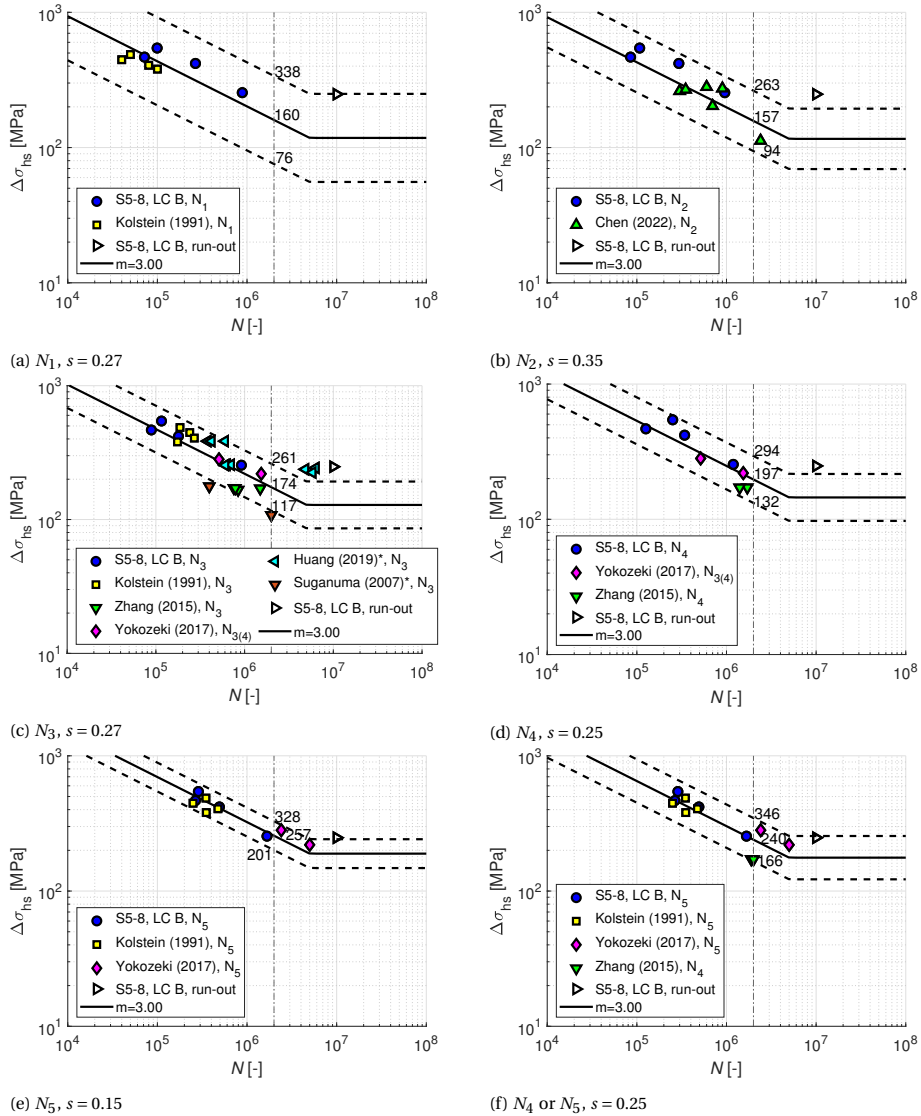


Figure 7.13: Statistical analysis for detail C3a using the authors' own experiments and the results in the literature [1, 100, 148, 149, 154, 158] (results with mark “*” are excluded from the statistical analysis).

7.4. INFLUENCE OF THE LOAD POSITION ON THE HOT SPOT STRESS

The hot spot stress in OBDs depends not only on the load level, but also on the position of loads (*i.e.* tyres of heavy vehicles) and on the support of the structure. The load application and specimen supports applied in all experiments deviate from actual OBDs, in order to simplify the experimental setups. This section examines the hot spot stress for realistic boundary conditions of crossbeams and load positions by the FE method.

In practice, crossbeams are connected to main girders, contrary to the fully supported crossbeams investigated in Section 7.2.

The simplified FE simulations are therefore carried out with the crossbeams simply supported at their edges. The component dimensions are equal to those of Section 7.2.3. The 20-node quadratic brick element with reduced integration (C3D20R) is also commonly used in Abaqus [179]. The element type C3D20R gives higher strains than the same number of elements using type C3D8R, see Appendix D.3. A more realistic fatigue verification with higher stress caused by the wheel loading is carried out in this section using C3D20R. The overall dimensions are adapted as shown in Figure 7.14. The red box indicates the region of applied wheel loading. The tyre contact surface is equal to that of a super single tyre in the Fatigue Load Model 4 of EN 1991-2 [80], *i.e.* 270 mm in the transverse direction and 320 mm in the running direction. The applied load is equal to the maximum wheel load applied to a super single tyre in the same fatigue load model, *i.e.* 45 kN. The hot spot stress of the critical location, S5 S at $Y = 0$, is extracted from Path 1, Path 2, and Path 3. The Degree Of Bending (DOB) [216] is calculated by Equation (7.1) where σ_b and σ_m are the bending stress and the membrane stress at the weld toe cross section, respectively.

$$\text{DOB} = |\sigma_b| / (|\sigma_b| + |\sigma_m|) \quad (7.1)$$

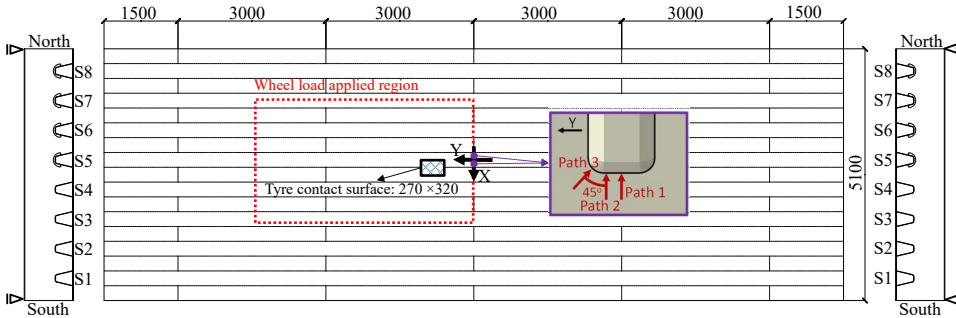


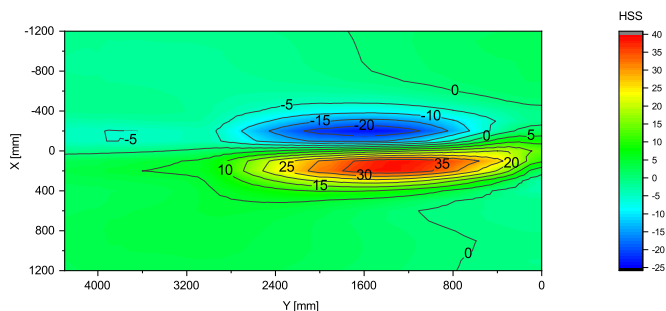
Figure 7.14: Overview of the FE model for the load position of S5 S connection (unit: mm).

Figure 7.15 shows the resulting Hot Spot Stress (HSS) and DOB at the previously mentioned three paths. Path 1 and Path 2 show similar results in HSS and DOB, whereas Path 3 gives smaller HSS. The largest absolute values of HSS are created for a load centred between approximately $Y = 800$ mm to 2000 mm, and $X = -300$ mm to -100 mm. The tensile or compressive stress occurs depending on which side of the web the load is applied, either S5 S or S5 N. Hence, the largest stress ranges occur if subsequent vehicles (or axles) have a different transverse position on the bridge deck. The DOB is large, > 0.8 , when $\sigma_{hs} > 30$ MPa or < -15 MPa. This agrees with the DOB in the experiments of Section 7.2.

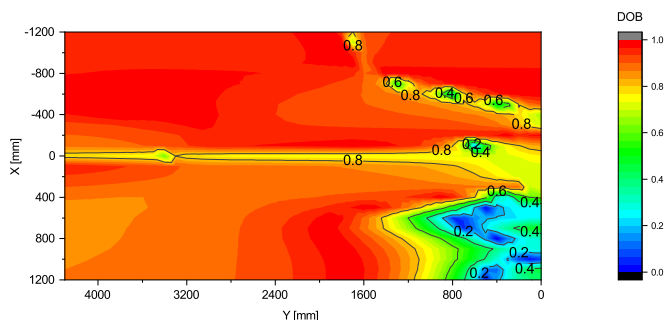
The maximum HSS range from the current study is approximately 63 MPa. The fatigue resistance curve in (pr)EN 1993-1-9 [16, 17] for such a detail has a slope $m = 5$ for the stress ranges lower than the constant amplitude limit, which is assumed at a number of cycles equal to $N_D = 5 \times 10^6$. A cut-off limit $N_L = 10^8$ is designated below which the

stress ranges of a variable-amplitude loading do not contribute to fatigue damage.

The fatigue resistances $\Delta\sigma_{C,hs}$ of 112 MPa and 160 MPa obtained in Section 7.3 have constant amplitude fatigue limits $\Delta\sigma_D$ of 83 MPa and 118 MPa, and cut-off stresses $\Delta\sigma_L$ of 45 MPa and 65 MPa, respectively. The maximum HSS range in the considered deck is hence lower than the cut-off stress range proposed for the assessment of existing bridge decks, and it is in between the constant amplitude fatigue limit and the cut-off stress range for new bridge decks. This implies that the considered deck will pass the assessment of detail C3a, and it will also be sufficient for most designs. Only in the case of very heavily loaded bridges, a stronger deck might be required for detail C3a.

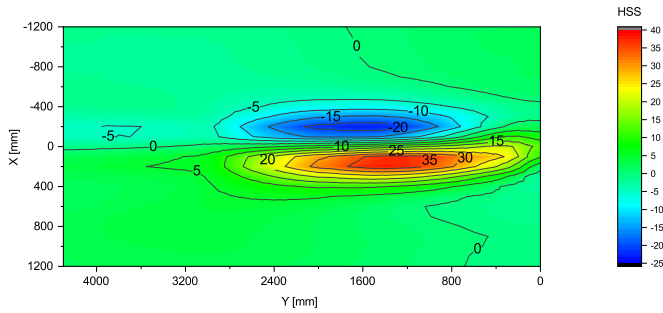


(a) Path 1, HSS

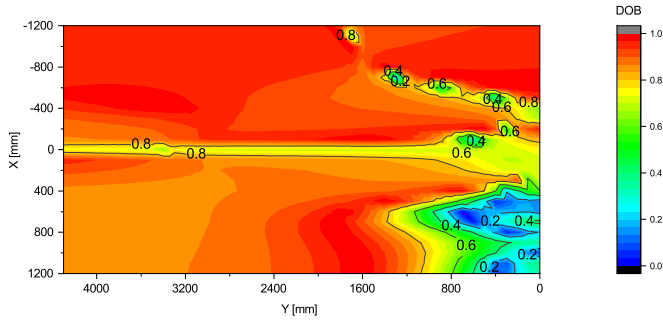


(b) Path 1, DOB

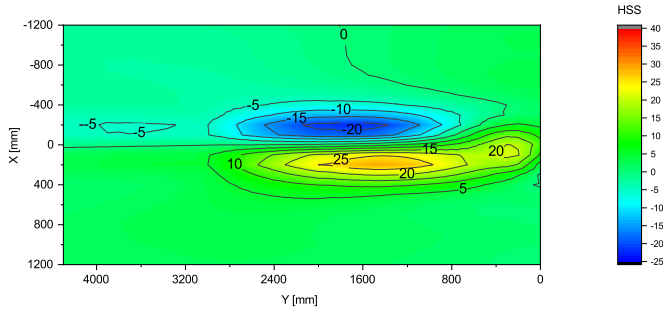
(c) Path 2, HSS



(d) Path 2, DOB



(e) Path 3, HSS



(f) Path 3, DOB

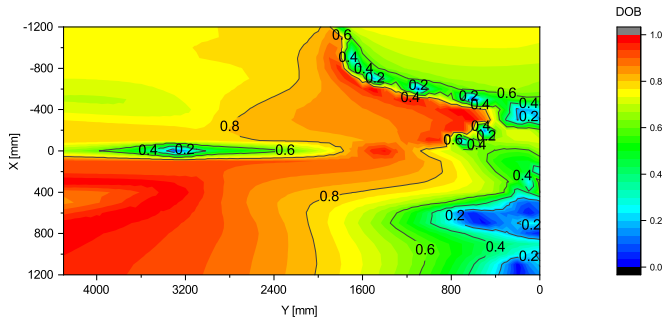


Figure 7.15: Effect of the load position on the HSS and the DOB at the S5 S connection.

7.5. EFFECTIVE NOTCH STRESS ASSESSMENT OF FATIGUE RESISTANCE

A weld flank angle $\theta = 45^\circ$ is used in the simulations in Section 7.2. In practice, θ depends on the welding procedure. For instance, overhead welding may result in a convex weld profile. The HSS is insensitive to the local weld geometry such as the flank angle. However, it is known that the flank angle can influence fatigue resistance [17]. Therefore, the influence of weld flank angle is examined by the Effective Notch Stress (ENS). The sub modelling method available in Abaqus [179] bridges the difference in scale between the local weld geometry (sub model) and the overall specimen geometry (global model). The overall specimen geometry is the same as that of Section 7.2.3. The displacements of the sub model are constrained to match the displacements of the global model at the intersections. IIW recommends a notch radius $R_{\text{ref}} = 1$ mm and local refined mesh at the studied locations [23]. Both recommendations are applied in this study. Element type C3D20R is used in the sub models. The outer (grey) and inner (green) parts have element sizes of approximately $0.5 \times 0.5 \times 0.5 \text{ mm}^3$ and $0.2 \times 0.2 \times 0.2 \text{ mm}^3$, respectively. Figure 7.16 shows the sub models with three types of weld profiles. “Tie” constraints connect the two parts [179].

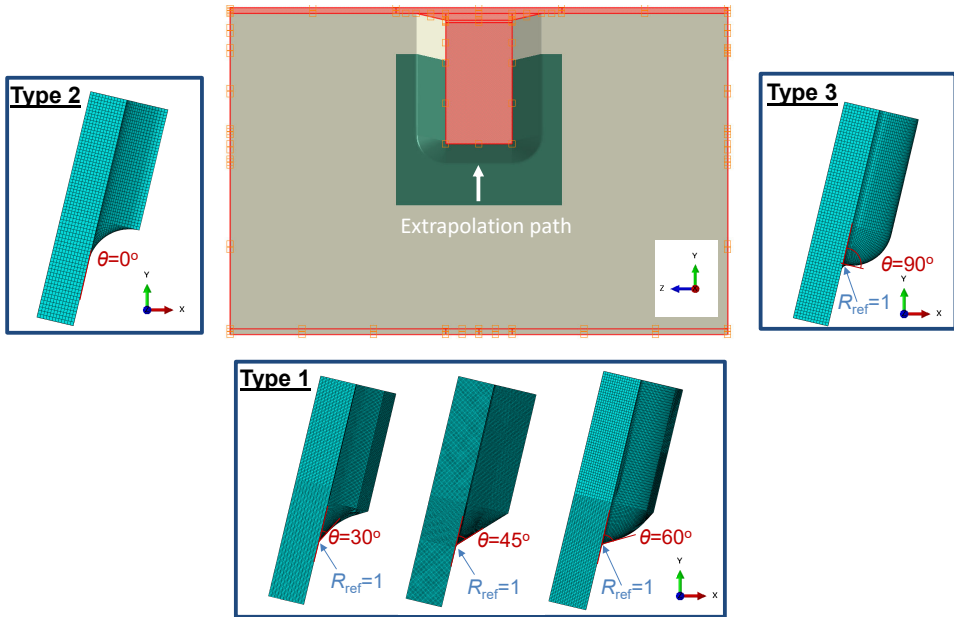
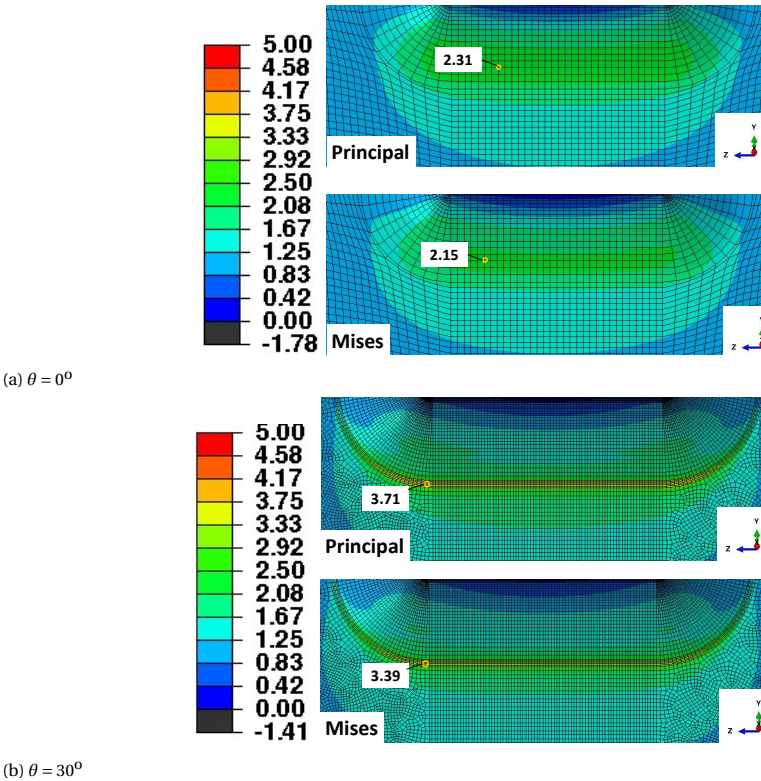


Figure 7.16: Overview of the sub models used for the ENS (unit: mm).

The highlighted surfaces of the outer part are driven by the global model, as shown in Figure 7.6. Type 1 represents the weld profiles with $\theta = 30^\circ$, 45° , or 60° , which are all within the requirements of prEN 1993-1-9 [17]. Types 2 and 3 represent the extreme cases with $\theta = 0^\circ$ and 90° , respectively. Type 2 represents a post-weld grind treatment [2, 154],

whereas Type 3 is considered to study the potentially detrimental effect of a large flank angle, which might be a consequence of the limited space at the lower end of weld. Note that the element size in the Type 2 detail model is larger than the other two types. This is assumed not to affect the analysis as the transition between the weld and the base metal is smooth, resulting in smaller stress concentrations.

Figure 7.17 shows the principal stress and the von Mises equivalent stress under the load of 2×1 kN. The maximum principal stress criterion is more suitable than the von Mises stress criterion for proportional loading [53]. Both criteria are presented for comparison. The maximum stress in the Type 2 detail is predicted on the weld profile with a small shift to the free edge side. The models of Type 1 and Type 3 details predict the maximum stresses in the transition region, at the corner of the lower end at the free edge side. Interestingly, the Type 2 detail predicts a higher maximum stress for the $\theta = 45^\circ$ flank angle than the other flank angles of 30° or 60° . This is attributed to a complex interaction between the loaded stiffener and the crossbeam web. It is also observed that the HSS calculated from the sub models is affected by θ . The Type 3 detail model predicts a higher maximum stress than the other two.



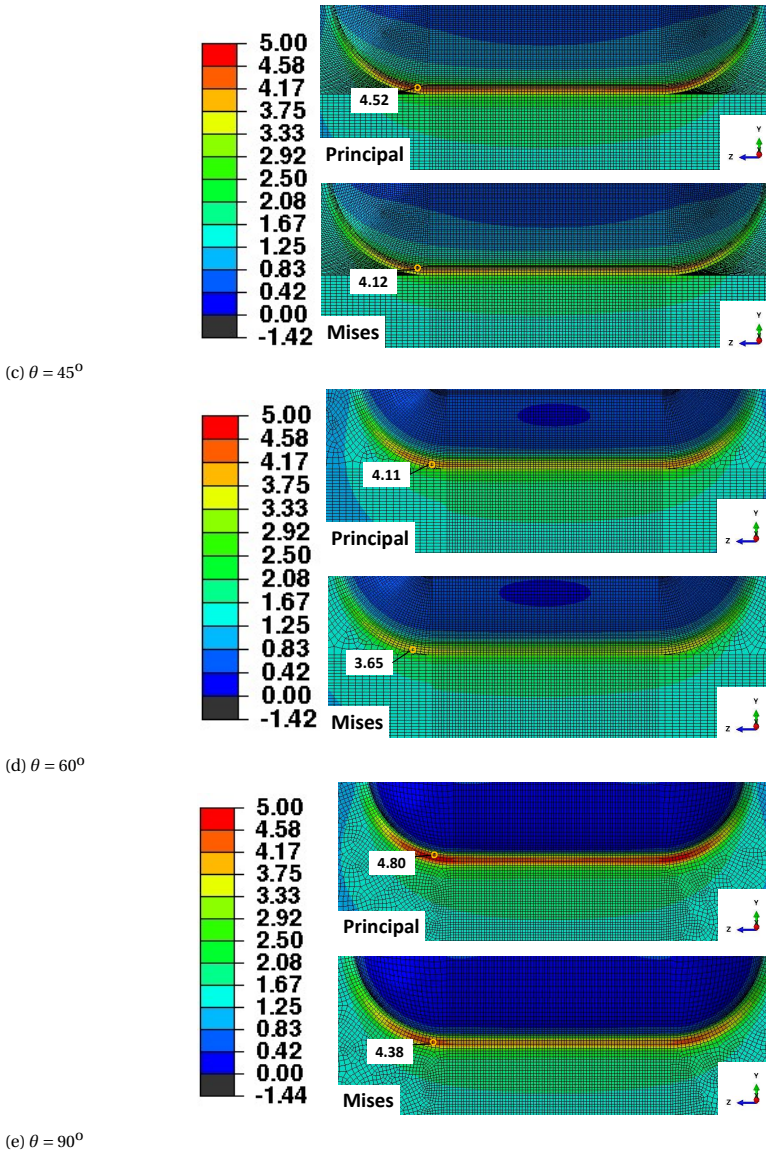


Figure 7.17: The ENS of detail C3a with θ under the load of 2×1 kN (unit: MPa).

A scale factor, K_{en} , is calculated by Equation (7.2) [34]. Either maximum principal stress or von Mises equivalent stress is used for σ_{en} . K_{en} is, therefore, calculated using the HSS from the sub models. The HSS is extrapolated following the white arrow direction in Figure 7.16.

$$K_{en} = \sigma_{en} / \sigma_{hs} \quad (7.2)$$

The predicted fatigue resistance of HSS, $\Delta\sigma_{C,hs}$, can be obtained using the characteristic

values of ENS, $\Delta\sigma_{C,en}$, as shown in Equation (7.3) with 225 MPa and 200 MPa for the principal stress and the von Mises equivalent stress criteria [17], respectively.

$$\Delta\sigma_{C,hs} = \Delta\sigma_{C,en} / K_{en} \quad (7.3)$$

Figure 7.18 compares the predicted $\Delta\sigma_{C,hs}$ with θ . As R_{ref} is not available for the $\theta = 0^\circ$ (Type 2) model, the values 166 MPa and 158 MPa are presented in hollow columns for comparison only.

The scale factor K_{en} increases with θ in the range of 0° to 45° and afterwards keeps constant until $\theta = 90^\circ$. The von Mises equivalent stress criterion gives lower K_{en} and $\Delta\sigma_{C,hs}$ predictions than the principal stress criterion. The $\Delta\sigma_{C,hs}$ based on the ENS ranges from 97 MPa to 115 MPa in the Type 1 detail, close to the statistical analysis obtained values of 94 MPa and 117 MPa in Section 7.3 using the failure criteria of N_2 and N_3 , respectively. Only 3% to 4% lower fatigue resistance results for $\theta = 90^\circ$ compared with the $\theta = 45^\circ$ case. The results imply that, compared with the standard flank angle of $\theta = 45^\circ$, a higher fatigue resistance is obtained for a smaller angle, whereas a larger angle gives no significant reduction in the resistance.

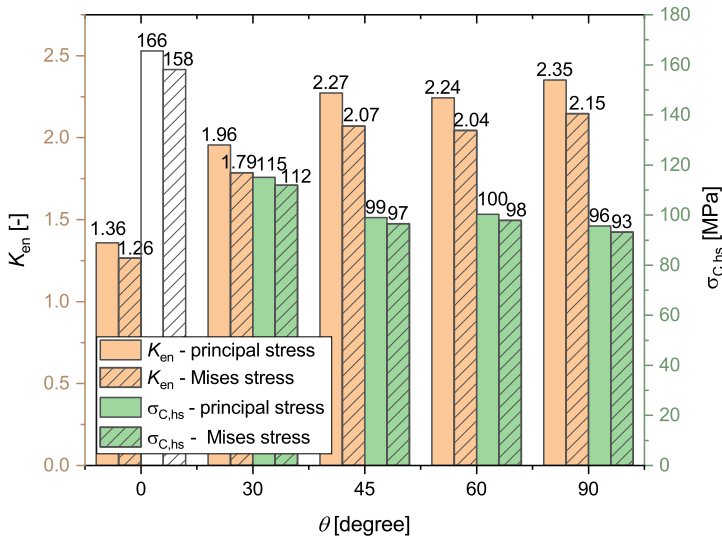


Figure 7.18: K_{en} and $\Delta\sigma_{C,hs}$ versus θ for detail C3a.

7.6. FATIGUE BEHAVIOUR OF DETAIL C3A CONSIDERING WRS

7.6.1. INTRODUCTION OF THE FE MODEL

Figure 7.19 shows the FE model for the welding simulation of the detail at the S5 S connection. The brick element meshed part in the FE model presented in Figure 7.7 is used for the welding simulation. The model is rotated upside down from Figure 7.6 with three-dimensional displacement constraints, $UX = UY = UZ = 0$, applied at the top of the deck plate (bottom in Figure 7.19). Symmetric boundary conditions are applied to Sides 1 and 3 ($UX = URY = URZ = 0$), and Sides 2 and 4 ($UZ = URX = URY = 0$). The FE model contains a global model (in yellow) and a sub model 1 (in green, in short Sub 1) that are connected by “tie” constraints [179]. The elements in Sub 1 are refined, whereas the elements in the global model remain unchanged when “tie” constraints are used. The global model has a size of $3200 \times 5100 \times 1000 \text{ mm}^3$ with the crossbeam located approximately in the middle. Sub 1 consists of half of the lower part of one closed stiffener and part of the crossbeam with dimensions shown in the close-up. At the weld toe, a flank angle $\theta = 45^\circ$ and a local notch with a radius $R_{\text{ref}} = 1 \text{ mm}$ are created, see Figure 7.16 for the geometric characteristics. The welding simulation procedure used in this section is the same as the validated model as shown in Appendix D.4 where a fillet welded connection in the literature [217] is studied. In Appendix D.4, the FE predicted WRS agrees well with the experimental measurements.

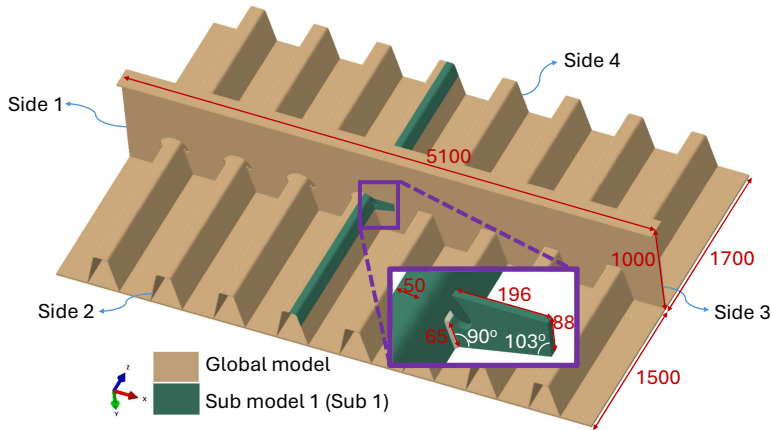


Figure 7.19: FE model for the welding simulation of S5 S (unit: mm).

An uncoupled thermal-mechanical procedure is used for the welding simulation. First, the thermal simulation is carried out to analyse the temperature distribution. Afterwards, the mechanical simulation is conducted for the deformation, the strain and stress states by mapping the temperature distribution calculated from the thermal simulation to the mechanical model. In order to correctly map the nodal temperature obtained in the thermal simulation to the mechanical simulation, same mesh and increments need to be used in the thermal and the mechanical models. The following sections

introduce: the thermal and mechanical material properties (Section 7.6.2), the results of the thermal simulation (Section 7.6.3), and the results of the mechanical simulation (Section 7.6.4).

7.6.2. MATERIAL PROPERTIES IN THE FE MODEL

Figure 7.20 shows the thermal and mechanical properties of the mild strength structural steel S355 used in the simulation which is adopted from [217, 218]. The elastic modulus, expansion coefficient, specific heat capacity, thermal conductivity, and yield stress show strong temperature dependence, see Figure 7.20. The bilinear hardening constitutive model is used with the von-Mises yield criterion, Figure 7.20b. Note that the elastic modulus and the yield stress are assumed to be constant when the temperature exceeds 800 °C and 1000 °C, respectively. The post heated (after cooling down) mechanical properties of the region that receives a maximum temperature exceeding 750 °C are assumed to be affected [219]. The anneal temperature is fixed to 1300 °C [220], and the melting point is assumed at 1400 °C [217] (the maximum value shown in Figure 7.20). The thermal conductivity is assumed to be 300 W/(m·°C) at 1400 °C to consider the fluid flow [218, 221]. Moreover, the expansion coefficient is approximately assumed to be 0 for the weld filler when the temperature exceeds 550 °C. Note that the phase transformation is not considered in the simulation due to its insignificant effect on the WRS of low carbon steel (e.g. S355J2) [221].

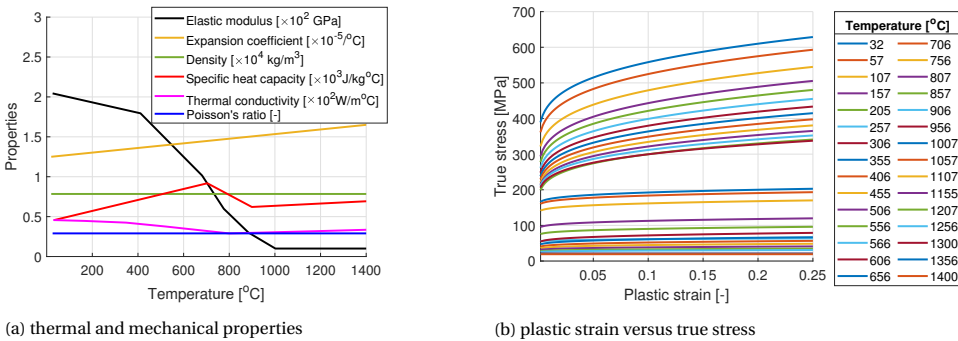


Figure 7.20: Temperature-dependent material properties used in the welding simulation [217, 218].

7.6.3. THERMAL SIMULATION

WELDING PROCEDURE

Figure 7.21 illustrates the welding procedure in the simulation. 8-node linear brick diffusive heat transfer elements (DC3D8) [179] are used with approximate element sizes of $45 \times 45 \times 45$ mm³ and $2 \times 2 \times 2$ mm³ in the global model and Sub 1, respectively. Local mesh in the vicinity of the weld is further refined up to approximately $0.5 \times 0.5 \times 0.5$ mm³. The welding is simplified into five steps. The welding speed is kept constant, 5 mm/s. The welding is assumed to start from the location 10 mm away from the top of Sub 1 and moves in a straight line for 55 mm (Step 1), and is followed by the corner welding (Step

2). The torch then passes the lower end (Step 3) and the corner of the other side (Step 4). Finally, the welding ends at the other side at the start height (Step 5). A total welding time of 27 seconds is simulated. Results of seven locations, from a3 to b3, will be discussed in the following sections.

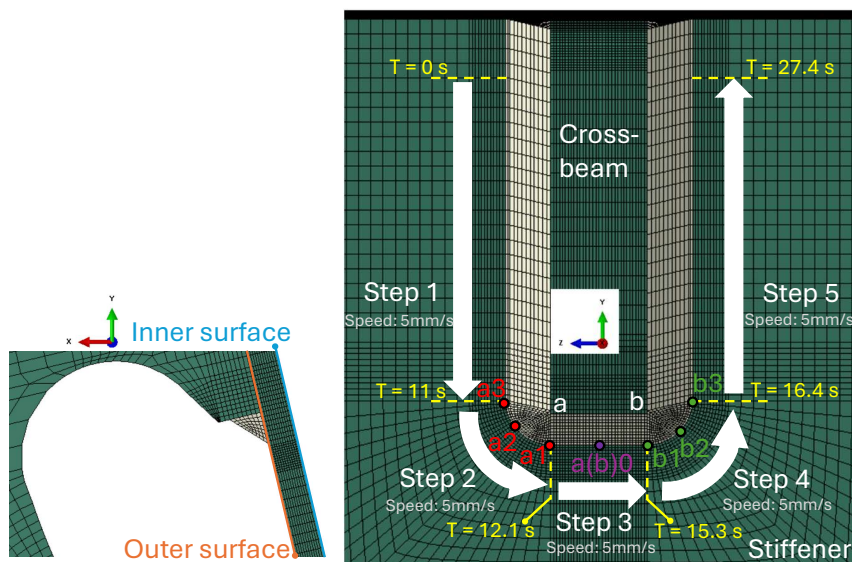


Figure 7.21: Illustration of the welding procedure in the simulation (part of Sub 1).

HEAT SOURCE

Arc welding uses an electric arc from alternating current or direct current to generate heat energy. At the torch location, the weld filler and the base metal are melted. When cooling down, the originally separated parts are merged into one. Note that the filled region after solidification is also called the region of Weld Metal (WM). One commonly used heat source model is the double-ellipsoidal heat source model which was proposed by Goldak [222], as illustrated in Figure 7.22. The model combines two half-ellipses with dimensions a_f (ahead of the origin) and a_r (behind the origin) in the welding direction (X-axis direction). b and c are the dimensions in the Y-axis and the Z-axis directions, respectively. The volumetric heat flux q (heat per unit volume per unit time) of the welding arc is distributed as defined by Equations (7.4) to (7.6) [221, 222]. In the welding simulation, the local coordinate system in Figure 7.22 is transferred and rotated to the weld surface as shown in Figure 7.23.

In the FE simulation, the weld filler is first deactivated. It is activated block-by-block following the time increment. In the current study, one block is 5 mm long in the X-axis direction in Steps 1 and 5, corresponding to one second of welding. Steps 2 and 4 contain 2 blocks. Step 3 contains 3 blocks. The blocks are also approximately 5 mm long in Steps 2, 3, and 4. When a block is activated, an initial temperature of 1450 °C is assigned to the elements in the block, which is immediately released. This procedure is used to simulate the drop of melted weld filler. In the meantime, the double-ellipsoidal

heat source is moved to the activated block.

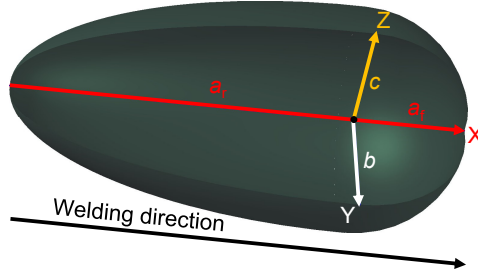


Figure 7.22: Illustration of the double-ellipsoidal heat source model [222].

$$q(x, y, z, t) = \frac{6\sqrt{3}f_f Q}{a_f b c \pi \sqrt{\pi}} e^{-3(x-vt-x_0)^2/a_f^2} e^{-3y^2/b^2} e^{-3z^2/c^2} \quad (7.4)$$

$$q(x, y, z, t) = \frac{6\sqrt{3}f_r Q}{a_r b c \pi \sqrt{\pi}} e^{-3(x-vt-x_0)^2/a_r^2} e^{-3y^2/b^2} e^{-3z^2/c^2} \quad (7.5)$$

where: The fraction of heat input in the front or the rear part sum to be 2 ($f_f + f_r = 2$) as assumed by the double-ellipsoidal heat source model [222]. The welding speed v , welding time t , initial location x_0 determine the location of the double-ellipsoidal heat source model. The heat input per unit time, Q , is calculated as:

$$Q = \eta UI \quad (7.6)$$

The arc efficiency η is assumed as 0.85 for Flux-Cored Arc Welding (FCAW). Following the technical specifications provided by the manufacturer, the current (I) and the voltage (U) in the simulation are set as 300 amp and 30 volts, respectively.

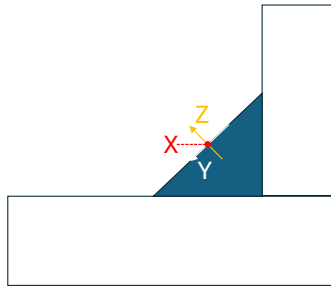


Figure 7.23: Illustration of the local coordinate system of the double-ellipsoidal heat source model in a welded connection.

Table 7.3: Parameters of the double-ellipsoidal heat source model used in the welding simulation.

Variable	f_f	f_r	a_f	a_r	b	c
Value	0.67	1.33	15	30	6	6
Unit	-	-	mm	mm	mm	mm

HEAT TRANSFER

The heat loss consists of the convection loss and the radiation loss. The convection-induced heat flux, q_c , is modelled by the Newton's equation, Equation (7.7), with a uni-form film coefficient, $h = 20 \text{ W}/(\text{m} \cdot ^\circ\text{C})$.

$$q_c = -h(T_s - T_a) \quad (7.7)$$

where T_s and T_a are the surface temperature and the ambient temperature, respectively. The ambient temperature is assumed to be 20°C in the current study.

The radiation-induced heat loss follows the Stefan-Boltzmann equation with a uni-form emissivity coefficient of 0.7. Note that the steel-to-steel interfaces between the global model and Sub 1, as shown in Figure 7.19, are assumed to have an emissivity co-efficient of 1 to avoid the temperature discontinuity at the interfaces. Other physical constants in the FE model are: absolute temperature (-273.15°C), Stefan-Boltzmann constant ($5.67 \times 10^{-8} \text{ W}/(\text{m}^2 \cdot \text{K}^4)$), and universal gas constant ($8.314 \text{ J}/(\text{mol} \cdot \text{K})$).

RESULTS OF THERMAL SIMULATION

Figure 7.24 shows the first 200 seconds time history of the nodal temperatures (in degrees of Celsius) at the designated locations. The maximum temperatures are observed near a3 and b3, followed by a(b)0. The temperatures at the outer surface side (Figure 7.21) are larger than the ones at the inner surface side (Figure 7.21). Note that the nodal temperatures drop to ambient temperature after approximately 1000 seconds and keep constant until the end of the simulation (the total cooling time exceeds 2000 seconds). Figure 7.25 shows the contours of the nodal temperatures (NT11) in the cross section at location a(b)0 with the normal direction of the Z-axis during the welding time ranging from 13.2 seconds to 20.4 seconds. The stiffener has received a maximum temperature $\text{NT11} > 750^\circ\text{C}$ through the thickness. Note that the contour presented in Figure 7.25a is discontinuous as the torch reaches location a(b)0 at 13.7 seconds and the region shown in Figure 7.25a is behind location a(b)0. Figure 7.26 shows the region affected by high temperature (in grey) including a simplified region with the size of $1 \text{ mm} \times 2 \text{ mm}$ in front of the weld toe at the lower end, which is used to define the region of heat affected zone in Section 7.6.5.

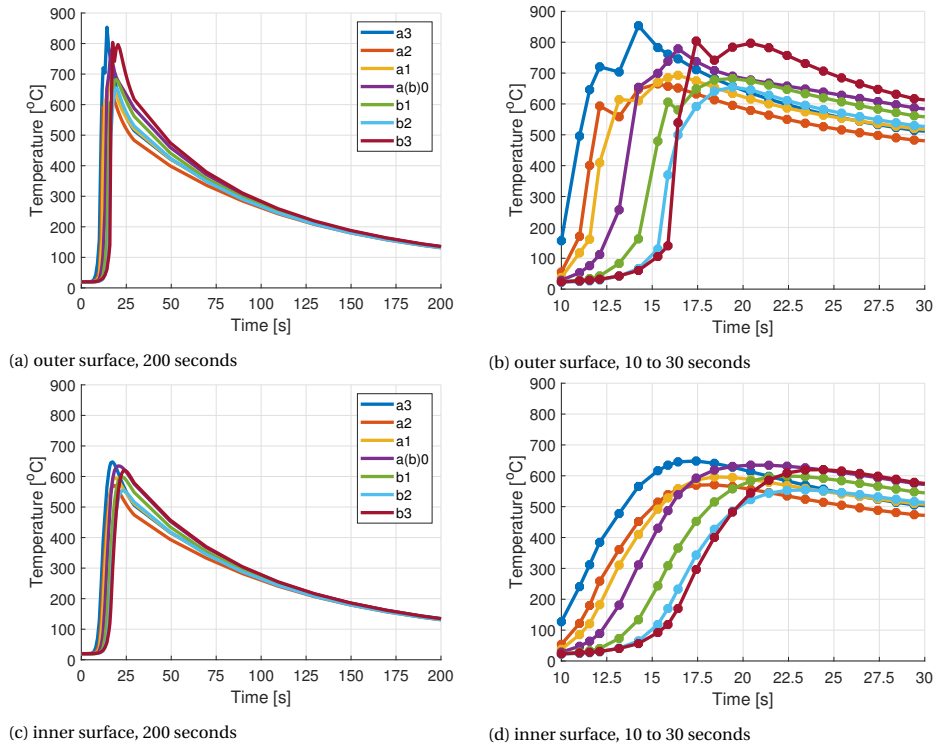


Figure 7.24: Time history of nodal temperatures at the critical locations of the welded connection during the first 200 seconds (left) with close-ups during 10 to 30 seconds (right), see Figure 7.21 for the designation of the locations.

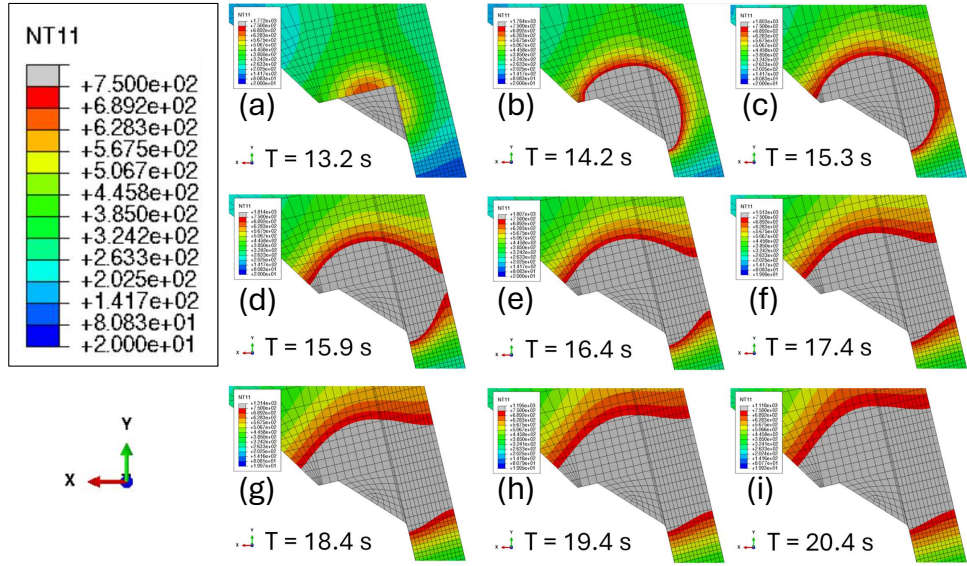


Figure 7.25: Nodal temperature distributions in the vicinity of the lower end with grey colour for $NT11 > 750^{\circ}\text{C}$ (cross section with the normal direction of the Z-axis and crossing location a(b)0).

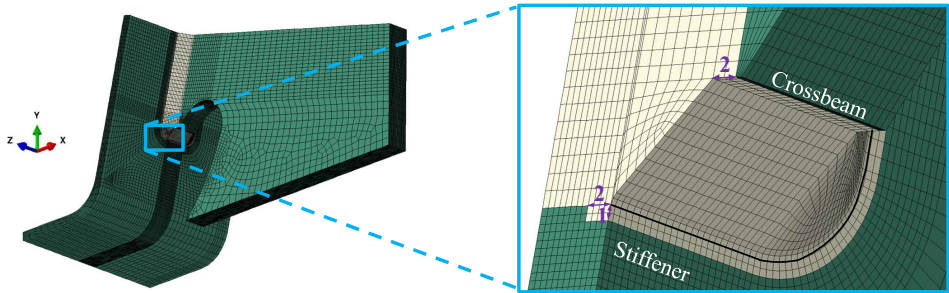


Figure 7.26: Region affected by high temperature ($NT11 > 750^{\circ}\text{C}$) designated in the FE model (in grey) according to the welding simulation (unit: mm).

7.6.4. MECHANICAL SIMULATION

Same mesh as the heat transfer model is used, whereas the 8-node linear brick elements with reduced integration (C3D8R) [179] are used in the mechanical model. Figure 7.27 shows the distribution of the WRS (the principal stress in absolute values $\sigma_{\text{principal}}$) at the selected locations across the stiffener thickness. The stress perpendicular to the weld toe is designated as the transverse stress (σ_{11}). The stress in parallel with the weld toe is designated as the longitudinal stress (σ_{22}) in sub figures b, c, and d. d_{norm} is the normalised depth calculated using the distance divided by the stiffener thickness. High $\sigma_{\text{principal}}$ up to approximately 600 MPa is found at the outer surface side of location

a1, which decreases to the range from 300 MPa to 400 MPa when reaching the inner surface side. At locations a3 and b3, the values remain approximately constant when $d_{\text{norm}} \geq 0.5$. Locations a2 and b2 have close $\sigma_{\text{principal}}$, whereas the values at location a(b)0 are smaller when $0.17 \leq d_{\text{norm}} \leq 0.83$. The trend for σ_{11} is similar but the values are slightly smaller. σ_{22} has the maximum values at the outer surface side (in the range of 300 MPa to 400 MPa) which are 100 MPa to 250 MPa at the inner surface side, except for locations a2 and b2 where smaller values are observed.

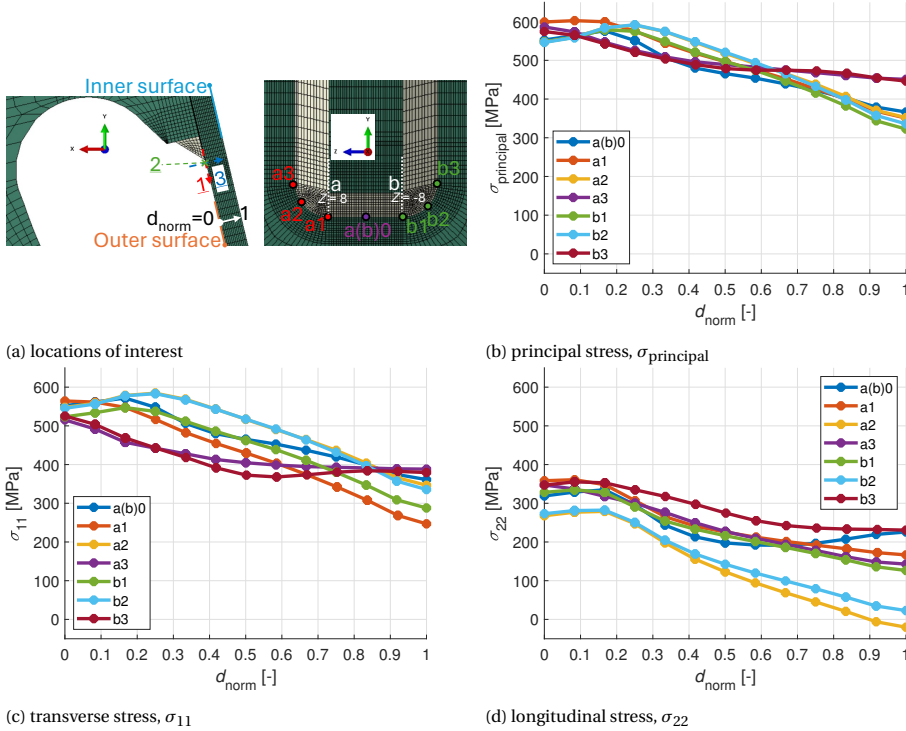


Figure 7.27: WRS at the locations of interest across the stiffener thickness, $t_s = 6$ mm.

Figure 7.28 shows $\sigma_{\text{principal}}$ and σ_{11} at three normalised depths along the Z-axis (3-axis in the local coordinate system) at the lower end with a distance of 1 mm from the weld toe. The WRS distribution is approximately uniform except for σ_{11} at $d_{\text{norm}} = 1.0$ (inner surface) where concentration occurs in the middle of the crossbeam centreline ($Z = 0$ mm).

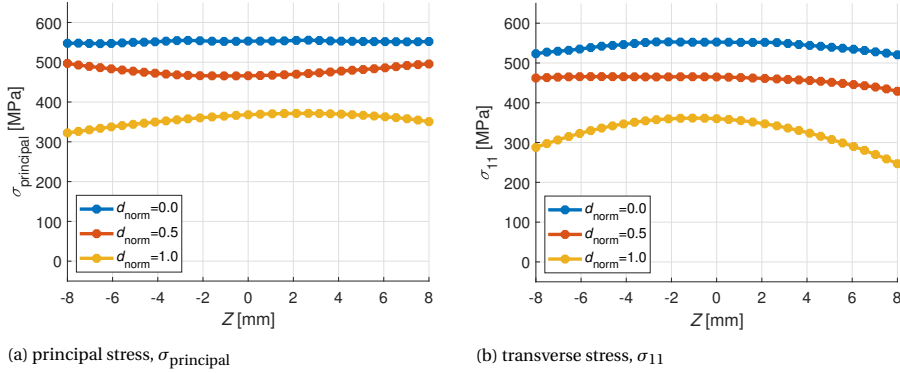


Figure 7.28: WRS located 1 mm away from the weld toe at the lower end at the three normalised depths.

7.6.5. MECHANICAL RESPONSE UNDER THE CYCLIC LOADING INCLUDING WRS

The WRS of S5 S after welding is analysed in Section 7.6.4. This section studies the mechanical response of S5 S under external cyclic loading considering WRS. The WRS calculated in Section 7.6.4 is introduced into the mechanical model as the “initial stress” [179] with the mesh unchanged. One hundred cycles of loading are afterwards applied to Sub 1 in which the displacements at the intersections are driven by the results of $\Delta F = 2 \times 150$ kN and $\Delta F = 2 \times 250$ kN (see Figures 7.3 and 7.6) using the sub modelling method available in Abaqus [179]. After the one hundred cycles of loading, the loads are set to $F = 0$ (unloaded status of the driving model).

CYCLIC CONSTITUTIVE MODELS

The cyclic mechanical behaviour is described by the combined hardening model with three back stresses. Parameters of the cyclic elastic-plastic constitutive models are summarised in Table 7.4. The values of the Heat Affected Zone (HAZ) and the Base Metal (BM), using the structural steel S355J2+H as the BM in [223], are adopted in the current study. The material properties of HAZ are assigned to the region affected by high temperature ($NT11 > 750$ °C) as shown in Figure 7.26. The material properties of BM are assigned to the remaining part of the model. Note that the WM is assumed to have the same material properties as the HAZ. This simplified assumption is made because the mechanical loading induced stress at the WM is relatively small and the performance is insensitive to the plastic properties.

Table 7.4: Parameters of the cyclic elastic-plastic constitutive models for the welded connection using the structural steel S355J2+H as base metal in [223].

Parameter	HAZ	BM	Unit
Elastic modulus (E)	195	197	[GPa]
Initial yield stress ($\sigma_{y,0}$)	415.33	243.43	[MPa]
Poisson's ratio (ν)	0.3	0.3	[-]
Isotropic hardening magnitude for cyclic hardening (Q_∞)	8.95	13.37	[MPa]
Isotropic hardening rate parameter for cyclic hardening (b)	32.20	26.81	[-]
Kinematic hardening magnitude for 1 st back stress (C_1)	78105	16268	[MPa]
Kinematic hardening rate parameter for 1 st back stress (γ_1)	12808	168.9	[-]
Kinematic hardening magnitude for 2 nd back stress (C_2)	84330	635071	[MPa]
Kinematic hardening rate parameter for 2 nd back stress (γ_2)	1487.7	10850	[-]
Kinematic hardening magnitude for 3 rd back stress (C_3)	34798	116551	[MPa]
Kinematic hardening rate parameter for 3 rd back stress (γ_3)	228.60	1343.6	[-]

7.6.6. STRESS AND STRAIN IN CYCLIC MECHANICAL LOADING

Figures 7.29 and 7.30 show the loading history of the stresses and strains at the integration points of the first element and the second element from the weld toe at the outer surface of stiffener ($d_{\text{norm}} = 0.0$), located at a(b)0 under load ranges $\Delta F = 2 \times 150$ kN and $\Delta F = 2 \times 250$ kN, respectively. The applied force is normalised to F_{norm} and presented as well. F_{norm} increases to the mean value of 0.55 in the first cycle, and then ranges between 0.1 and 1.0 following the sine shape wave. When $1.00 \leq N \leq 1.25$, $\sigma_{\text{principal}}$, σ_{Mises} , and σ_{11} are almost constant, see Figures 7.29a and b. σ_{Mises} is present for the purpose of comparison. The values drop following the decrease in F_{norm} when $N \geq 1.25$. $\sigma_{\text{principal}}$ first drops from 747 MPa to 400 MPa and then grows back to 747 MPa in one cycle ($\Delta N = 1$). σ_{11} is slightly smaller than $\sigma_{\text{principal}}$ and contributes as the dominant stress component. σ_{22} is smaller than σ_{11} . $\Delta\sigma_{\text{Mises}}$ is between $\Delta\sigma_{11}$ and $\Delta\sigma_{22}$. σ_{33} is constantly the smallest and close to 0 at the second element. Interestingly, ϵ_{33} is very close to $\epsilon_{\text{principal}}$ at the first element with values around 0.025. $\epsilon_{\text{principal}}$ and ϵ_{33} are approximately constant in cyclic loading. At the second element, ϵ_{11} is slightly smaller than $\epsilon_{\text{principal}}$ with values approximately 0.015. ϵ_{11} and $\epsilon_{\text{principal}}$ vary slightly in cyclic loading. ϵ_{22} is approximately -0.01 and almost keeps constant in cyclic loading in both elements. It is noticed that ϵ_{11} is approximately -0.015 at the first element. It indicates that the welding induces local distortion at the weld toe.

Similar behaviour is observed for the stresses and strains under $\Delta F = 2 \times 250$ kN, see Figure 7.30. One exception is that σ_{22} dominates $\sigma_{\text{principal}}$ at the first element, and dominates $\sigma_{\text{principal}}$ and σ_{Mises} at the second element, both when $F_{\text{norm}} < 0.25$. σ_{11} show the sine wave shape change following the load when $N \geq 1.25$.

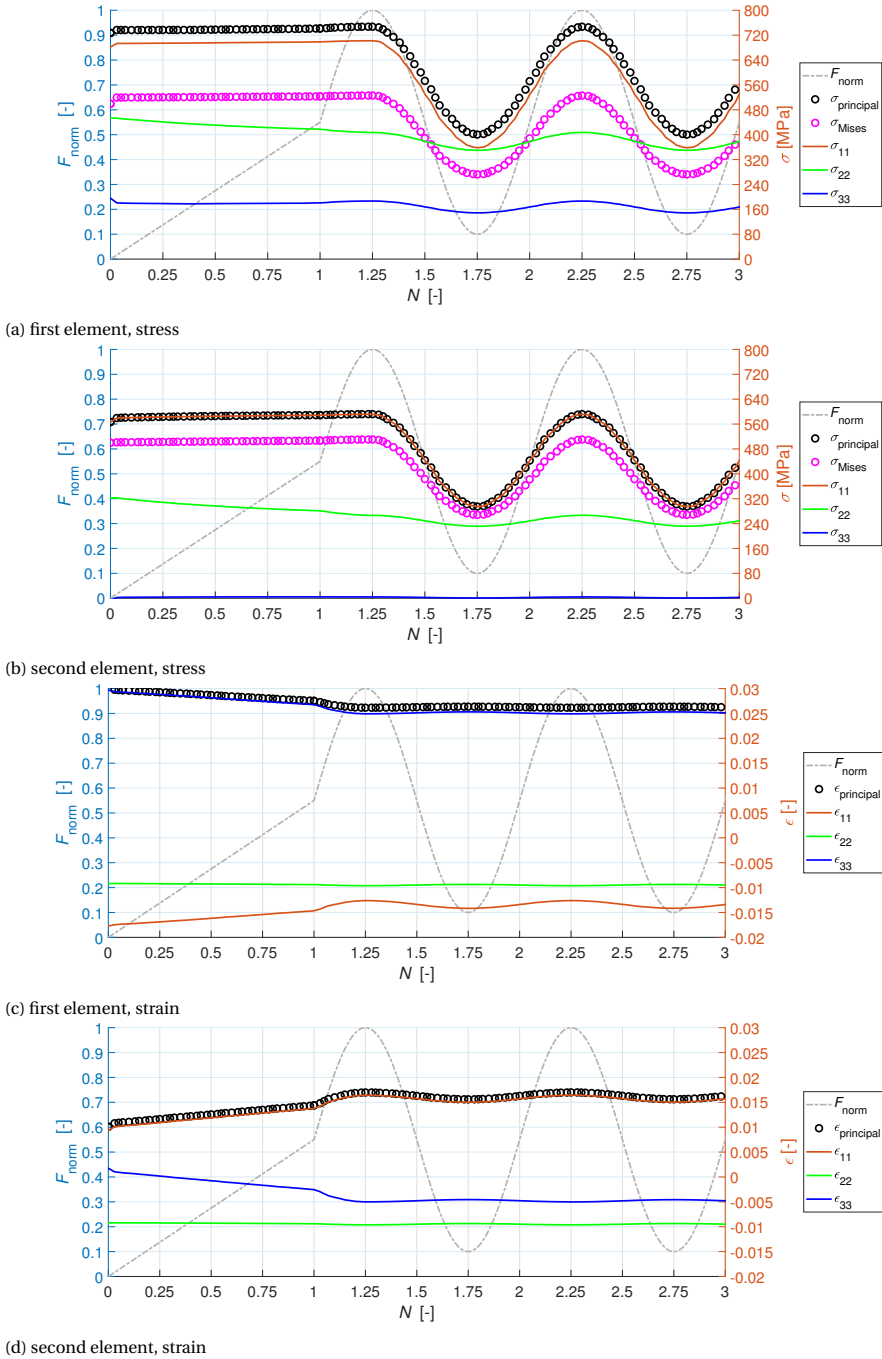
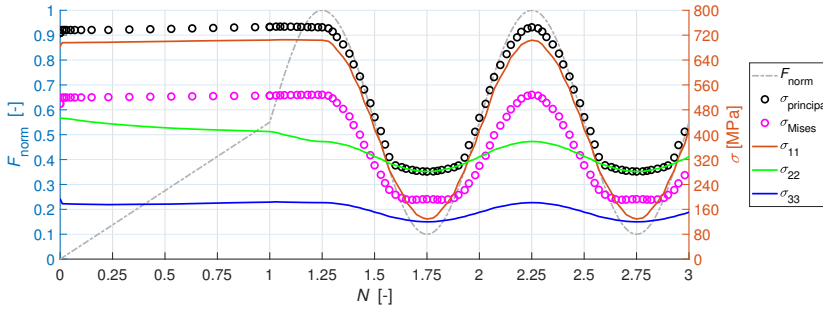
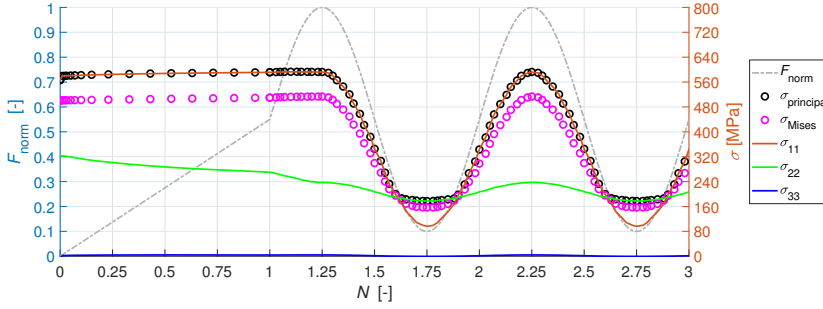


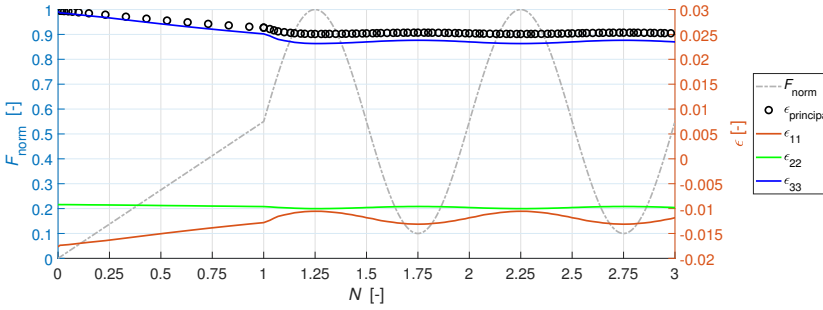
Figure 7.29: Loading history of the stresses and strains in the first two elements in front of the lower end of S5 S, ab(0) at $d_{\text{norm}} = 0.0$, under $\Delta F = 2 \times 150$ kN.



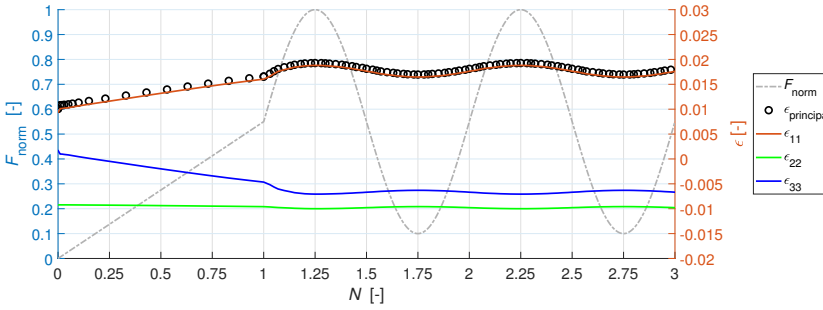
(a) first element, stress



(b) second element, stress



(c) first element, strain



(d) second element, strain

Figure 7.30: Loading history of the stresses and strains in the first two elements in front of the lower end of S5 S, $ab(0)$ at $d_{\text{norm}} = 0.0$, under $\Delta F = 2 \times 250$ kN.

7.7. FATIGUE RESISTANCE PREDICTION CONSIDERING WRS

7.7.1. NOTCH STRAIN METHOD

Figure 7.31 shows the Notch StrAin (NSA) in the current experiments and the prediction curves obtained in the literature [224, 225], in which the material parameters were obtained using small-scale coupon specimens with WRS significantly released. The $\Delta\epsilon_{11}$ of the second element close to the weld toe is, as shown in Figures 7.29 and 7.30, is used as $\Delta\epsilon_{\text{eff}}$. The earliest detected failure among N_i of each detail, as summarised in Table 7.12, is defined as the crack initiation. The strain-based method covers both low and high cycle fatigue, which can be used for the assessment under high strain (or stress) levels. It is therefore adopted for the fatigue crack initiation prediction of detail C3a considering WRS, where the stresses close to the weld toe exceeding the yield values in the current study. Note that the experiments in the literature used the load ratio $R = -1$, whereas high stress ratios are observed at the weld toe in the current study as shown in Section 7.6.5. Two types of predictions are therefore given following the assumptions:

- NSA 1: The compressive semi-cycle does not contribute to the fatigue crack initiation. The effective strain range is equal to the tension range: $\Delta\epsilon_{\text{eff}} = \Delta\epsilon_{\text{tension}}$.
- NSA 2: The complete cycle contributes to the fatigue crack initiation. The effective strain range is equal to the sum of the tension and compression ranges: $\Delta\epsilon_{\text{eff}} = \Delta\epsilon_{\text{tension}} + \Delta\epsilon_{\text{compression}}$.

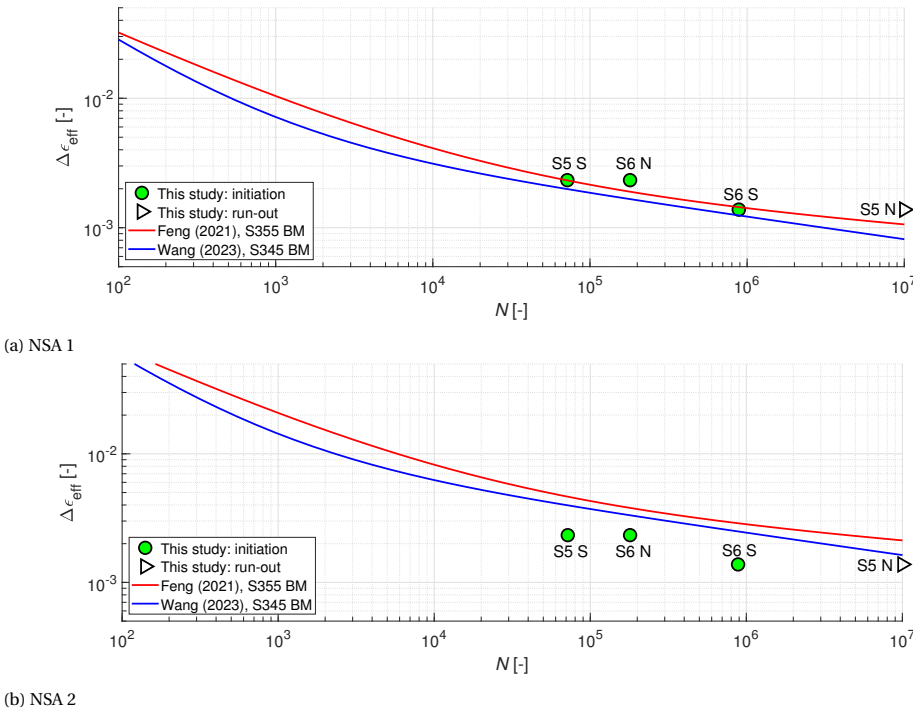


Figure 7.31: Comparison of the current experiments and the prediction curves used in [224, 225] for the fatigue crack initiation using the NSA method.

NSA 1 gives a good prediction for the details in the current experiments, see Figure 7.31a. The curve based on Feng (2021) [224] gives excellent predictions for the crack initiations (earliest detected value in Table 7.2) of S5 S and S6 S, whereas the predictions using parameters from Wang (2023) [225] are slightly conservative. NSA 2 on the other hand over predicts the fatigue crack initiations, Figure 7.31b. It can be concluded that the effect of load ratio should be considered when using the NSA method. Note that BM instead of HAZ in [224, 225] is used in the predictions as parameters for HAZ are currently limited.

7.7.2. FATIGUE CRACK STATES ANALYSED BY ΔJ

INTRODUCTION

With the material degradation under cyclic loading, fatigue crack initiates from highly stressed locations (weld toe here). The homogeneous material assumption is valid when the crack size is larger than several grain [226], although the short crack propagation with crack length at the grain size level is heavily dependent on the microstructure properties. Fracture mechanics becomes an attractive method to study the crack tip states and analyse the crack propagation for the homogeneous material. Stress intensity factors in conjunction with the Paris' equation can be used for the fatigue propagation analysis of long cracks within the Linear Elastic Fracture Mechanics (LEFM) framework. Rela-

tively small plastic zone compared with the crack length is required which limits its application for short cracks [227]. The LEFM is widely applied to fatigue life assessment as the cracks smaller than approximately 0.5 mm may not be detected by typical non-destructive testing methods [228]. However, high WRS exists in welded connections and the initial crack sizes are normally assumed between 0.1 mm to 0.5 mm [23, 24, 34, 64]. The small plastic zone assumption may not be valid and the stress intensity factors calculated based on the LEFM may not always be suitable for the short cracks. The cracking driven parameters, *e.g.* cyclic J-integral ΔJ [72, 229] and Crack Tip Open Displacement range ($\Delta CTOD$) [230] can be used for both the linear elastic and the elastic-plastic situations [231]. Between them, ΔJ is naturally linked with ΔK : $\Delta K_J = \sqrt{E' \cdot \Delta J}$ [232] where ΔK_J is the J-integral based cyclic stress intensity factor. The effective elastic modulus $E' = E/(1 - \nu^2)$ is calculated under the plane strain assumption, E and ν are the elastic modulus and the Poisson's ratio, respectively. This section studies the crack tip states of cracks with a depth from 0.05 mm (the approximate size of small intrusions at the weld toe [233]) to 0.5 mm (practical detectable flaws size [34]). The elastic-plastic material properties are used with the redistributed WRS implemented. The WRS is mainly redistributed in the first cycle of loading as discussed in Section 7.6.6. The results at the end of the calculations (driving model under the unloaded status) in Section 7.6.6 are therefore used as implementation. Refined FE models with a local element size of approximately 0.025 mm are used. The crack is simulated using the element removal method.

ΔJ CALCULATION

Figure 7.32 illustrates the cyclic stress-strain states at the crack tip. Four zones (regions) are identified, in which Zones 1 and 2 represent the elastic behaviour at the far-field region and the near crack tip region [65], respectively. In Zone 3, the plastic stage is reached in the monotonic half cycle loading which is followed by the elastic cyclic behaviour. In Zone 4, the yield value is reached under reversed loading and plasticity occurs.

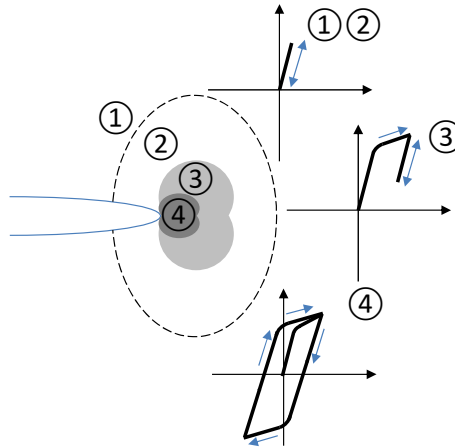


Figure 7.32: Illustration of the stress-strain response at the crack tip [65].

Figure 7.33 illustrates the J-integral calculation in the vicinity of crack tip with the grey part representing the process zone. The integral of strain energy density factor W and traction force T_i along the contour Γ are calculated by Equations (7.8) and (7.9) [63]. The J-integral is path independent and can be calculated by arbitrary contour, e.g. Γ_1 , Γ_2 , or Γ_3 .

The J-integral concept has been extended to cyclic J-integral (ΔJ) for fatigue crack propagation analysis. ΔJ is calculated by Equations (7.10) and (7.11) [75]. ΔJ can be calculated using either loading (ΔJ_{ascend}) or unloading ($\Delta J_{\text{descend}}$) stage if the crack keeps open, see Figure 7.34. Note that the integral of cyclic strain energy density factor ΔW is different from the range of the integral of the strain energy density factor W calculated by Equation (7.9). Thus, ΔJ is not equal to the range of J-integral (maximum value minus minimum value) but can be calculated by Equation (7.10) [75].

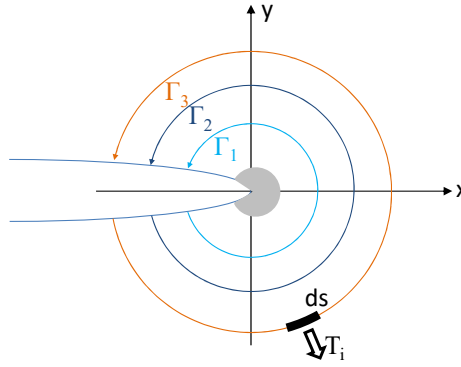


Figure 7.33: Illustration of the contours in the J-integral calculation [63, 75].

$$J = \int_{\Gamma} \left(W \, dy - T_i \frac{\partial u_i}{\partial x} \, ds \right) \quad (7.8)$$

where:

$$W = \int_0^{\varepsilon_{ij}} \sigma_{ij} \, d\varepsilon_{ij} \quad (7.9)$$

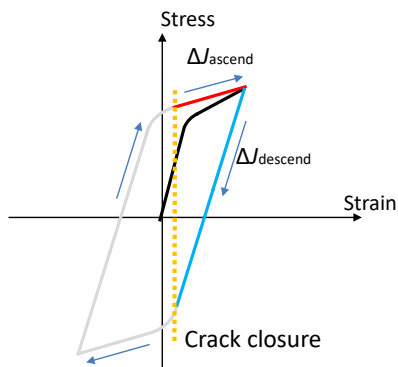


Figure 7.34: Illustration of ΔJ [72, 75, 229].

$$\Delta J = \int_{\Gamma} \left(\Delta W \, dy - \Delta T_i \frac{\partial \Delta u_i}{\partial x} \, ds \right) \quad (7.10)$$

where:

$$\Delta W = \int_0^{\Delta \epsilon_{ij}} \Delta \sigma_{ij} \, d\Delta \tilde{\epsilon}_{ij} \quad (7.11)$$

NUMERICAL CALCULATION OF ΔJ FOR S5 S

The numerical calculation of ΔJ for S5 S follows the flowchart presented in Figure 7.35. Welding simulation is first conducted. Sub 1 is created afterwards to study the mechanical response under cyclic loading considering WRS, as shown in Section 7.6. Sub model 2 (in short Sub 2) is then created with the partitions added in the model using the shapes of elements in Sub 1. One partitioned region in Sub 2 represents one element in Sub 1, whereas the mesh is refined. Partition by partition, the redistributed (after several cycles of loading) WRS obtained from the element in Sub 1 is assigned to Sub 2 as the initial stress. This means that all the elements within one partition have the same initial stress. Fatigue crack propagation is afterwards simulated using Sub 2. Note that Sub 1 has been built in the welding simulation step with coarse mesh than Sub 2 for the reason of reducing the total number of elements in the welding simulation step in the current study. This Sub 1 is used in the following steps.

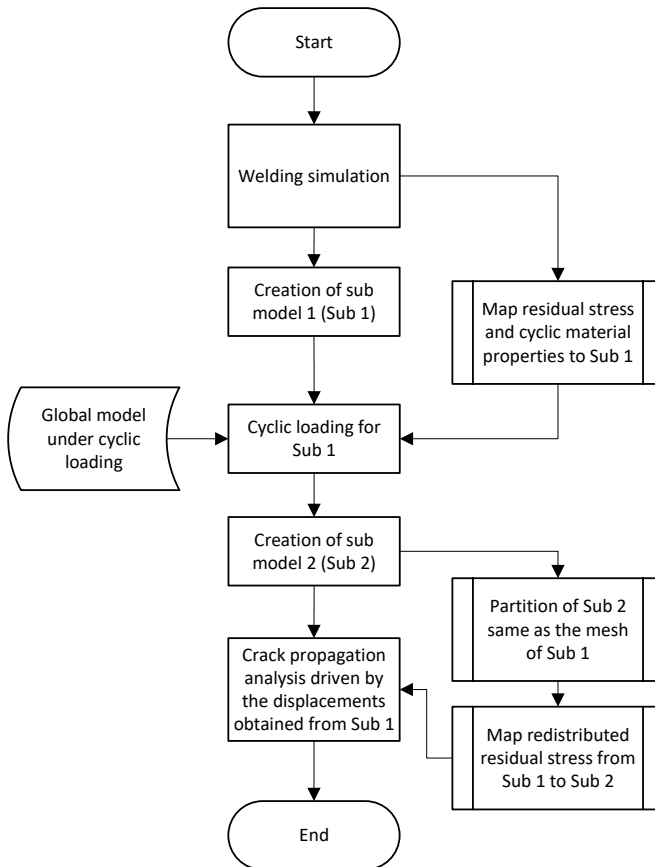


Figure 7.35: Flowchart of the numerical calculation of ΔJ for S5 S.

Sub 2 is introduced at lower end of the weld with a size of $6.30 \times 4.00 \times 3.06 \text{ mm}^3$, see Figure 7.36. A notch radius $R_{\text{ref}} = 1 \text{ mm}$ is designated at the weld toe, same as Sub 1. Sub 2 is placed in the middle of Sub 1 (Z-axis direction), and consists of the coarse and refined mesh zones which are meshed using 10-node quadratic tetrahedral elements (C3D10) and 8-node brick elements (C3D8) [179], respectively. The refined mesh zone has a size of $3.00 \times 3.00 \times 1.00 \text{ mm}^3$ with an approximate element size of $0.05 \times 0.05 \times 0.05 \text{ mm}^3$. The element sizes gradually change to $2.00 \times 2.00 \times 2.00 \text{ mm}^3$ in the coarse mesh zone. Spatial displacements in three dimensions are applied at the highlighted red intersections using the results from Sub 1 to simulate the cyclic loading. Fatigue cracking is assumed to initiate from the middle of Sub 2 in the Z-axis direction, with a distance of 0.25 mm away from the weld toe (half of the original element size). ΔJ is calculated using the domain integral method, see Appendix D.5 for the verification of the method. In Appendix D.5, excellent agreement of the J-integral calculations between the Abaqus embedded algorithm [179] and the author's post process code is observed from the fourth contour around the crack tip. Note that this comparison is carried out under the monotonic loading where the findings are assumed to suit ΔJ . Contour 4 is therefore used to calculate ΔJ in this section, see the highlighted white path as an example.

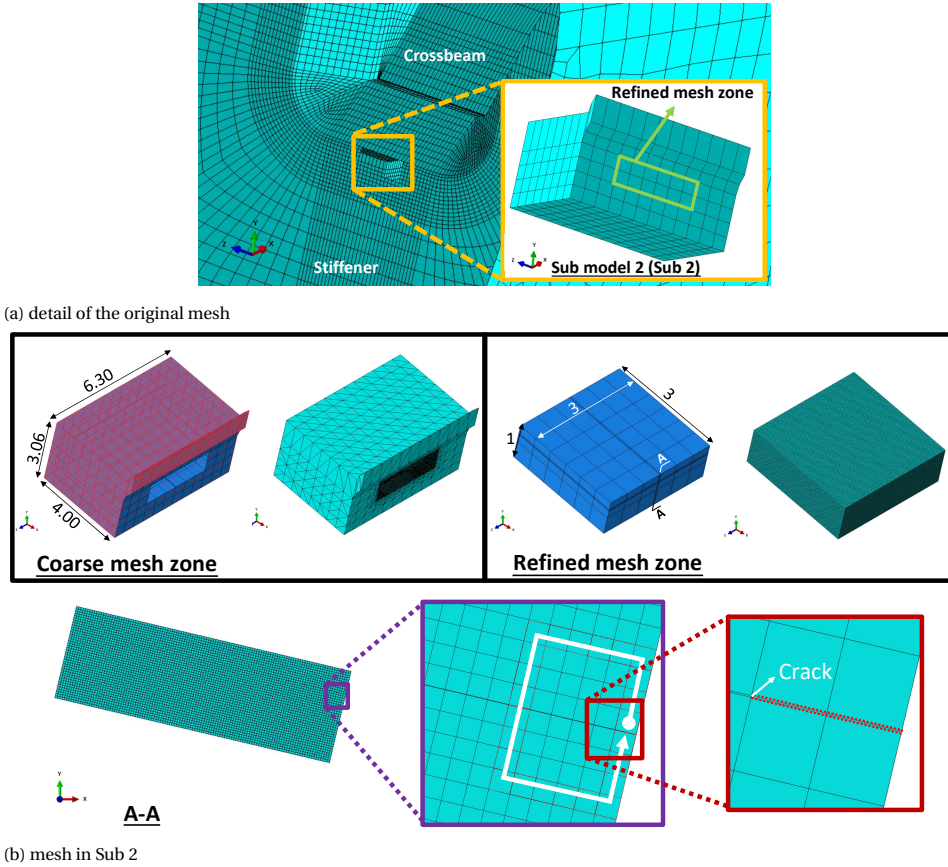


Figure 7.36: Mesh modification of the FE models for the ΔJ calculation of S5 S (unit: mm).

The crack propagation is modelled by removing the elements in the cracked region. The crack plane is assumed to be perpendicular to the stiffener surface with a semi-circular shape $a = c$ (crack depth = half crack length) [72, 229]. Figure 7.37 shows the triangular prism shape cracks modelled using hexahedron shape elements. The 5×10^{-4} mm thick elements are removed step-by-step (from $a = 0.05$ mm and $2c = 0.10$ mm to $a = 0.50$ mm and $2c = 1.00$ mm) to simulate the surface crack propagation. The crack propagates with increments $\Delta a = 0.05$ mm and $\Delta c = 0.10$ mm. In total 10 cyclic loading are created for each step. In each load cycle, 40 fixed increments are set. ΔJ is calculated using the procedure introduced in Section 7.7.2. To keep consistent with the stress intensity factor concept, ΔK_J is calculated based on ΔJ .

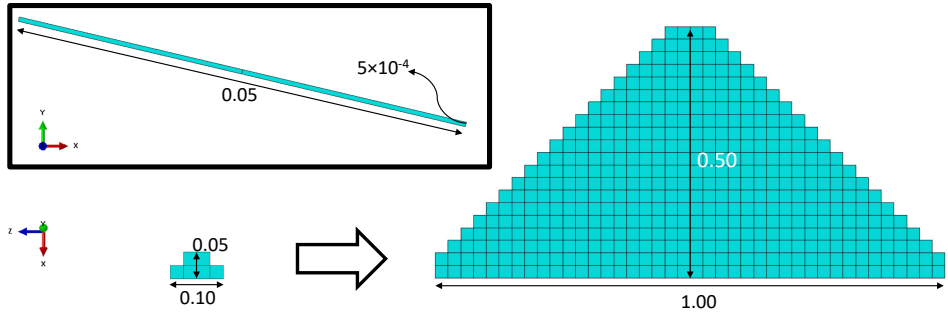


Figure 7.37: The crack shapes in the ΔJ calculation (unit: mm).

ΔK_I IN FATIGUE CRACK PROPAGATION

Figure 7.38 shows the values of ΔK_I with crack depths $a = 0.05$ mm to $a = 0.5$ mm under $\Delta F = 2 \times 150$ kN or 2×250 kN. ΔK_I grows rapidly when the crack depth increases from 0.05 mm to 0.1 mm, followed by a slower increase until 0.3 mm, and becomes almost constant when the depth is between 0.3 mm and 0.5 mm. Note that the ΔK_I calculated in both phases are close, with the ascend phase slightly larger than the descend phase.

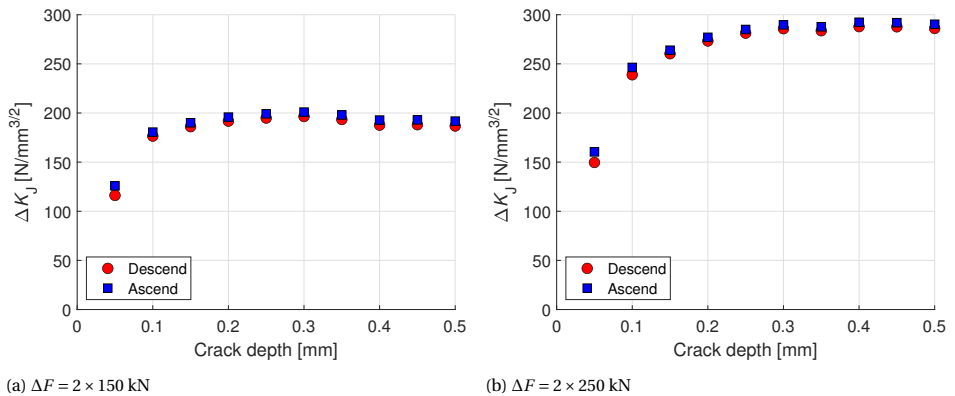


Figure 7.38: Crack depth ($a = 0.05$ mm to $a = 0.5$ mm) versus ΔK_I for S5 S under $\Delta F = 2 \times 150$ or 2×250 kN.

7.8. LONG CRACK PROPAGATION SIMULATION USING THE XFEM WITHOUT WRS

This section simulates the long crack propagation of S5 S using the XFEM, starting from the first measured crack in the experiment with a length of 10 mm. The stress intensity factors at the crack front are calculated by the XFEM assuming LEFM. The simulation starts with the first observed crack inspected by the naked eye. The crack is inserted into the (X)FE model. Afterwards, predefined load cycles are applied with the crack increment calculated by the Paris' equation using the explicit forward Euler method. When creating a new crack front, the curve fitting method is used to smooth the crack front as some jitter is observed in the XFE calculations.

7.8.1. FE MODEL FOR THE LONG CRACK PROPAGATION

The global model for the long crack propagation has the same loading and boundary conditions as described in Section 7.2.3. Figure 7.39 shows Regions 1, 2, and 3 created in the FE model. Region 1 is meshed with element type S8R. Regions 2 and 3 are meshed with element types C3D8R and C3D20R, respectively. The latter is meshed with refined elements to improve the accuracy of stress intensity factor calculations. Element sizes in the range from 100 mm to 0.25 mm can be found in the three regions as summarised in Table 7.5. “Tie” constraints [179] are used to connect the three regions. A change has been made for the local weld shape as shown in Figure 7.39. The weld leg length is measured and the (X)FE model is adapted to the measured shape. The inserted crack is 1 mm away from the weld toe.

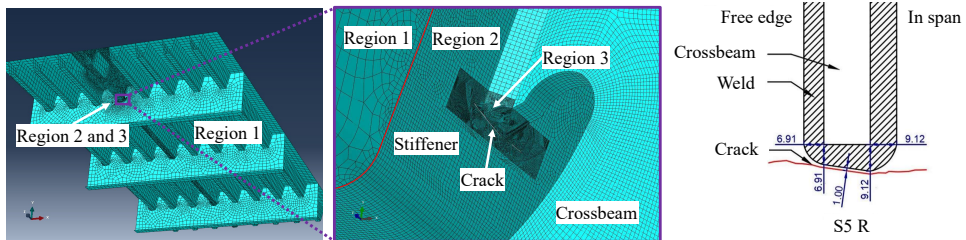


Figure 7.39: (X)FE model for the long crack propagation of S5 S (unit: mm).

Table 7.5: Element type and size in the (X)FE model [179].

Region	Element type	Element order	Approx. element size [mm]
1	Shell, S8R	2	100
2	Brick, C3D20R	2	2
3	Brick, C3D8R	1	0.25

Figure 7.40 compares the experimental measurement and the FE calculations at the lower end of S5 S. The legends with titles “global” and “sub” are the results of the models described in Figures 7.6 and 7.39, respectively. Note that in Figure 7.39, only the results of Region 3 are extracted and presented. Strains perpendicular to the weld are extracted at the lower end and two sides of the crossbeam web. Good agreement is observed for both the global model and the model containing local regions. Strains along three lines at the lower end are extracted from the latter. The strain ranges are close in these three lines.

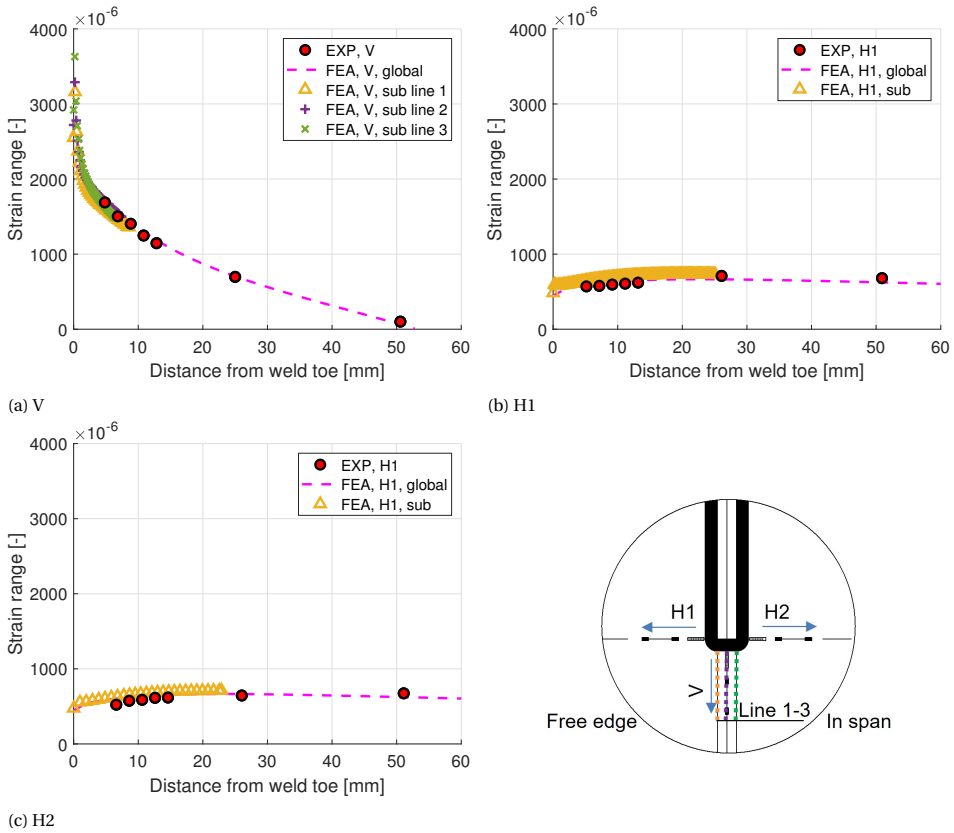


Figure 7.40: Comparison of the experimental measurement and the FE calculations for S5 S under $\Delta F = 2 \times 250$ kN.

7.8.2. DISCONTINUITY MODELLING AND CALCULATION OF STRESS INTENSITY FACTORS USING THE XFEM

Discontinuities in a FE model can be described by creating geometric discontinuities. Refined mesh with rings around the crack front is normally used to guarantee the accuracy of stress intensity factors calculation [191]. This causes the inconvenience when cracks need to be updated frequently. The XFEM was proposed in 1999 [234] and becomes popular as remesh is not needed in the XFEM for the update of cracks. En-

riched functions are added to the selected region with the jump function $H(x)$ across the crack surface and the associated elastic asymptotic crack tip function $F_\alpha(x)$, see Equations (7.12) to (7.14) [179]. Equation (7.14) is only suitable for linear elastic materials. Figure 7.41 gives an illustration of the cracked area, the node and element types in an XFE model. Three types of nodes and four types of elements are used in the calculation. In Abaqus, the enriched region can be directly selected in the definition of crack [179].

The Level Set Method (LSM) is used to define the crack surface and front in the XFEM as shown in Figure 7.42. Arbitrary crack geometry and its growth can therefore be described without remeshing. The crack surface and the orthogonal surface are described with $\phi = 0$ and $\Psi = 0$, respectively. The normal directions to crack surface and front are represented by $\mathbf{n}^+$ and $\mathbf{m}^+$, respectively.

K_I , K_{II} , and K_{III} can be calculated using the interaction integral method [179, 235]. The contour integral calculation can only be applied to first order brick elements. In Section 7.8.1, type C3D8R elements are applied in Region 3. Note that the accuracy of stress intensity factors depends on element sizes, crack paths, the number of contours *etc.* Based on previous experience [236], elements with sizes 0.2 to 0.3 mm and 10 contour integrals provide satisfactory results. The averaged values of contours 6 to 10 with an approximate element size of 0.25 mm in the enriched region are therefore used in this calculation.

$$\mathbf{u} = \sum_{i=1}^N N_i(x) \left[\mathbf{u}_i + H(x) \mathbf{a}_i + \sum_{\alpha=1}^4 F_\alpha(x) \mathbf{b}_i^\alpha \right] \quad (7.12)$$

$$H(x) = \begin{cases} 1 & \text{if } (\mathbf{x} - \mathbf{x}^*) \cdot \mathbf{n} \geq 0, \\ -1 & \text{otherwise.} \end{cases} \quad (7.13)$$

$$F_\alpha(x) = \left[\sqrt{r} \sin\left(\frac{\theta}{2}\right), \sqrt{r} \cos\left(\frac{\theta}{2}\right), \sqrt{r} \sin(\theta) \sin\left(\frac{\theta}{2}\right), \sqrt{r} \sin(\theta) \cos\left(\frac{\theta}{2}\right) \right] \quad (7.14)$$

Where:

$\mathbf{u}$: displacement vector function.

$N_i(x)$: the usual nodal shape functions.

$H(x)$: discontinuous jump function across the crack surfaces.

$\mathbf{b}_i^\alpha$, $F_\alpha(x)$: the nodal enriched degree of freedom vector and the associated elastic asymptotic crack tip functions.

$\mathbf{x}$, $\mathbf{x}^*$, $\mathbf{n}$: $\mathbf{n}$ is a Gauss point in a cracked element, $\mathbf{x}^*$ is the point on the crack closest to $\mathbf{n}$,

and $\mathbf{n}$ is the unit outward normal to the crack at $\mathbf{x}^*$.

(r, θ) : polar coordinate system with its origin at the crack tip with $\theta = 0$ tangent to the crack at the tip.

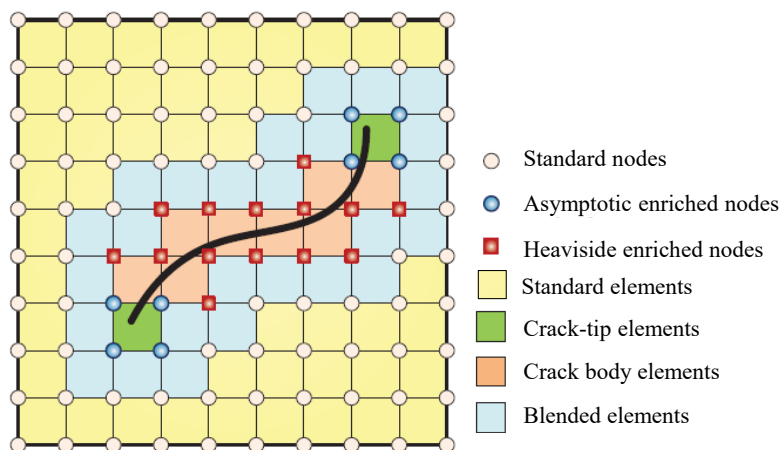


Figure 7.41: Illustration of the node and element types in the enriched region of XFEM, an example of the cracked area with enriched nodes and elements [237].

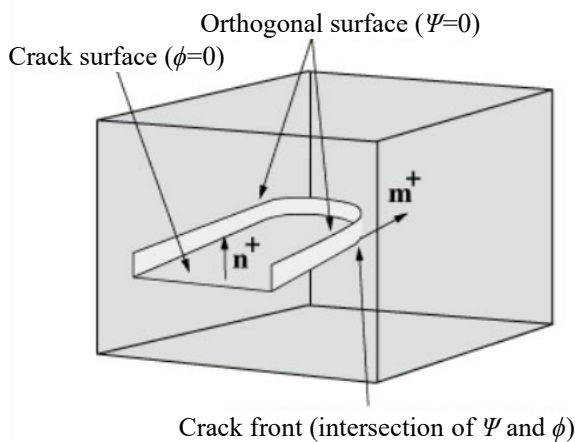


Figure 7.42: Schematic representation of a three-dimensional non-planar crack [179].

7.8.3. CRACK PROPAGATION SIMULATION USING THE XFEM

The stationary crack insertion and the stress intensity factors calculation can be achieved using the XFEM function in Abaqus [179]. However, the three-dimensional fatigue crack propagation using the Paris' equation is not available in the employed 2019 version of Abaqus [179]. An "in-house" Python code is therefore developed to update cracks under cyclic loading using the Paris' equation. Figure 7.43 presents the flowchart of the user-defined crack propagation procedure.

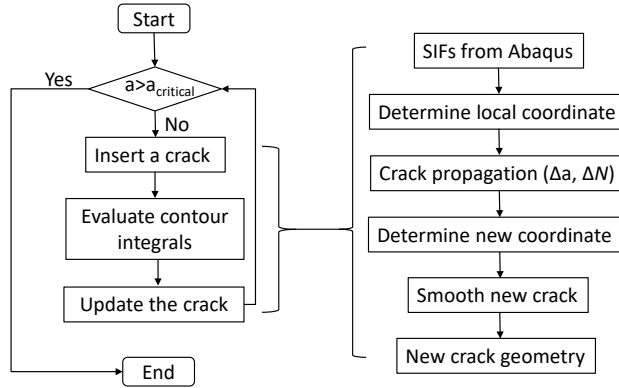


Figure 7.43: Flowchart of the long crack propagation calculation using Abaqus with the "in-house" Python code ("a" can be either crack depth or length).

When the crack size does not exceed the critical value, the crack is inserted into the XFE model. The stress intensity factors along the crack front are calculated using the XFEM. The number of cycles in the increment, ΔN , is predefined. The corresponding crack front increment, Δa , is obtained following the Paris' equation. The details of the crack update on the right of the flowchart are illustrated in Figure 7.44 using semi-elliptical crack as an example.

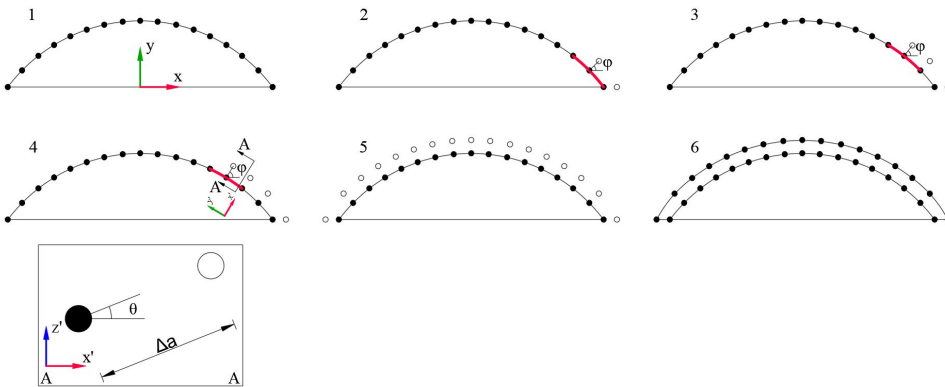


Figure 7.44: Illustration of a semi-elliptical crack update using the "in-house" Python code.

Six steps applied in the update are shown below:

- Step 1: The values of the global coordinates for all dots along the crack are obtained.
- Steps 2 to 4: The extension values for each dot in the global coordinate system (X, Y, Z) are calculated using the values transferred from the local coordinate system (X', Y', Z').
- Step 5: The locations of the dots are updated as shown in the empty circles.
- Step 6: The new crack is obtained after smoothing the crack front.

This paragraph describes the crack propagation calculation using the Paris' equation. The increment of the crack front at a point can be described using Equation (7.15). The crack propagation is divided into small increments. An explicit or implicit Euler method is used in the increment. Non-conservative and conservative results may be predicted based on the explicit and the implicit extrapolation, respectively. The effective stress intensity factor range ΔK_{eff} is calculated by Equation (7.16) [238]. The Maximum Tangential Stress (MTS) criterion available in Abaqus is used to calculate the angle of the three dimensional crack propagation $\hat{\theta}$ at the crack front, see Equation (7.17) [179]. Note that the calculated $\hat{\theta}$ is based on the local coordinate system at the crack front. A transfer from $\hat{\theta}$ to the crack extension length Δa and the angle in the global system φ needs to be carried out for the crack propagation of the whole crack front.

$$\Delta a_k = \Delta N_k \cdot C \cdot (\Delta K_{\text{eff}, k/(k+1)})^m \quad (7.15)$$

$$\Delta K_{\text{eff}} = \sqrt[4]{(\Delta K_I)^4 + 8(\Delta K_{II})^4} \quad (7.16)$$

$$\hat{\theta} = \cos^{-1} \left(\frac{3K_{II}^2 + \sqrt{K_I^4 + 8K_I^2 K_{II}^2}}{K_I^2 + 9K_{II}^2} \right) \quad (7.17)$$

Where:

C, m : material properties in the Paris' equation.

Δa_k : crack increment at step k.

ΔN_k : number of cycles responsible for Δa_k .

$\Delta K_{\text{eff}, k/(k+1)}$: effective stress intensity factor range at the crack front of step k or k+1.

ΔK_I : stress intensity factor (range) corresponding to the mode I crack.

ΔK_{II} : stress intensity factor (range) corresponding to the mode II crack.

$\hat{\theta}$: crack propagation angle calculated based on the local coordinate system.

7.8.4. PREDICTION OF THE LONG CRACK PROPAGATION FOR S5 S

This section shows the XFEM predictions for S5 S. The calculation starts from the first measured crack 1'-1" and ends with a surface crack approximately 50 mm long. The explicit Euler method is applied in the calculation with the increment of the number of cycles in each step defined ranging from 5,000 to 10,000. Material properties: $C = 3.98 \times 10^{-13}$ (in millimetre and Newton) and $m = 2.88$, are used [24]. It is assumed that the crack is sufficiently long and propagates in the base steel.

Figure 7.45 compares the crack path of S5 S predicted by the XFEM and measured from the experiment. The initial crack length is defined at points 1" and 1' with a depth of 1 mm in the stiffener thickness direction. Good match of the crack path is found between the XFEM prediction and the experimental measurement. In total, 19 steps are calculated with numerical values summarised in Table 7.6.

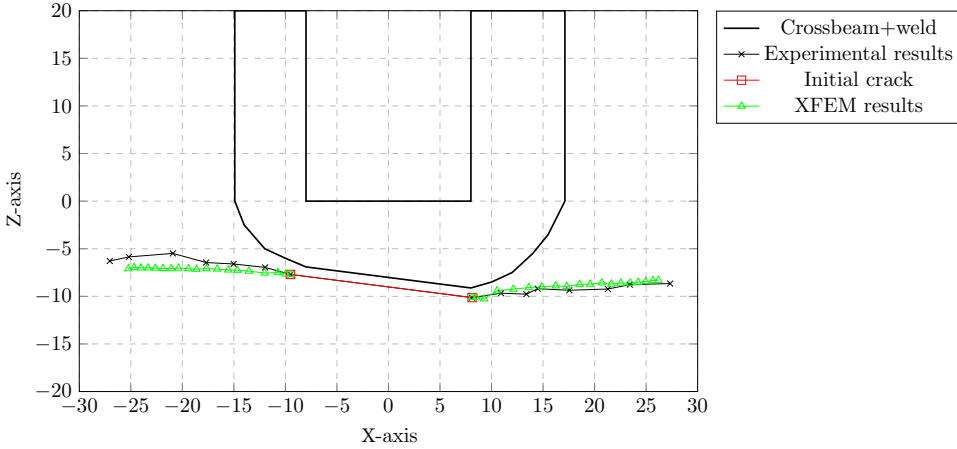


Figure 7.45: Comparison of the XFEM prediction and the experimental measurement for the crack path of S5 S (unit: mm).

Figure 7.46 shows the three dimensional crack shapes predicted by the XFEM in the top and the isometric views where the coordinate system is defined as in Figure 7.45. In the Y-axis direction, a relatively quick increment is found from step 0 to step 3. The growth rate slows down afterwards. When the step numbers are larger than 10, slow increments can be seen in the depth direction. For the crack length (in the X-axis direction), a similar trend is observed, whereas the difference in the propagation rate is smaller. The crack rotates towards the horizontal direction (in the X-Y plane) during the propagation period, see Figures 7.45 and 7.46b.

Figure 7.47 shows the fatigue life calculation based on the Paris' equation. Various values of C are used to check the possible effect of the material properties on the fatigue life. The predefined value $C = 3.98 \times 10^{-13}$ gives a conservative prediction, whereas the results of $C = 2.00 \times 10^{-13}$ match the experiment well. Note that the crack propagation rate of the base steel is experimentally obtained using the compact tension specimens manufactured from the stiffener web of the full-scale specimen, see Appendix C.2. The compact tension experiments are loaded in fatigue with the load ratio $R = 0.1$ or $R = 0.5$, resulting in $2.00 \times 10^{-13} \leq C \leq 3.00 \times 10^{-13}$ with $R = 0.1$. This indicates that the long crack propagation of S5 S is independent of WRS as the propagation rate is close to the values with $R = 0.1$.

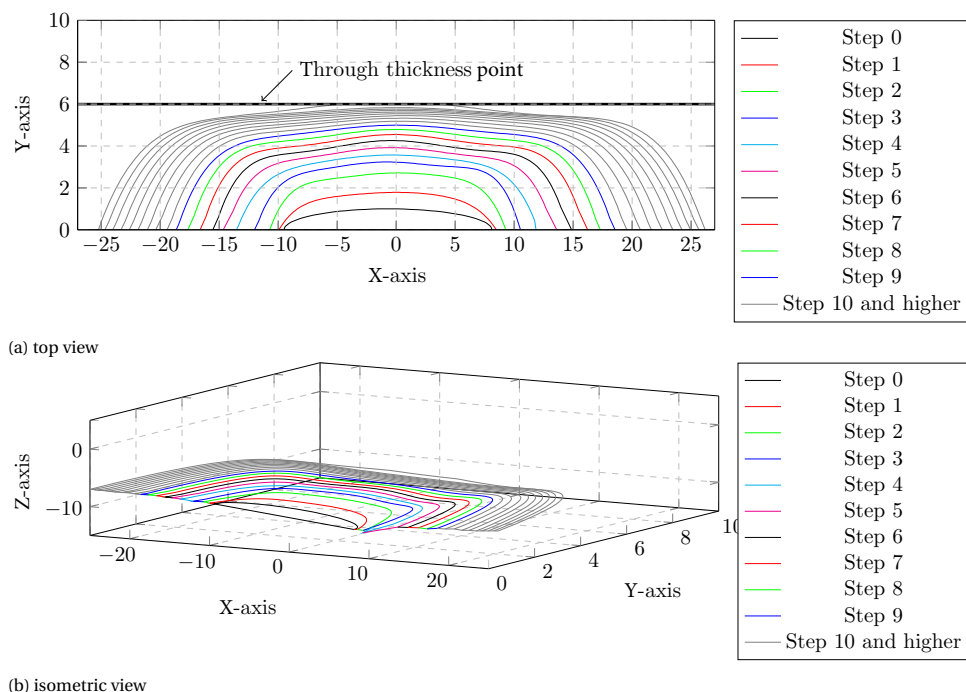


Figure 7.46: Three dimensional crack shapes predicted by the XFEM for S5 S (unit: mm).

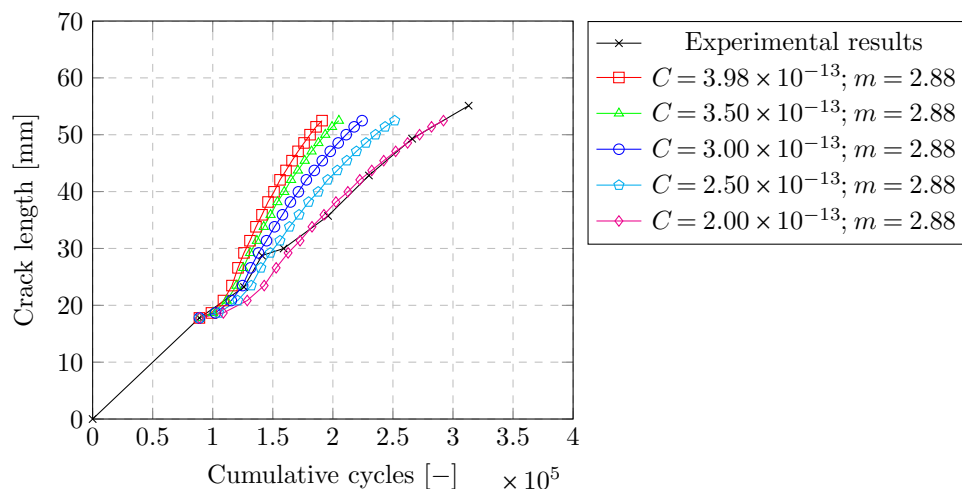


Figure 7.47: Effect of the material parameter C on the fatigue crack prediction for S5 S.

Table 7.6: Crack increment in the XFEM prediction for S5 S (Δa is the crack increment at surface).

Step	ΔN	N	Δa [mm]	Crack length [mm]
0	-	88,900	-	17.78
1	10,000	98,900	0.88	18.66
2	10,000	108,900	2.18	20.84
3	7,100	116,000	2.63	23.47
4	5,000	121,000	3.12	26.59
5	5,000	126,000	2.63	29.22
6	5,000	131,000	2.14	31.36
7	5,000	136,000	2.45	33.81
8	5,000	141,000	2.11	35.92
9	5,000	146,000	2.25	38.17
10	5,000	151,000	1.79	39.96
11	5,000	156,000	2.11	42.07
12	5,000	161,000	1.65	43.72
13	5,000	166,000	1.72	45.44
14	5,000	171,000	1.64	47.08
15	5,000	176,000	1.48	48.56
16	5,000	181,000	1.45	50.01
17	5,000	186,000	1.38	51.39
18	5,000	191,000	1.1	52.48

7.9. 2⁺ STAGE MODEL

BACKGROUND

Sections 7.7 and 7.8 study the crack initiation and the short crack propagation considering WRS, and the long crack propagation without WRS, respectively. The NSA in conjunction with the $\epsilon - N$ curve, and the fracture mechanics (either EPFM or LEFM) in conjunction with the (modified) Paris' equation are applied, respectively. Three phases are, therefore, defined as shown in Figure 7.48:

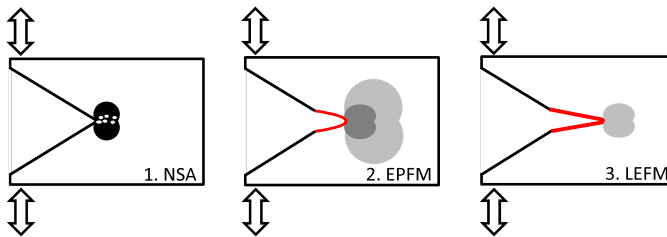


Figure 7.48: Illustration of the three phases and the corresponding methods in fatigue analysis.

1. The crack initiation prediction using the NSA method. The damage of the first grain to several grains size is designated as the crack initiation.

2. The short crack propagation analysis with relatively large-scale plasticity in front the crack tip using the EPFM and the modified Paris' equation. The WRS and cyclic material properties at the studied region should be defined, *e.g.* the material model of HAZ.
3. The long crack propagation analysis starts with the small-scale yield assumption at the crack tip using the LEFM and the Paris' equation. The WRS may be included if it is significant.

INTRODUCTION OF THE 2⁺ STAGE MODEL

A 2⁺ stage model is therefore proposed as a general methodology for realistic (accurate) fatigue life prediction covering from the crack initiation to the long crack propagation. The 2⁺ stage model is an improvement of the 2 stage model [71]. The 2 stage model assumes a crack depth to connect the NSA and the LEFM, which has the limits in mechanics by nature of being specific. Full range fatigue analysis can be carried out based on the three phases. The (numerical) framework for the application of the 2⁺ stage model for (but not limited to) welds is summarised as shown in Figure 7.49:

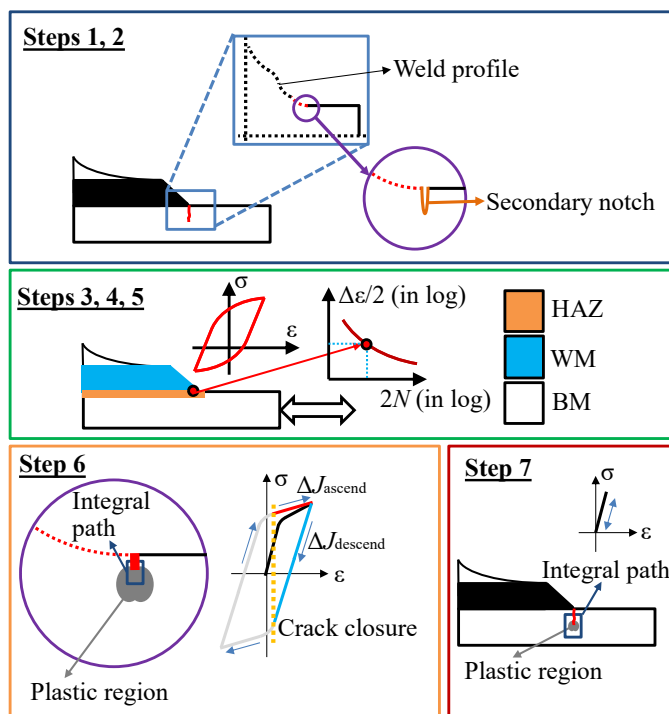


Figure 7.49: Illustration of the flowchart for the 2⁺ stage model.

Step 1. Identify the geometric characteristics, initial stress states and notches at the critical location, *e.g.* the weld toe.

- Step 2. Create the FE model including the local characteristics and execute the welding simulation to predict WRS.
- Step 3. Identify the HAZ, BM, and WM, and apply the corresponding material properties to these three distinct regions.
- Step 4. Apply cyclic loading and obtain $\Delta\epsilon$ at the critical (crack initiation) location, implicit FEA, considering the calculated WRS (element-to-element map the results of Step 2 to the cyclic loaded model) using different constitutive models for the HAZ, BM, and WM.
- Step 5. Predict the crack initiation using the stabilised $\Delta\epsilon$ at the critical location and the $\epsilon - N$ curve.
- Step 6. Execute the short crack propagation calculation using the EPFM with ΔJ considering the redistributed WRS (stabilised after the cyclic loading in Step 4). The crack closure can be considered by ΔJ_{eff} using the calculated stresses and strains when the crack is open.
- Step 7. Execute the long crack propagation calculation using the LEFM with ΔK until the fracture criterion is reached. The WRS can be considered either by the weight function method or excluded when the crack is sufficiently long and away from the weld. The stress ratio and the overloading can be considered, *e.g.* by the Walker's correction [76] and the Wheeler's model [77, 78].

The 2⁺ stage model is developed to give a framework for fatigue analysis, by addressing the most relevant parts in a crack lifetime. The methodology is checked and gives insights to the full range of fatigue analysis. The 2⁺ stage model accurately shows the strain states at the critical location. The crack initiation prediction is based on the realistic material performance of the detail. If the secondary notch is negligibly small, the NSA method should be used to predict the grain scale cracking. If the secondary notch is detected and cannot be ignored, the prediction should start from the short crack, where the EPFM with ΔJ is adopted. In both cases, WRS should be implemented and the accurate cyclic elastic-plastic material properties should be defined in the corresponding regions. With the crack growth, the small-scale yield assumption at the crack tip becomes valid and the LEFM is used for the long crack propagation until failure. As a practical solution, the WRS may be considered by the weight function method. An alternative option is to exclude the WRS if it is sufficiently redistributed and does not affect the crack propagation. Note that the accuracy of prediction will be improved with detailed geometric characteristics of the weld and material properties.

The 2⁺ stage model overcomes the limits of the ENS method where the WRS and material properties in welds are uniformly treated for all cases.

The welding simulation provides quantitative WRS based on the identified region that is affected by high temperature. Boundaries of HAZ, BM, and WM regions can therefore be obtained. The cyclic elastic-plastic material properties are used for the regions with the WRS quantitatively implemented into the model. The realistic material response under cyclic loading is studied to obtain the stabilised stresses and strains in

the cyclic loading for the crack initiation prediction. This is not considered by the 4R method [66], LEFM method [23, 24, 64], UniGrow model [68–70], and 2 stage model [71].

The EPFM on one hand covers the situations where secondary notches cannot be ignored, and also bridges the NSA method (crack initiation) and the LEFM method (long crack propagation). It overcomes the limits of using an assumed crack depth used in the 2 stage model to connect the crack initiation and propagation. The EPFM studies the behaviour of short crack with WRS implemented in order to connect the crack initiation to the long crack propagation. The effect of mean stress and the crack closure in the short crack propagation stage can be considered by the EPFM.

A complete framework is thus established for the full range fatigue analysis, starting from the crack initiation until final failure with a smooth transition considered using the EPFM.

7.10. SUMMARY OF THE CHAPTER

Examination of the fatigue behaviour of the lower end of the weld in the connection between the crossbeam and stiffener with cut-out is presented in this chapter. This construction detail was not included in either the first or the second generation of (pr)EN 1993-1-9 [16, 17]. Eight independent experiments obtained from one full-scale specimen are loaded in fatigue. The fatigue resistances are determined based on the experiments in the current study and 27 other experiments found in the literature. The study has led to the following conclusions and recommendations:

- The bias in fatigue resistance between different series is small for most evaluated cases, despite the significantly different specimen geometries applied in different series. This confirms that the hot spot stress can be used as the basis for evaluating the fatigue resistance of the detail of study for a wide range of geometries.
- Relatively slow crack growth is observed for this detail, which is attributed to the crack growing away from the weld, *i.e.* towards a region with smaller stress-raising effect. This implies that the fatigue resistance from the experiments depends significantly on the criterion used for evaluation. The current study evaluates the fatigue resistance for the criteria ranging between the first deviation of the strain ranges measured with strain gauges and a crack length of approximately 50 mm. The corresponding characteristic fatigue resistance values of hot spot stress, defined with the 95% exceedance fraction at 2 million cycles, range between 76 MPa and 166 MPa, respectively.
- For the design of new bridge decks, the author considers a crack that is first visually observed as a proper evaluation criterion for fatigue resistance, as a balance between full structural integrity (not compromising the load-bearing resistance) and relatively high fatigue resistance (allowing for relatively low weight of the components). The associated fatigue resistance is **112 MPa** (rounded-down value), resulting from the 14 available experiments for this criterion.
- An additional S-N curve is proposed for assessing existing bridge decks. The author considers a surface crack with an approximate length of 50 mm appropriate

as a criterion for this case because it corresponds with a reasonable probability of the detection of surface crack with the naked eye [125]. The associated fatigue resistance of **160 MPa** resulting from 10 experiments is relatively high. This implies that most existing bridge decks will pass the assessment for this detail.

- In the geometry analysed in the current study, the configuration includes 6 mm thick trapezoidal closed stiffeners with Haibach shape cut-outs and 16 mm thick crossbeam webs, the maximum hot spot stress range obtained at detail C3a for the highest wheel load applied to a super single tyre of the Fatigue Load Mode 4 in EN 1991-2 [80] is approximately 63 MPa. Using the S-N curve format of Eurocodes, the constant amplitude fatigue limit is 83 MPa and the cut-off value is 45 MPa for the proposed design resistance curve. These values do not account for the partial factors. The maximum stress range of Fatigue Load Model 4 is between these values. This implies that a modified geometry than the one studied here might be required for very heavily loaded bridge decks, whereas the design is expected to be sufficient for more common, moderately loaded bridge decks.
- The effect of weld flank angle is studied using the effective notch stress. The effective notch stress with a flank angle $\theta = 30^\circ$ is approximately 18% smaller than the value with a flank angle $\theta = 45^\circ$. The effective notch stress is almost equal between $\theta = 45^\circ$ and 90° . Within the requirements of prEN 1993-1-9 [17] ($30^\circ \leq \theta \leq 60^\circ$), the effect of weld flank angle on fatigue resistance is hence limited.
- The predicted fatigue resistance for the hot spot stress, as derived from the effective notch stress method ranges from 97 MPa to 115 MPa. This prediction is close to the statistical analysis of the experimental results at the onset of cracking, which ranges from 94 MPa (for the criterion of 25% drop of the strain range at 4 mm distance from the weld toe) to 117 MPa (first visible crack).
- The finite element model gives welding-induced residual stress perpendicular to the weld toe up to approximately 550 MPa at the outer side of the stiffener, which drops to 300 MPa to 400 MPa at the inner side. At the lower end of the weld, the region in front of the weld toe experiencing a maximum temperature exceeding 750°C during the welding has a size of $2\text{ mm} \times 1\text{ mm}$ (the stiffener thickness direction $\times$ the direction perpendicular to the weld toe at surface).
- The notch strain method accurately predicts the fatigue life of the author's three experiments with the earliest failure detection criterion of either 10% or 25% deviation in the strain range at 4 mm distance from the weld toe, or first visible crack.
- The ΔJ based stress intensity factor range (ΔK_I) is successfully applied to S5 S detail with the crack depth a increasing from 0.05 mm to 0.5 mm. ΔK_I grows rapidly with a increasing from 0.05 mm to 0.1 mm, then increases with a less steep slope until $a = 0.3\text{ mm}$, and afterwards keeps almost constant until $a = 0.5\text{ mm}$.
- The long crack propagation without residual stress can be predicted using the extended finite element method available in Abaqus [179] in combination with the

Python code. An approximately linear relation with the cumulative number of cycles is found for the crack length propagating from 10 mm to 52 mm. The extended finite element method predicted fatigue lives are approximately 20% lower than the experimentally measured ones. Possible reasons are: the assumption of initial crack depth, the simulation of local fillet weld shape and the effect of rolling direction on fatigue propagation rate. It is found that the effect of residual stress on the long crack propagation of the studied crack is limited.

- A 2⁺ stage model is proposed by the author based on the full range fatigue analysis using the notch strain method for the crack initiation, the elastic-plastic fracture mechanics for the short crack propagation, and the linear elastic fracture mechanics for the long crack propagation using the finite element method as an improved model of the 2 stage model [71]. The welding-induced residual stress is quantitatively implemented into the cyclic loaded model element by element using the results from the welding simulation. The realistic material properties of heat affected zone, base metal, and weld metal should be used to calculate the strain range at the critical location for the crack initiation with potential stress and strain redistribution taken into consideration. The elastic-plastic fracture mechanics using ΔJ with the modified Paris' equation is used for the short crack propagation with the (redistributed) welding residual stress. Finally, the long crack propagation is analysed using the linear elastic fracture mechanics with the (improved) Paris' equation. The residual stress may be taken into consideration by the weight function method or ignored if it is sufficiently released. The 2⁺ stage model is able to realistically study the state of crack initiation considering the welding residual stress and the weld toe radius. An accurate (smooth) transition between the crack initiation and the long crack propagation is obtained using the elastic-plastic fracture mechanics instead of the sharp transition using an assumed crack depth in the 2 stage model [71].

8

CONCLUSIONS AND RECOMMENDATIONS

8.1. CONCLUSIONS

This dissertation studies the fatigue behaviour of six types of welded connections in modern Orthotropic Bridge Decks (OBDs) mainly based on a full-scale specimen with a 20 mm thick deck plate. Additional experiments on single connections ($350 \times 200 \times 200 \text{ mm}^3$) are carried out for stiffener-to-deck plate weld between crossbeams. The Hot Spot Stress (HSS) is used to predict the resistance of the weld toe crack or the weld root crack that propagates into the base metal. The Nominal Stress (NS) is used for the weld root crack in double-sided welded connection that propagates into the weld throat. The Structural Stress (SS) calculated from the axial force and the bending moment is used for the root crack in single-sided welded connection. Higher fatigue resistances than the standardised value of 100 MPa (weld toe) or 40 MPa (weld root) in prEN 1993-1-9: 2023 [17] are obtained for the studied details. This is mainly due to combinations of steep stress gradients towards the hot spots, the possibility to redistribute loads from a weakened component to adjacent parts, loaded in cyclic compression or out-of-plane bending or in a combination of the two, relatively thin plate thickness of stiffener, and strict requirements for the geometry of the weld between stiffener and deck plate. Using the author's experiments and the results in the literature, detail categories are established for design purposes, which were used as input for the Technical Specification TS 1993-1-901 "Fatigue design of orthotropic bridge decks with the hot spot stress method" [27]. In addition to the main failure criterion of the First Visible Crack (FVC), various fatigue criteria are used in this study to provide a flexible framework for the life-cycle assessment of OBDs, each resulting in a set of S-N curves for each studied detail. The effects of plate thickness, Degree Of Bending (DOB), Partial Joint Penetration (PJP), fatigue crack propagation rate, crack arrest, and weld profile on the fatigue resistance or behaviour are analysed using the local (stress, strain, or energy) methods or the Linear Elastic Fracture Mechanics (LEFM).

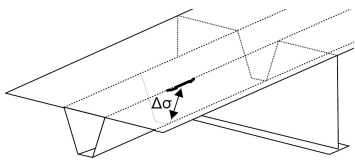
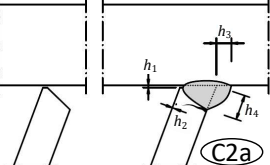
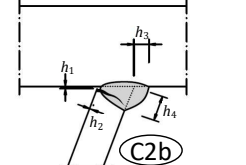
Welding simulation is carried out for one detail. The Welding Residual Stress (WRS) obtained from Finite Element (FE) simulations is studied and implemented for fatigue life predictions. The Notch StrAin (NSA) is used to predict the crack initiation, which agrees with the experimental results. The Elastic-Plastic Fracture Mechanics (EPFM) is adopted for the short crack where the small-scale yield assumption is invalid. Finally, the framework of the 2^+ stage model is established in combination of the NSA, EPFM, and LEFM for spanning all stages of fatigue life. An overview of the applied methods for the studied details can be found in Figure 1.4. The main conclusions are summarised in Sections 8.1.1 and 8.1.2.

8.1.1. FATIGUE RESISTANCES OF THE STUDIED DETAILS

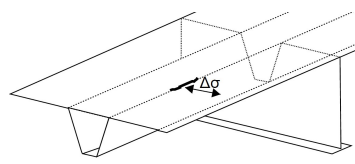
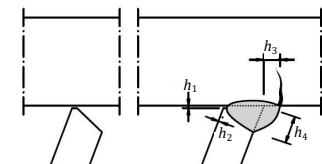
The results of this study were used as input for prTS 1993-1-901 [27]. Table 8.1 presents modified tables included in prTS 1993-1-901 of the details studied in this dissertation. The HSS method is used for details C2a, C1a, C1c, and C3a, the SS method is used for detail C2b, and the NS method is used for detail C4d. Table 8.1 summarises the fatigue resistances obtained from this study and the corresponding detail categories in prTS 1993-1-901 [27]. The characteristic fatigue resistances $\Delta\sigma_{95\%}$ (95% exceedance fraction at 2 million cycles) are obtained by statistical analysis with the fixed slope parameter $m = 3$ (fully informative prior), in line with (pr)EN 1993-1-9 [16, 17]. The standardised

characteristic fatigue resistances $\Delta\sigma_C$ are the values used in prTS 1993-1-901 [27].

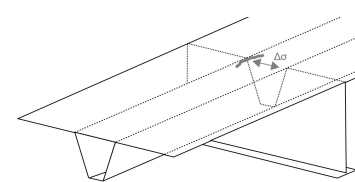
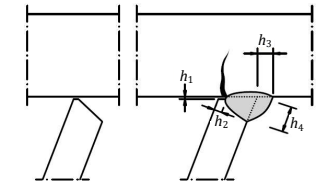
Detail	Description	$\Delta\sigma_C$	N_D	N_L	m_1	m_2	Detail category requirements
C2a	Stiffener-to-deck plate weld, weld toe crack in stiffener	160	5×10^6	10^8	3	5	Automatic or (overhead) manual weld
C2b	Stiffener-to-deck plate weld, weld root crack in weld	140 100	10×10^6 10×10^6	10^8 10^8	3 3	5 5	Automatic weld (overhead) manual weld

Detail	Description	$\Delta\sigma_C$	N_D	N_L	m_1	m_2	Detail category requirements
C1a	Stiffener-to-deck plate weld, weld toe crack in deck plate	125 140	5×10^6 5×10^6	10^8 10^8	3 3	5 5	$t_d > 14\text{mm}$ $10\text{mm} \leq t_d \leq 14\text{mm}$

Detail	Description	$\Delta\sigma_C$	N_D	N_L	m_1	m_2	Detail category requirements
C1c	Stiffener-to-deck plate weld, weld root crack in deck plate at crossbeam	170	30×10^6	10^8	3	5	$t_d \leq 14\text{mm}$

Execution requirements		
Fit between plates:	either	$h_1 = 0 \text{ mm}$ for $\geq 90\%$ of weld length; and $h_1 < 0.5 \text{ mm}$ for $\leq 10\%$ of weld length;
	or	$h_1 < 2 \text{ mm}$.
Lack of penetration		$h_2 = 1,0 \text{ mm} \pm 0,5 \text{ mm}$
Extension of weld		$h_3 = t_s \pm 1 \text{ mm}$
		$h_4 = t_s \pm 1 \text{ mm}$
Weld geometry	Melt through at inner side of the stiffener is not allowed. Smooth weld toe transitions required.	
Preparation	Chamfer stiffener web.	

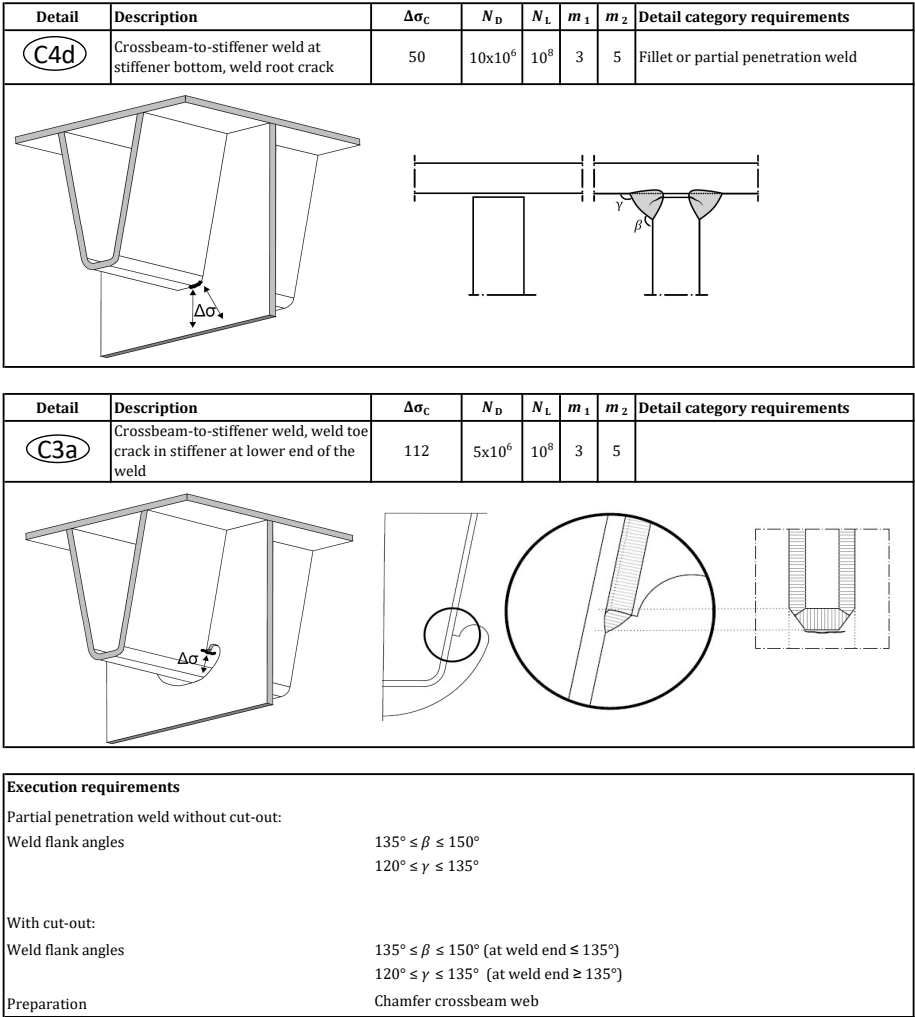


Figure 8.1: Detail categories studied in this dissertation, reprinted in a modified version from prTS 1993-1-901 [27].

Table 8.1: Summary of the fatigue resistances obtained from this study and the detail categories in prTS 1993-1-901 [27] (unit: mm or MPa).

Detail	Method	Equation	Source	$\Delta\sigma_{95\%}$	$\Delta\sigma_C$ in [27]
C2a	HSS	(8.1)	Figure 3.13a	158	160
C2b (automatic weld)	SS	(8.2)	Figure 3.13b	158	140
C2b (manual weld)	SS	(8.2)	Figure 3.11b	106	100
C1a ($14 \leq t_d \leq 30$)	HSS	(8.1)	Figure 4.23	141-125	125
C1a ($10 \leq t_d \leq 14$)	HSS	(8.1)	Figure 4.23	149-141	140
C1c ($10 \leq t_d \leq 12$)	HSS	(8.1)	Figure 5.15c	167	170
C4d	NS	(8.3)-(8.5)	Figure 6.21c	50	50
C3a	HSS	(8.1)	Figure 7.13c	117	112

$$\Delta\sigma_{hs} = 1.5 \cdot \Delta\sigma_{0.5t} - 0.5 \cdot \Delta\sigma_{1.5t} \quad (8.1)$$

$$\Delta\sigma_{wf} = \frac{\Delta F}{b \cdot a_w} + \frac{6}{b \cdot a_w^2} (\Delta M + \Delta F \cdot e) \quad (8.2)$$

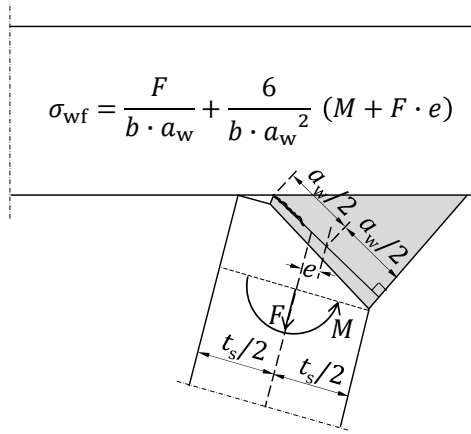


Figure 8.2: Illustration in prTS 1993-1-901 [27] for defining the SS in single-sided partial penetration welded connection.

$$\Delta\sigma_{w,n} = \Delta\sigma_{b,n} + \Delta\sigma_{m,n} \quad (8.3)$$

where:

$$\Delta\sigma_{b,n} = \frac{\Delta M}{(t_w + t_{cb}) t_w b} \quad (8.4)$$

$$\Delta\sigma_{m,n} = \frac{\Delta F}{2 t_w b} \quad (8.5)$$

The variables t , a_w , t_w and b are the thickness of plate (deck plate t_d , stiffener t_s , or crossbeam t_{cb}), the thickness of weld throat in single-sided partial penetration welded connection, the effective thickness of weld throat in welded connection, and the width of detail used in calculation, respectively. The eccentricity, e , can be calculated using a_w and t_s as shown in Figure 8.2.

The HSS range $\Delta\sigma_{hs}$ is calculated by the linear extrapolation of the stresses normal to the weld from locations $0.5t$ and $1.5t$ away from the hot spot (weld toe) to the hot spot, see Equation (8.1). The SS range in single-sided partial penetration welded connection $\Delta\sigma_{wf}$ is calculated by Equation (8.2) using the force pair of the bending moment range ΔM and the axial force range ΔF at the weld root, which are obtained from FE simulations. The NS range of weld throat $\Delta\sigma_{w,n}$ consists of the nominal bending stress range $\Delta\sigma_{b,n}$ and the membrane stress range $\Delta\sigma_{m,n}$ both in the weld throat, see Equations (8.3) to (8.5).

8.1.2. FATIGUE PERFORMANCE OF THE STUDIED DETAILS

DECK PLATE THICKNESS

The deck plate thickness not only affects the stress values at the crack initiation location but can also influence the crack propagation path. The LEFM predicts a linear decrease of the fatigue resistance as a function of the deck plate thickness for detail C1a. Fatigue resistance of 26 mm thick deck plate (equal to 125 MPa) is 0.89 of 14 mm thick deck plate (141 MPa). For detail C1c, the fatigue resistances associated with 10% or 25% deviation in the strain range are insensitive to deck plate thickness, whereas the crack propagation behaviour heavily depends on deck plate thickness. Most of the author's experiments run out with the FVC failure criterion for the 20 mm thick deck plate of the full-scale specimen. The eXtended Finite Element Method (XFEM) is used to predict the crack propagation path with deck plate thicknesses of 10 mm, 12 mm, 16 mm, and 20 mm. Through-thickness crack occurs in the 10 mm thick deck plate. Crack arrest starts with the deck plate thickness of 12 mm where the final crack is very close to the top surface and can break through at high load levels. For the deck plates thicker than 12 mm, after reaching 75% of the total deck plate thickness, the crack stops propagating in the thickness direction. This situation results in a very high fatigue resistance for the deck plate thicker than 12 mm when the FVC is adopted. However, the deck plate has no influence on the characteristic fatigue resistances of details C2a and C2b with the deck plate thickness ranging from 12 mm to 20 mm based on the author's small-scale experiments and the results in the literature. It should be clarified that the deformation of stiffener web is affected by the thickness of deck plate in practice; therefore, the fatigue lives of details C2a and C2b are related to the deck plate thickness (larger resistance for larger deck plate thickness).

WELDING PROCESS

The effect of welding process on fatigue depends on the studied details. Stiffener-to-deck plate welded connections ($350 \times 200 \times 200 \text{ mm}^3$) are prepared using either two-pass automatic welding or one-pass manual welding with PJPs between 75% and 100%. The stiffener webs are cyclically loaded in bending and the deck plate is fixed at the upper deck

side. Both C2a- and C2b-type cracks are observed in the experiments with a significantly higher fatigue resistance observed in the automatically welded connections. Detail C2a has a HSS characteristic fatigue resistance of 272 MPa (standard deviation $s = 0.09$) or 159 MPa ($s = 0.14$) for automatically or manually welded connections, respectively. The values are 158 MPa ($s = 0.29$) or 106 MPa ($s = 0.22$) for detail C2b. However, detail C1c shows low sensitivity to welding process in the full-scale experiments. The difference in the HSS at 2 million cycles with the survival probability $P_s = 50\%$ is within 20% (detected by the 10% or 25% deviation in the strain range) between the automatic weld and the manual weld.

PARTIAL JOINT PENETRATION

The effect of PJP on the crack initiation location and the fatigue resistance of details C2a and C2b is studied based on the plane strain FE models using the Effective Notch Stress (ENS) with a notch radius $R_{\text{ref}} = 1.00$ mm or 0.05 mm, the Strain Energy Density (SED) factor of the averaged strain energy over (part of) a circle with a radius $R_{\text{ref}} = 0.32$ mm, and the LEFM with an initial crack depth $a_i = 0.1$ mm. An increase of PJP from 25% to 75% significantly reduces the ENS and SED for details C2a and C2b. For PJPs exceeding 75%, detail C2a is predicted as critical and no obvious change of the ENS and SED is observed. For detail C1c, FE models are built using a symmetric model with a size of $1300 \times 900 \times 1000$ mm³, simplified from the full-scale specimen. The ENS with $R_{\text{ref}} = 1.00$ mm is applied with PJPs between 75% and 100%, which appears to provide similar stress values, indicating that detail C1c is insensitive to PJP.

WELD PROFILE

Despite the standardised straight weld profile with the flank angle of 45°, allowable flank angles in prEN 1993-1-9: 2023 [17] are between 30° and 60°. In this study, flank angles up to 90° are considered to account for the most extreme weld profile in case of manual (overhead) welding and the flank angle of 0° as the other extreme case to consider grinding as a post-weld treatment option. Within the requirements of [17], the effect of weld flank angle on the fatigue resistance of detail C3a is limited as predicted by the ENS. The ENS with the flank angle of 30° is approximately 18% smaller than the value with the flange angle of 45°, resulting in a 16% higher fatigue resistance. The ENS is almost equal with flank angles between 45° and 90°, indicating insensitivity of the fatigue resistance of this detail to this flank angle range. This finding is confirmed by the LEFM predictions for detail C1a. An up to 25% higher fatigue resistance is observed for the flank angle of 0° compared with 45°, whereas the difference between the 45° and 90° cases is within 4%.

FAILURE CRITERION

The FVC is the main failure criterion used for the full-scale experiments in this study. In addition, a certain change (normally 10% or 25%) of the strain range close to the crack initiation location and a certain length (normally 25 or 50 mm long) of surface crack are used as the early and practical detection criteria, respectively.

Detail C1c is of special interest as the FVC failure criterion is only obtained for the deck plate thickness of 10 or 12 mm with a characteristic fatigue resistance of 167 MPa, whereas the values are 86 MPa and 117 MPa for the 10% and 25% change of the strain

range at the top of deck plate (above the crack initiation location) for 10, 12 and 20 mm thick deck plates.

Detail C4d shows a relatively slow crack propagation until a surface crack length of approximately 50 mm. The cracking process consists of four stages:

- Stage 1: Fatigue cracks initiate from the weld root and propagate into the weld throat.
- Stage 2: Fatigue cracks propagate on the weld surface. Short cracks merge into long ones on the weld surface.
- Stage 3: The main fatigue crack propagates to the crossbeam and grows as a surface crack.
- Stage 4: The surface crack on the crossbeam propagates through the crossbeam web and propagates as a through-thickness crack.

The fatigue performance (with 50% exceedance fraction at 2 million cycles) based on Stage 4 with a surface crack length of 50 mm is approximately 1.4 of the values based on Stage 1 or 2.

Relatively slow crack growth is also observed in detail C3a as the crack growing away from the weld. The recommended characteristic fatigue resistance is 112 MPa (rounded down from 117 MPa) for fatigue design using the FVC failure criterion. However, a 50 mm long surface crack failure criterion is recommended for existing bridge decks considering the probability of detection. This results in a characteristic fatigue resistance of 160 MPa (rounded down from 166 MPa).

Details C2a and C2b are studied using small-scale experiments. The failure (rupture) of specimen is designated as the failure criterion. The fatigue resistance is independent of the failure criterion for the small-scale specimens.

LOAD RATIO

High compressive stresses are induced by the load in many details in OBDs. Some details are loaded even completely in compression. In the full-scale experiments, a pulsating pressure load is applied on a tyre patch of realistic dimensions. In this study, the induced stresses at the crack initiation location of details C1c and C4d are in nominal compression. The load ratio applied in the small-scale experiments is taken as $R = -1$ for details C2a, C2b, and C1a in order to be closer to the applied load ratio in real OBDs.

DEGREE OF BENDING

The DOB is close to 1 for details C2a, C2b, C1a, C1c, and C3a. The DOB of approximately 0.5 is found for detail C4d in the author's experiments. This is a possible explanation of the relatively high fatigue resistance in these details compared with the standardised 100 MPa or 40 MPa, because the bending stress is less detrimental to the fatigue performance than the membrane tensile stress.

WELDING RESIDUAL STRESS

Welding simulation is carried out using a validated uncoupled thermal-mechanical FE model for detail C3a. The WRS perpendicular to the weld toe is predicted to be up to approximately 550 MPa (yield stress approximately 380 MPa at 20°C) at the outer side of the stiffener, which drops to the range between 300 MPa and 400 MPa at the inner side. The WRS redistributes at the first load cycle.

COMPARISON OF FATIGUE PREDICTIONS BASED ON EXISTING METHODS AND THE EXPERIMENTS

When assuming that the crack is closed under the external compressive stress for details C2a and C2b, the LEFM with initial crack depth $a_i = 0.1$ mm gives excellent fatigue resistance predictions for detail C2a (automatic or manual welding) with an overestimate within 5% using $C = 3.00 \times 10^{-13}$ (units in millimetre and Newton) and $m = 3.00$. The LEFM with $C = 5.21 \times 10^{-13}$ (units in millimetre and Newton) and $m = 3.00$ is useful for automatically welded C2b with PJPs ranging from 75% to 100%. The initial crack size assumption of $a_i = 0.1$ mm may lead to a 29% overestimation of the predicted resistance for manually welded C2b. The SED method also gives good predictions for detail C2a, whereas the ENS method gives conservative predictions by up to 18% or 30% with $R_{\text{ref}} = 0.05$ mm or 1.00 mm, respectively. For detail C2b, assuming a PJP of 75%, the fatigue resistance prediction with the ENS is suitable for manually welded connections. The fatigue resistances predicted by the SED method for the weld toe crack (type C2a) with $\text{PJP} \geq 75\%$ are within 5% of the experimentally obtained fatigue resistance, whereas the SED method is considered unsuited for this type of weld root crack (type C2b) because of the large over-predictions compared with the experimental results.

The predicted fatigue resistances for detail C3a, as derived from the ENS method, range from 97 MPa to 115 MPa. They are close to the experimental results at the onset of cracking, which range from 94 MPa (the criterion of 25% drop of the strain range at 4 mm distance from the weld toe) to 117 MPa (the criterion of the FVC inspected by the naked eye).

The probabilistic fracture mechanics simulation using the linear propagation model within the LEFM framework with the initial crack depth $a_i = 0.15$ mm and the initial crack shape $a_i/c_i = 0.62$ (initial crack depth/initial half crack length) gives excellent predictions for the fatigue resistance of detail C1a loaded under $R = -1$ in small-scale experiments, which are within 5% difference with the experimental results.

2⁺ STAGE MODEL FOR FULL RANGE FATIGUE ANALYSIS

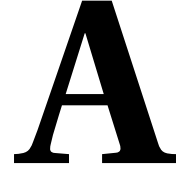
The author proposes a 2⁺ stage model based on the full range fatigue analysis using the NSA for the crack initiation, the EPFM for the short crack propagation, and the LEFM for the long crack propagation, all with the (X)FE method. This model is developed as an improvement of the 2 stage model proposed in [71]. In this study, detail C3a is analysed as an example in five steps. First, welding simulation is carried out for the studied detail. Second, the regions of heat affected zone, base metal and weld metal in the weld are defined. The corresponding constitutive models are assigned to these regions. Third, cyclic loading is applied with WRS implemented element-by-element (same mesh as the second step) and the crack initiation is predicted by the NSA using the $\epsilon - N$ curve. Fourth, the short crack propagation is predicted by the EPFM with the redistributed WRS using

a refined mesh sub model driven by the model in the third step. Fifth, the long crack propagation is calculated using the LEFM where WRS can be considered by the weight function or excluded when the crack is sufficiently long, so that WRS is released.

8.2. RECOMMENDATIONS FOR FUTURE WORK

A comprehensive study of the fatigue behaviour of six types of cracks in OBDs has been carried out. Recommendations for future work are given as follows:

1. Fatigue resistance curves are established using a primary failure criterion for each detail. The detection threshold of practical inspection limits may differ from the criterion used in this study. The various failure criteria presented in Section 8.1 were used, but fracture mechanics models should be used if it is needed to predict crack propagation.
2. Fatigue evaluation of the root crack in a partial penetration welded connection using the ENS and the SED needs to be improved. The realistic notch characteristics measured from specimens should be used instead of the fictitious notch radius recommended by guidelines when analysing the fatigue behaviour of the studied specimens. A comparison of the predictions between the fictitious notch radius recommended by guidelines *e.g.* 1 mm and the measured notch radii helps to evaluate the suitability of the fictitious notch radius for the studied detail.
3. The LEFM is a useful method for the fatigue resistance and the crack propagation prediction of welded connections. However, the prediction is affected by the uncertainties caused by initial crack sizes, short crack propagation, crack closure, residual stress, applied stress ratio, and interaction of multiple cracks, *etc.* The construction details and their stress states in OBDs may differ from standardised specimens. Calibrations of the LEFM model against the experiments specified for OBDs are recommended.
4. The crack path and the arrest of crack have been qualitatively analysed using the XFEM for detail C1c. The analysis of the crack size and the corresponding load cycles need to be carried out. A suitable failure criterion should be developed for detail C1c.
5. Studies are carried out on detail C4d with fillet welds, but the partial penetration welded detail C4d also needs to be analysed.
6. Welding simulation is carried out for detail C3a. The simulation procedure is validated against the measurements in a fillet welded T-joint. It is recommended to measure the WRS in the studied detail. The welding analysis for other details should be carried out and the optimisation of welding procedure is recommended.
7. The framework of the 2^+ stage model has been established, whereas the material properties are assumed. The ϵ -N curves for crack initiation at the crack initiation location, *e.g.* heat affected zone, considering the statistical distribution should be developed. Material parameters of the ΔJ -based fatigue propagation model for the short crack propagation analysis should be identified by experiments. Application of the 2^+ stage model to variable amplitude loading is interesting to study.



STATISTICAL ANALYSIS PROCEDURE

The characteristic value of the stress range at 2 million cycles is given as $\Delta\sigma_C$ with one-sided survival probability of 95% at a two-sided 75% confidence level in (pr)EN 1993-1-9 [16, 17] and prTS1993-1-901 [27] for fatigue resistance analysis. A fixed slope $m = 3$ is used for connections under direct stress. A summary of the statistical calculation procedure following EN 1990 [177] is presented below:

$$y_i = a_N + b \cdot x_i + \varepsilon \quad (\text{A.1})$$

where:

$$b = \frac{n \cdot \sum(x_i \cdot y_i) - \sum x_i \cdot \sum y_i}{n \cdot \sum x_i^2 - (\sum x_i)^2} \quad (\text{A.2})$$

$$a_N = \frac{1}{n} \cdot (\sum y_i - b \cdot \sum x_i) \quad (\text{A.3})$$

The stress range with the survival probability of 50% at 2 million cycles can be obtained using Equations (A.4) to (A.6):

$$\sigma_{50\%} = 10^{x_{50\%}} \quad (\text{A.4})$$

where:

$$x_{50\%} = \frac{y_{50\%} - a_N}{b} \quad (\text{A.5})$$

$$y_{50\%} = \log(2 \times 10^6) \quad (\text{A.6})$$

The stress range with the survival probability of 95% at 2 million cycles can be obtained using Equations (A.7) to (A.13):

$$\sigma_{95\%} = 10^{x_{95\%}} \quad (\text{A.7})$$

where:

$$x_{95\%} = \frac{y_{50\%} - a_{95\%}}{b} \quad (\text{A.8})$$

$$a_{95\%} = y_{95\%} - b \cdot x_{50\%} \quad (\text{A.9})$$

in which:

$$y_{95\%} = y_{50\%} - t_{0.95, n-k} (k=1,2) \cdot S \cdot \sqrt{1 + \frac{1}{n} + \frac{(x_{50\%} - \bar{x})^2}{S_{xx}}} \quad (\text{A.10})$$

$$S^2 = \begin{cases} \frac{\sum [y_i - (a_N + b \cdot x_i)]^2}{n-2} & \text{when } b \text{ is calculated,} \\ \frac{\sum [y_i - (a_N + b \cdot x_i)]^2}{n-1} & \text{when } b \text{ is fixed.} \end{cases} \quad (\text{A.11})$$

$$\bar{x} = \sum \frac{x_i}{n} \quad (\text{A.12})$$

$$S_{xx} = \sum x_i^2 - n \cdot (\bar{x})^2 \quad (\text{A.13})$$

Definitions of the variables and constants:

y_i : logarithm of the number of cycles to failure N_f .

x_i : logarithm of the applied stress range $\Delta\sigma$.

a_N : intercept on the axis of $\log(N_f)$.

n : number of experimental results included in statistical analysis.

b : regression coefficient.

ε : sum of unknown random errors.

$\Delta\sigma_{50\%}$: stress range at 2 million cycles with the survival probability of 50%.

$\Delta\sigma_{95\%}$: stress range at 2 million cycles with the survival probability of 95%.

S : sample standard deviation.

$t_{0.95, n-k} (k=1,2)$: Student's t-distribution with the one-sided survival probability of 95%;

$k = 1$ or 2 is applied for the fixed or the calculated b , respectively.

$\bar{x}$: mean value of x_i .

S_{xx} : sum of squares about the mean.

The statistical analysis for fatigue detail classification requires the regression coefficient b equal to the slope of the S-N curve $-m$, which is set as $b = -m = -3$ for direct stress. Only the experimental results that fail with the number of cycles not exceeding the constant amplitude fatigue limit value N_D are included in the statistical analysis.

B

ADDITIONAL EQUATIONS RELATED TO CHAPTER 4

The empirical equations for the mode I stress intensity factor K_I along an elliptical shape surface crack (depth a and half length c) in a finite-width plate (width b and thickness t) were proposed by Newman and Raju [192] as shown below:

$$K_I = (M_m \sigma_m + M_b \sigma_b) \sqrt{\frac{\pi a}{Q}} \quad (\text{B.1})$$

where

$$Q = 1.0 + 1.464 \left(\frac{a}{c}\right)^{1.65} \left(\frac{a}{c} \leq 1\right) \quad (\text{B.2})$$

The membrane correction factor:

$$M_m = [M_1 + M_2 \left(\frac{a}{t}\right)^2 + M_3 \left(\frac{a}{t}\right)^4] g f_\phi f_w \quad (\text{B.3})$$

where

$$M_1 = 1.13 - 0.09 \frac{a}{c} \quad (\text{B.4})$$

$$M_2 = \frac{0.89}{0.2 + a/c} - 0.54 \quad (\text{B.5})$$

$$M_3 = 0.5 - \frac{1.0}{0.65 + a/c} + 14 \left(1 - \frac{a}{c}\right)^{24} \quad (\text{B.6})$$

with

$$g = 1 + [0.1 + 0.35 \left(\frac{a}{c}\right)^2] [1 - \sin(\phi)]^2 \quad (\text{B.7})$$

where ϕ is the parametric angle of the ellipse.

The angle function for the embedded elliptical crack:

$$f_\phi = \left[\left(\frac{a}{c}\right)^2 \cos(\phi)^2 + \sin(\phi)^2 \right]^{0.25} \quad (\text{B.8})$$

The finite-width correction function:

$$f_w = [\sec(\frac{\pi c}{2b} \sqrt{\frac{a}{t}})]^{0.5} \quad (\text{B.9})$$

The bending correction factor:

$$M_b = M_m H \quad (\text{B.10})$$

where the function H is developed by curve fitting using

$$H = H_1 + (H_2 - H_1) \sin(\phi)^q \quad (\text{B.11})$$

with

$$H_1 = 1 - 0.34 \frac{a}{t} - 0.11 \left(\frac{a}{c}\right) \left(\frac{a}{t}\right) \quad (\text{B.12})$$

$$H_2 = 1 + G_1 \frac{a}{t} + G_2 \left(\frac{a}{t}\right)^2 \quad (\text{B.13})$$

$$q = 0.2 + \frac{a}{c} + 0.6 \frac{a}{t} \quad (\text{B.14})$$

in which

$$G_1 = -1.22 - 0.12 \frac{a}{c} \quad (\text{B.15})$$

$$G_2 = 0.55 - 1.05 \left(\frac{a}{c}\right)^{0.75} + 0.47 \left(\frac{a}{c}\right)^{1.5} \quad (\text{B.16})$$

C

ADDITIONAL EXPERIMENTS

C.1. ADDITIONAL CRACKS OBSERVED IN THE EXPERIMENTS FOR DETAIL C1C

In addition to C1c, three other types of cracks are detected, as shown in Figure C.1. Fatigue cracks are found at the bottom of the deck plate, in the weld, and in the stiffener (Figure C.1a), at the bottom of the deck plate and in the weld (Figure C.1b), and at the lower part of the stiffener-to-crossbeam connection (Figure C.1c).

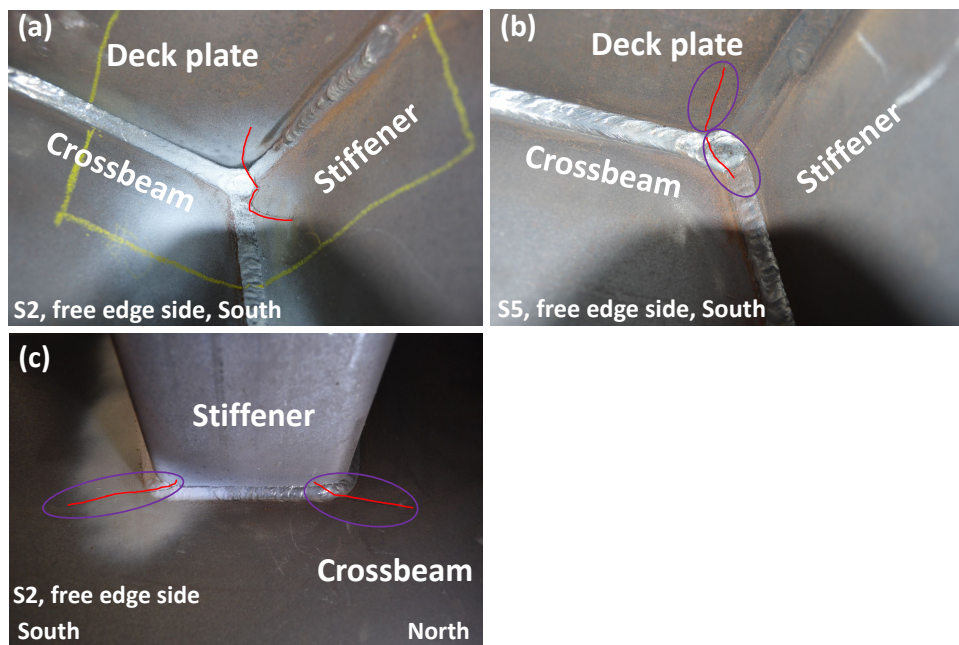


Figure C.1: Fatigue cracks besides type C1c observed from the bottom of the deck plate and the lower part of the stiffener-to-crossbeam connection (three photos taken at the end of Phase 2 loading).

A summary of the deck plate cracks besides C1c detected at the bottom of the deck plate is presented in Table C.1. S1 and S8 had been cut for other studies before the inspections were performed. Therefore, no information about the cracks at the deck plate is available for S1 and S8. Fatigue cracks are found at the lower part stiffener-to-crossbeam connections of S1 S, S2 S, and S2 N at both free edge and in span sides. It is unknown whether the first two types of cracks are secondary (*i.e.* occurring as a result of the C1c type cracks).

Table C.1: Summary of fatigue cracks besides C1c observed from the bottom of the deck plate at the end of Phase 2 loading.

	S1		S2		S3		S4		S5		S6		S7		S8	
	S	N	S	N	S	N	S	N	S	N	S	N	S	N	S	N
Free edge	-	-	×		×	×	×		×	×		×			-	-
In span	-	-	×				×	×	×	×		×	×	×	-	-

C.2. FATIGUE CRACK PROPAGATION PROPERTIES OF THE STIFFENER WEB

Material properties used in the Paris' equation are obtained using Compact Tension (CT) specimens following ASTM E647-15 [239], see Figure C.2. Fatigue crack propagation of the 6 mm thick stiffener web of the full-scale specimen is obtained using the 4 mm thick CT specimens. Cyclic loads with a nominal stress range of 150 MPa and a load ratio $R = 0.1$ or 0.5 (2 specimens for each load ratio) are applied via the steel pins. The crack length is counted from the notch to the fatigue crack front which is measured by the Direct Currents Potential Drop (DCPD) technique.

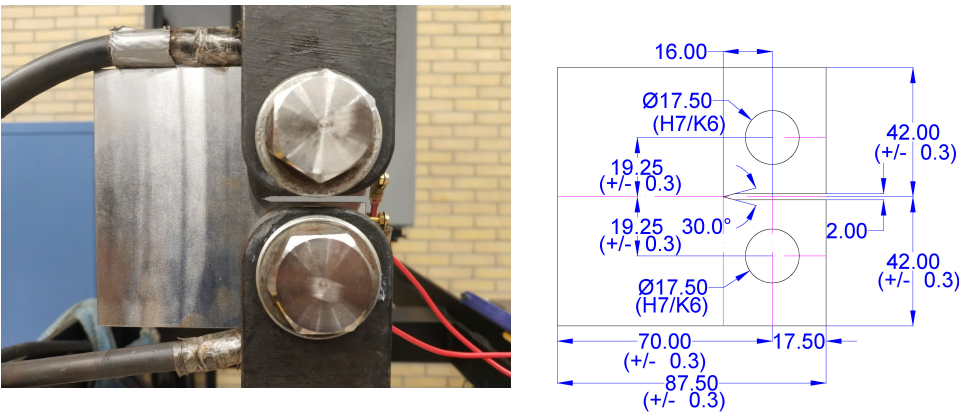


Figure C.2: Experimental setup and dimensions of the 4 mm thick CT specimens manufactured from the 6 mm thick stiffener web (unit: mm).

Figure C.3 shows the crack propagation measured from the experiments and predicted by the analytical equations available in [239]. The predictions use different C , whereas m is fixed to 2.88. Among them, the experiments with the load ratio $R = 0.1$ lies between the predictions with $C = 2.5 \times 10^{-13}$ or $C = 3.0 \times 10^{-13}$ (units in millimetre and Newton). Higher propagation rates are observed when $R = 0.5$. Note that the crack propagation direction, *i.e.* the angle of the crack and the rolling direction, may cause an approximately 20 % deviation [240] in the propagation rate, which is not further discussed in this study.

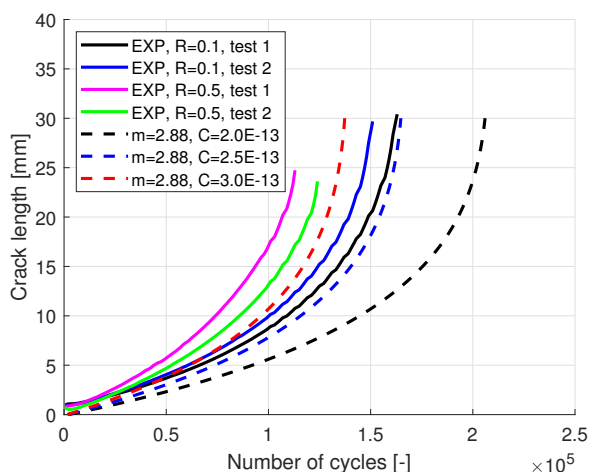


Figure C.3: Comparison of the experiments and the predictions using the analytical equations available in [239].

D

ADDITIONAL NUMERICAL ANALYSIS

D.1. MONTE CARLO SIMULATION WITH THE BILINEAR PROPAGATION MODEL FOR DETAIL C1a

Monte Carlo simulations for detail C1a with the bilinear propagation model available in [24] are carried out in this section. Table D.1 summarises the variables and probabilistic functions used in the simulations. In the bilinear propagation model, a transition point is set at ΔK_k . Different material properties $C_{a(c)}^1$ and $m_{a(c)}^1$, and $C_{a(c)}^2$ and $m_{a(c)}^2$ are assigned for $\Delta K \leq \Delta K_k$ (Stage A), and $\Delta K > \Delta K_k$ (Stage B), respectively. Considering the applied load ratio $R = -1$ and the existence of high residual stress at the surface of detail C1a, the values of $R \leq 0.5$ and $R > 0.5$ are used for the propagations in the crack depth and the crack length, respectively. Other parameters are the same as those in Table 4.3.

Table D.1: Variables and probabilistic functions for Monte Carlo simulations with the bilinear propagation model (units: mm and N).

Variable	Density function	Mean value	CoV
a_0	Lognormal	0.15 [64]	0.66 [64]
a_0/c_0	Lognormal	0.62 [64]	0.40 [64]
C_a^1	Lognormal	1.21×10^{-26} [24]	0.98
C_c^1	Lognormal	4.80×10^{-18} [24]	0.98
m_a^1	Deterministic	8.16 [24]	-
m_c^1	Deterministic	5.10 [24]	-
C_a^2	Lognormal	3.98×10^{-13} [24]	0.98
C_c^2	Lognormal	5.86×10^{-13} [24]	0.98
m_a^2	Deterministic	2.88 [24]	-
m_c^2	Deterministic	2.88 [24]	-
$\Delta K_{k,a}$	Deterministic	363.00 [24]	-
$\Delta K_{k,c}$	Deterministic	196.00 [24]	-
ΔK_{th}	Lognormal	170.00 [126]	0.20 [126]
B_{hs}	Lognormal	1.00 [197]	0.05 [197]
K_{mat}	Weibull	2250.00 [198]	0.25 [198]

Statistical analysis is carried out using either the generated samples by Monte Carlo simulations with the bilinear propagation model or the experimentally obtained results, as shown in Figure D.1. The statistical results show excellent agreement between the Monte Carlo simulations and the experiments. The predicted stress ranges at 2 million cycles with $P_s = 95\%$ and $P_s = 50\%$ (95% and 50% exceedance fractions) are 150 MPa and 203 MPa, respectively. The differences of the values between the Monte Carlo simulations and the experiments are within 7%. The predicted stress at 2 million cycles with $P_s = 5\%$ (5% exceedance fraction) is 273 MPa, exactly same as the value based on the experiments.

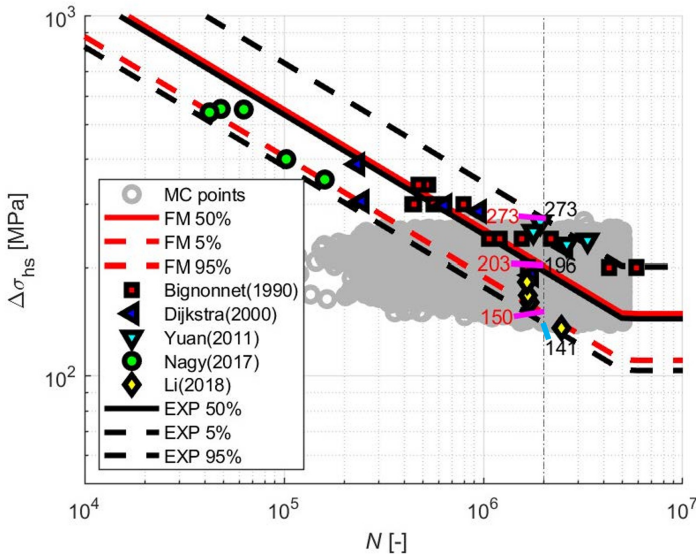


Figure D.1: Statistical analysis for detail C1a using Monte Carlo generated samples with the bilinear propagation model and the experimental results, both with $t_d = 10, 12, 15, \text{ or } 19 \text{ mm}$ [107, 110, 121, 127, 166].

D.2. FE CALCULATED STRESS DISTRIBUTION OF S2 IN CHAPTER 6

Figure D.2 shows the FE calculated stress in the direction perpendicular to the weld toe at the bottom of S2 under the normalised load $F = 1$ kN. Stress concentrations are observed when approaching the weld toe with higher values at the corners, as shown in Figure 6.6. Following the method designated for the weld toe in TS [27], σ_{hs} are approximately -1.3 MPa and -0.8 MPa at the corners at the free edge side and in span side, respectively. At the Middle position (M), between the North (N) and the South (S), σ_{hs} are -0.7 MPa at the free edge side and -0.6 MPa at the in span side. Under the normalised load $F = 1$ kN, the stresses in S3 and S4 are very close to the values in S2, whereas smaller values (in absolute) are observed in S1. These values are not plotted in the current study.

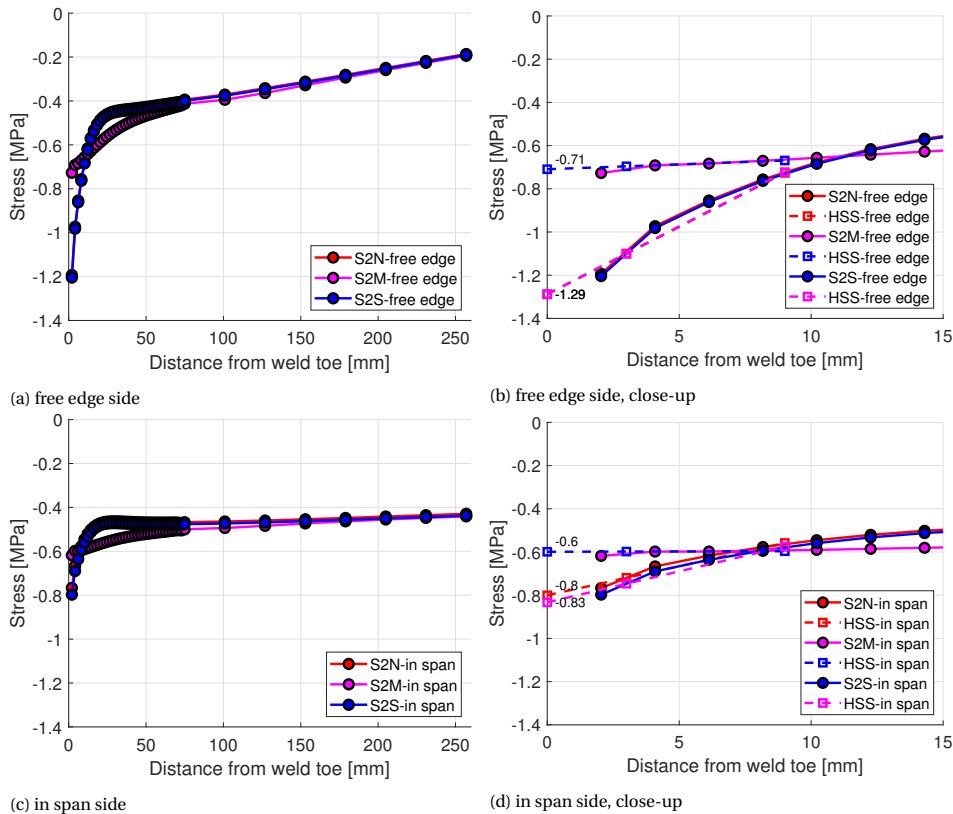


Figure D.2: FE calculated stress at the bottom of S2 under $F = 1$ kN.

D.3. COMPARISON OF THE ELEMENT TYPE FOR DETAIL C3A

Figure D.3 compares the strains computed by the FE models using 8-node linear brick elements with reduced integration (C3D8R) [179] plotted in solid lines or 20-node quadratic brick elements with reduced integration (C3D20R) [179] plotted in hash lines, with the measured strains in the experiments. Same colour is assigned to the curves and the filled region of marks for each compared detail. Higher strain ranges are observed with C3D20R. The models with C3D20R over-predict the strains for S8 N, S5 N, and S6 N compared with the experiments. The results with C3D8R are used in the statistical analysis as a conservative choice.

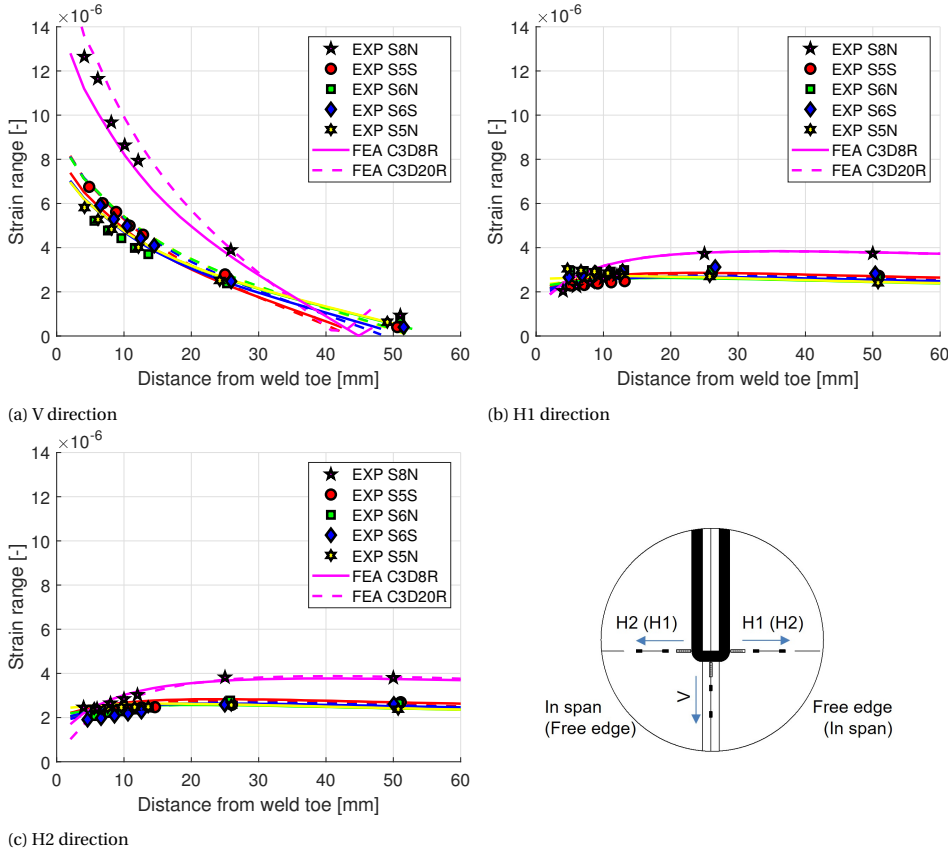


Figure D.3: Strain distribution from the experiments and the FE calculations using C3D8R or C3D20R in LC B under the load range of 2×1 kN.

D.4. VALIDATION OF THE WELDING SIMULATION

D.4.1. INTRODUCTION OF THE WELDING SIMULATION

A T-type fillet welded connection is studied in order to validate the welding simulation. Figure D.4 shows the dimensions and the boundary conditions of the connection described in [217]:

A $50 \times 130 \times 6$ mm³ steel plate is fillet welded to a $300 \times 130 \times 6$ mm³ steel attachment with a nominal weld leg of 6 mm. One side of the plate is clamped rigidly during the experiment. Two passes of welding with a speed of 8.3 mm/s are applied, see Figure D.5. Sufficient cooling time is given between the two welding passes. The welding residual stress perpendicular to the weld line (in the X-axis direction) at the weld toe side is measured using the X-ray diffraction, and the displacement at the free edge is measured to evaluate the angular deflection, see the red lines and the cross in Figure D.5.

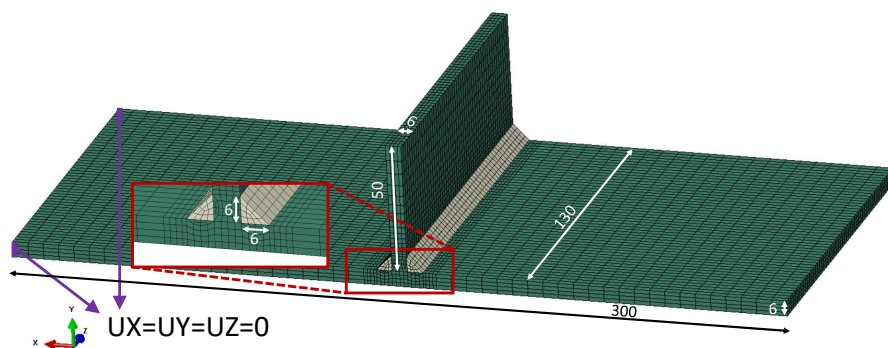


Figure D.4: The dimensions and the boundary conditions of the FE model for the T-type fillet welded connection used in [217] (unit: mm).

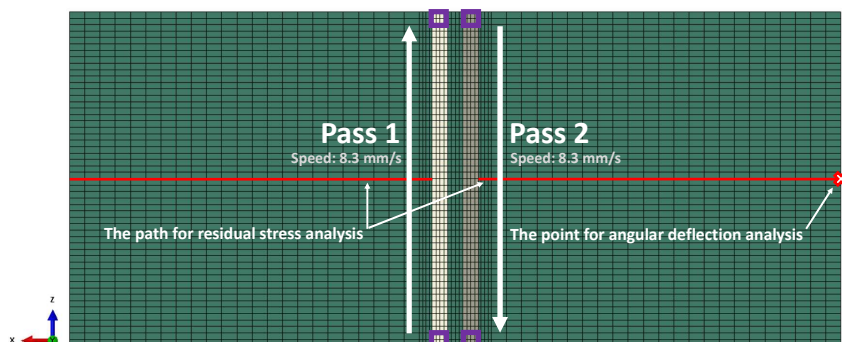


Figure D.5: The welding passes, the path for residual stress analysis, and the cross for angular deflection analysis used in the FE model of this study, designated following [217].

The uncoupled thermal-mechanical analysis is used in the FE model [179]. The steps “heat transfer” and “static” in Abaqus [179] are used for the thermal and the mechanical analysis, respectively. The nodal temperature distribution is calculated in the “heat transfer” step, which is afterwards mapped to the corresponding “static” step for the mechanical analysis. In both types of steps, the weld filler elements are first removed and afterwards reactivated following the welding procedure using the “element and contact pair removal/reactivation” technique available in [179]. Specifically in this case, the four corners circled by the purple colour squares as shown in Figure D.5 are assumed to be spot welded before the simulation. The remaining elements (weld filler) along the weld passes are first removed and afterwards reactivated block-by-block (one block is 5 mm long in the Z-axis direction): first reactivate elements following Pass 1, cool down for 3600 seconds, and then reactivate elements following Pass 2. Spatial constraints, $UX = UY = UZ = 0$, are applied in the FE model in the mechanical analysis at two edges as shown in Figure D.4. 8-node linear brick diffusive heat transfer elements (DC3D8) and stress-displacement elements with reduced integration (C3D8R) are used in the thermal and the mechanical analysis [179], respectively. Same methods as described in Section 7.6 are used in this simulation.

D.4.2. RESULTS OF THE WELDING SIMULATION

Figure D.6 compares the angular deflection and the residual stress in the X-axis direction between the EXPerimental investigation (EXP) and the Finite Element Analysis (FEA). A very good agreement is observed for the residual stress. The FEA gives an approximately 13% over-prediction for the angular deflection. It can be concluded that the welding simulation used in the current study can be applied for welding residual stress analysis.

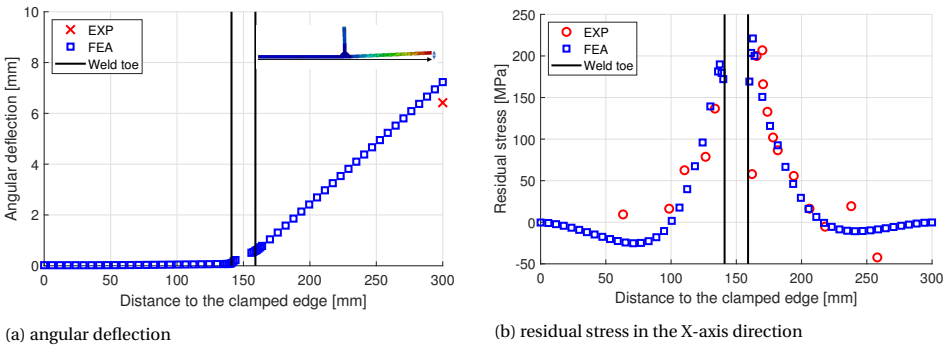


Figure D.6: Comparison of the angular deflection and the residual stress in the T-type welded connection between the EXP from [217] and the FEA in the current study.

D.5. VALIDATION OF THE J-INTEGRAL CALCULATION

D.5.1. INTRODUCTION OF THE FE MODEL

Figure D.7 shows the FE model of the CT specimen used for the J-integral calculation. The dimensions are the same as those in the specimen described in Figure C.2. A 15 mm (from the notch to the crack front) through-thickness crack is inserted into the model, see the red line in Figure D.7. Spatial displacement in the Z-axis direction ($U_Z = 0$) is applied at the front and back sides in order to prevent buckling. The 60° circular sector areas of the upper and lower holes (as shown in green) are linked to the reference points, which are located at the centre of the holes. Monotonic force is applied via the reference point RP-1 in the positive direction of the Y-axis with an increment of 1 kN until the maximum force of 30 kN. The reference points RP-1 and RP-2 are constrained as $U_X = U_Z = U_{RX} = U_{RY} = 0$ and $U_X = U_Y = U_Z = U_{RX} = U_{RY} = 0$, respectively. 8-node linear brick elements, type C3D8 [179], are used with local refined mesh size of $0.2 \times 0.2 \times 0.2 \text{ mm}^3$. The domain integral technique is used for the numerical calculation of J-integral in the FE method [179, 241]. Contours 1 to 10 are used for the J-integral calculation. Either linear elastic or elastic-plastic material is used in the FE model with the elastic modulus (E) of 210 GPa, and the Poisson's ratio (ν) of 0.3. Equation (D.1) defines the power plastic hardening model. The σ_y/E and N in the FE model are set to 0.002 and 0.1 following [242], respectively.

$$\varepsilon = \begin{cases} \sigma/E, & \sigma \leq \sigma_y \\ (\sigma_y/E)(\sigma/\sigma_y)^{1/N}, & \sigma > \sigma_y \end{cases} \quad (\text{D.1})$$

Where ε , σ , and σ_y represent strain, stress, and the yield stress, respectively.

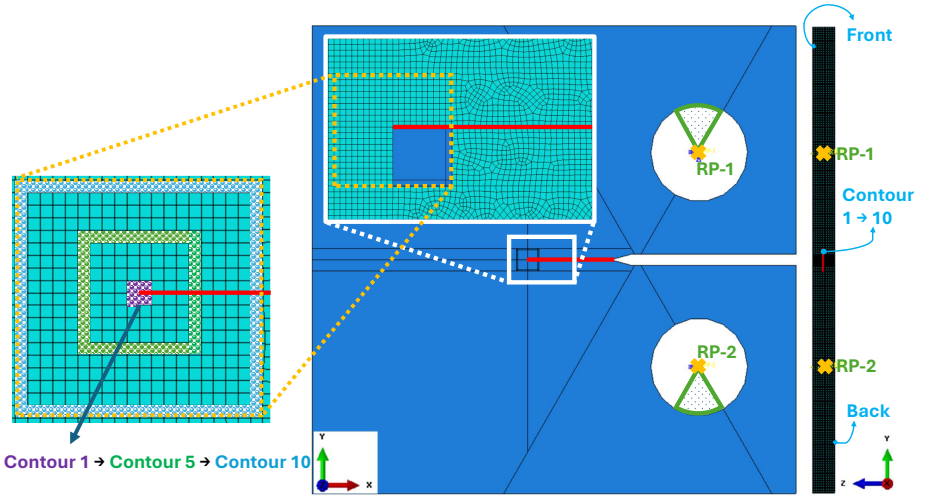


Figure D.7: The FE model of the CT specimen used for the J-integral calculation (see Figure C.2 for dimensions).

D.5.2. RESULTS OF THE FE CALCULATION

Figure D.8 compares the J-integrals calculated by the Abaqus embedded algorithm with the values calculated by the author's post-processing algorithm using Contour 5 with either linear elastic (E) or elastic-plastic (EP) material property. Excellent agreement is observed between the two algorithms. The EP case gives significantly larger J-integral than the E case when the applied force is larger than 21 kN. Figure D.9 shows the J-integral calculations using Contours 1 to 10 at the force of 10 kN, 20 kN, or 25 kN. The results from Contours 3 to 10 are constant in both cases. The maximum differences of the J-integrals calculated by the Abaqus and the author's algorithm using Contours 3 to 10 at 30 kN are 0.41% and 0.85% for the E and the EP cases, respectively. It can be concluded that the author's algorithm calculates the J-integrals accurately for both elastic and elastic-plastic materials.

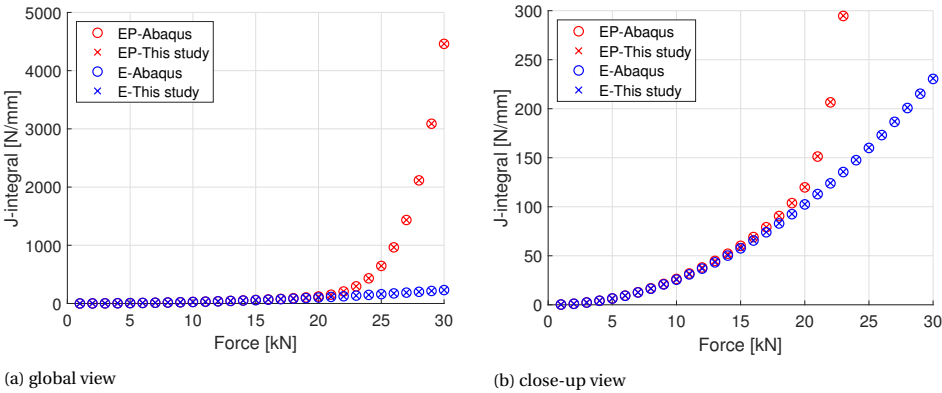


Figure D.8: Comparison of the J-integrals calculated by the Abaqus and the author's (this study) algorithm using Contour 5.

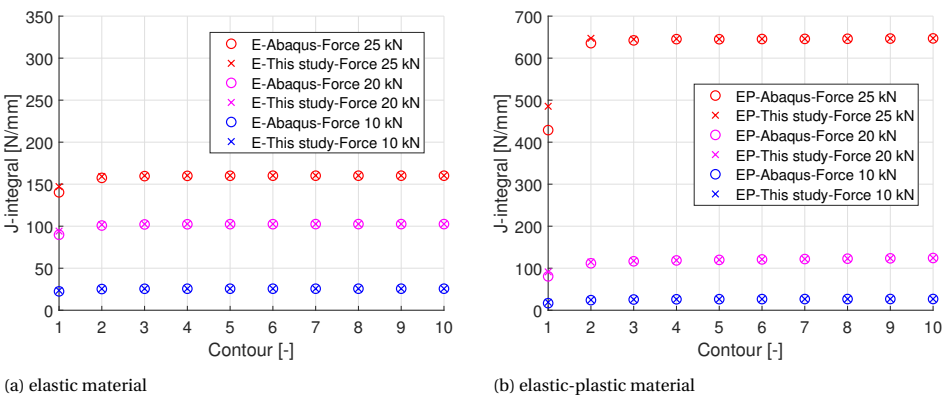


Figure D.9: The J-integral calculations using Contours 1 to 10 at the force of 10 kN, 20 kN or 25 kN.

REFERENCES

- [1] M. H. Kolstein, *Fatigue classification of welded joints in orthotropic steel bridge decks*. Doctoral dissertation, Delft University of Technology, Delft.
- [2] R. Connor, J. Fisher, W. Gatti, V. Gopalaratnam, B. Kozy, B. Leshko, D. L. McQuaid, R. Medlock, D. Mertz, T. Murphy, D. Paterson, O. Sorensen, and J. Yadlosky, *Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges*. United States. Federal Highway Administration, 2012.
- [3] R. Wolchuk, "Orthotropic Redecking of Bridges on the North American Continent," *Structural Engineering International*, vol. 2, no. 2, pp. 125–130, 1992.
- [4] R. Wolchuk, "The Use of Orthotropic Decks in Redecking of Bridges," *Engineering Journal*, vol. 38, no. 1, pp. 2–10, 2001.
- [5] C. Huang and A. R. Mangus, "An International Perspective: Widening Existing Bridges with Orthotropic Steel Deck Panels," *Structural Engineering International*, vol. 18, no. 4, pp. 381–389, 2008.
- [6] A. R. Mangus, "Orthotropic design meets cold weather challenges, An Overview of Orthotropic Steel Deck Bridges in Cold Regions," *Welding Innovation*, vol. 19, no. 1, 2002.
- [7] P. Mehue, "Cracks in Steel Orthotropic Decks," in *Bridge Management* (J. E. Harding, G. A. R. Parke, and M. J. Ryall, eds.), pp. 633–642, Boston, MA: Springer, US, 1990.
- [8] J. R. Cuninghame and C. Beales, "Strengthening and refurbishment of Severn Crossing Part 4: TRRL research on Severn Crossing," *Proceedings of the Institution of Civil Engineers - Structures and Buildings*, vol. 94, no. 1, pp. 37–49, 1992.
- [9] A. Touran and A. Okereke, "Performance of Orthotropic Bridge Decks," *Journal of Performance of Constructed Facilities*, vol. 5, no. 2, pp. 134–148, 1991.
- [10] H. Friedrich, "Orthotropic Steel Bridges in Germany," in *3rd Orthotropic Bridge Conference Proceedings* (C. A. Copelan and L. Duan, eds.), (Sacramento), pp. 665–677, 6 2013.
- [11] J. Maljaars, F. van Dooren, and M. H. Kolstein, "Fatigue assessment for deck plates in orthotropic bridge decks," *Steel Construction*, vol. 5, no. 2, pp. 93–100, 2012.

- [12] C. Cui, Q. Zhang, Y. Luo, H. Hao, and J. Li, "Fatigue reliability evaluation of deck-to-rib welded joints in OSD considering stochastic traffic load and welding residual stress," *International Journal of Fatigue*, vol. 111, pp. 151–160, 2018.
- [13] J. W. Fisher and S. Roy, "Fatigue Damage in Steel Bridges and Extending Their Life," *Advanced Steel Construction*, vol. 11, no. 3, pp. 250–268, 2015.
- [14] F. de Jong, *Renovation techniques for fatigue cracked orthotropic steel bridge decks*. Doctoral dissertation, Delft University of Technology, Delft, 2006.
- [15] J. Maljaars, S. Verdenius, H. Burggraaf, and S. van Es, "Prediction accuracy of fatigue-relevant load effects in an orthotropic deck," *Journal of Constructional Steel Research*, vol. 214, 2024.
- [16] EN 1993-1-9, *Eurocode 3 - Design of steel structures - Part 1-9: Fatigue*. European Committee for Standardization, 2005.
- [17] prEN 1993-1-9, *Eurocode 3 - Design of steel structures – Part 1-9: Fatigue. Doc N3751 (enquiry draft)*. European Committee for Standardization, 2023.
- [18] JSSC, *Fatigue design recommendations for steel structures*. Japanese Society of Steel Construction, 2012.
- [19] AASHTO, *AASHTO LRFD Bridge Design Specifications*. American Association of State Highway and Transportation Officials, 9 ed., 2020.
- [20] Herausgeber FKM, *Analytical Strength Assessment of Components made of Steel, Cast Iron and Aluminum Materials in Mechanical Engineering*. 7 ed., 2020.
- [21] P. Livieri and P. Lazzarin, "Fatigue strength of steel and aluminium welded joints based on generalised stress intensity factors and local strain energy values," *International Journal of Fracture*, vol. 133, pp. 247–276, 2005.
- [22] P. Paris and F. Erdogan, "A critical analysis of crack propagation laws," *Transactions of the ASME: Journal of Basic Engineering*, vol. 85, no. 4, pp. 528–533, 1963.
- [23] A. F. Hobbacher, *Recommendations for Fatigue Design of Welded Joints and Components*. IIW Collection, Springer International Publishing Switzerland, 2 ed., 2016.
- [24] BS 7910:2013+A1:2015, *Guide to methods for assessing the acceptability of flaws in metallic structures*. British Standards Institution, 3 ed., 2015.
- [25] M. Kuna, *Finite Elements in Fracture Mechanics, Theory - Numerics - Applications*, vol. 201 of *Solid Mechanics and Its Applications*. Springer Dordrecht, 1 ed., 2013.
- [26] DNVGL-RP-C203, *Fatigue design of offshore steel structures*. DET NORSKE VERITAS (DNV), 2016.
- [27] prTS 1993-1-901, *Fatigue design of orthotropic bridge decks with the hot spot stress method (final draft)*. European Committee for Standardization, 2023.

- [28] R. I. Stephens, A. Fatemi, R. R. Stephens, and H. O. Fuchs, *Metal fatigue in engineering*. John Wiley & Sons, Inc., 2 ed., 2000.
- [29] J. Schijve, *Fatigue of Structures and Materials*. Springer Dordrecht, 2 ed., 2009.
- [30] M. F. Horstemeyer, D. Farkas, S. Kim, T. Tang, and G. Potirniche, “Nanostructurally small cracks (NSC): A review on atomistic modeling of fatigue,” *International Journal of Fatigue*, vol. 32, no. 9, pp. 1473–1502, 2010.
- [31] P. Chowdhury and H. Sehitoglu, “Mechanisms of fatigue crack growth – a critical digest of theoretical developments,” *Fatigue & Fracture of Engineering Materials & Structures*, vol. 39, no. 6, pp. 652–674, 2016.
- [32] M. D. Sangid, “The physics of fatigue crack initiation,” *International Journal of Fatigue*, vol. 57, pp. 58–72, 2013.
- [33] P. Lehto and H. Remes, “EBSD characterisation of grain size distribution and grain sub-structures for ferritic steel weld metals,” *Welding in the World*, vol. 66, pp. 363–377, 2022.
- [34] D. Radaj, C. M. Sonsino, and W. Fricke, *Fatigue Assessment of Welded Joints by Local Approaches*. Welding and Other Joining Technologies, Woodhead Publishing Limited, 2 ed., 2006.
- [35] J. Pokluda and P. Sandera, *Micromechanisms of Fracture and Fatigue: In a Multi-scale Context*. Engineering Materials and Processes, Springer London, 1 ed., 2010.
- [36] M. A. R. Garcia, *Multiaxial fatigue analysis of high-strength steel welded joints using generalized local approaches*. Doctoral dissertation, Swiss Federal Institute of Technology in Lausanne, Lausanne, 2020.
- [37] V. Solutions, “Arc welding processes.” <https://www.vectorsolutions.com/courses/arc-welding-processes/>.
- [38] B. Schork, P. Kucharczyk, M. Madia, U. Zerbst, J. Hensel, J. Bernhard, D. Tchuindjang, M. Kaffenberger, and M. Oechsner, “The effect of the local and global weld geometry as well as material defects on crack initiation and fatigue strength,” *Engineering Fracture Mechanics*, vol. 198, pp. 103–122, 2018.
- [39] P. Kucharczyk, M. Madia, U. Zerbst, B. Schork, P. Gerwien, and S. Münstermann, “Fracture-mechanics based prediction of the fatigue strength of weldments. Material aspects,” *Engineering Fracture Mechanics*, vol. 198, pp. 79–102, 2018.
- [40] T. L. E. Company, “Weld fusion vs. weld penetration.” <https://www.lincolnelectric.com/sv-SE/Welding-and-Cutting-Resource-Center/Process-and-Theory/Weld-Fusion-vs-Weld-Penetration>.
- [41] C. R. Gagg, “Failure of components and products by ‘engineered-in’ defects: Case studies,” *Engineering Failure Analysis*, vol. 12, no. 6, pp. 1000–1026, 2005.

- [42] U. Zerbst, M. Madia, B. Schork, J. Hensel, P. Kucharczyk, D. Ngoula, D. Tchuindjang, J. Bernhard, and C. Beckmann, *Fatigue and Fracture of Weldments, The IBESS Approach for the Determination of the Fatigue Life and Strength of Weldments by Fracture Mechanics Analysis*. Springer Cham, 1 ed., 2019.
- [43] M. Farajian, "Welding residual stress behavior under mechanical loading," *Welding in the World*, vol. 57, no. 2, pp. 157–169, 2013.
- [44] S. Suresh, *Fatigue of Materials*. Cambridge University Press, 2 ed., 1998.
- [45] R. Ritchie, "Mechanisms of fatigue-crack propagation in ductile and brittle solids," *International Journal of Fracture*, vol. 100, pp. 55–83, 1998.
- [46] W. Fricke, "Fatigue analysis of welded joints: state of development," *Marine Structures*, vol. 16, no. 3, pp. 185–200, 2003.
- [47] L. Pook, *Metal fatigue what it is, Why it matters*, vol. 145 of *Solid Mechanics and its Applications*. Springer Dordrecht, 1 ed., 2007.
- [48] O. H. Basquin, "The Exponential Law of Endurance Tests," *American Society for Testing and Materials Proceedings*, vol. 10, pp. 625–630, 1910.
- [49] J. D. Sørensen, J. Tychsen, J. U. Andersen, and R. D. Brandstrup, "Fatigue Analysis of Load-Carrying Fillet Welds," *Journal of Offshore Mechanics and Arctic Engineering*, vol. 128, no. 1, pp. 65–74, 2006.
- [50] P. Dong, "A structural stress definition and numerical implementation for fatigue analysis of welded joints," *International Journal of Fatigue*, vol. 23, no. 10, pp. 865–876, 2001.
- [51] C. M. Sonsino, W. Fricke, F. de Bruyne, A. Hoppe, A. Ahmadi, and G. Zhang, "Notch stress concepts for the fatigue assessment of welded joints - Background and applications," *International Journal of Fatigue*, vol. 34, no. 1, pp. 2–16, 2012.
- [52] J. Schijve, "Fatigue predictions of welded joints and the effective notch stress concept," *International Journal of Fatigue*, vol. 45, pp. 31–38, 2012.
- [53] M. Bäckström and G. Marquis, "A review of multiaxial fatigue of weldments: experimental results, design code and critical plane approaches," *Fatigue and Fracture of Engineering Materials and Structures*, vol. 24, no. 5, pp. 279–291, 2001.
- [54] P. Lazzarin and R. Tovo, "A notch intensity factor approach to the stress analysis of welds," *Fatigue and Fracture of Engineering Materials and Structures*, vol. 21, no. 9, pp. 1089–1103, 1998.
- [55] P. Lazzarin and R. Zambardi, "A finite-volume-energy based approach to predict the static and fatigue behavior of components with sharp V-shaped notches," *International Journal of Fracture*, vol. 112, pp. 275–298, 2001.

- [56] P. Lazzarin, F. Berto, and M. Zappalorto, "Rapid calculations of notch stress intensity factors based on averaged strain energy density from coarse meshes: Theoretical bases and applications," *International Journal of Fatigue*, vol. 32, no. 10, pp. 1559–1567, 2010.
- [57] S. S. Manson, "Behavior of materials under conditions of thermal stress," tech. rep., Lewis Flight Propulsion Laboratory, 1954.
- [58] L. F. Coffin, "A Study of the Effects of Cyclic Thermal Stresses on a Ductile Metal," *Transactions of the ASME: Journal of Fluids Engineering*, vol. 76, no. 6, pp. 931–949, 1954.
- [59] J. Morrow, "Cyclic Plastic Strain Energy and Fatigue of Metals," in *STP378-EB: Internal Friction, Damping, and Cyclic Plasticity*, pp. 45–87, ASTM International, 1965.
- [60] K. N. Smith, P. Watson, and T. H. Topper, "A stress–strain function for the fatigue of metals," *Journal of materials*, vol. 5, pp. 767–778, 1970.
- [61] T. L. Anderson, *Fracture Mechanics, Fundamentals and Applications*. CRC Press, 3 ed., 2005.
- [62] M. Janssen, J. Zuidema, and R. J. H. Wanhill, *Fracture Mechanics*. Spon Press, 2 ed., 2004.
- [63] J. R. Rice, "A Path Independent Integral and the Approximate Analysis of Strain Concentration by Notches and Cracks," *Journal of Applied Mechanics*, vol. 35, no. 2, pp. 379–386, 1968.
- [64] JCSS, *JCSS PROBABILISTIC MODEL CODE, PART 3: RESISTANCE MODELS, SECTION 3.12 FATIGUE MODELS FOR METALLIC STRUCTURES*. Joint Committee on Structural Safety, 2013.
- [65] A. Saxena, *Advanced Fracture Mechanics and Structural Integrity*. CRC Press, 1 ed., 2019.
- [66] A. Ahola, T. Skriko, and T. Björk, "Fatigue strength assessment of ultra-high-strength steel fillet weld joints using 4R method," *Journal of Constructional Steel Research*, vol. 167, 2020.
- [67] M. Aygöl, M. Al-Emrani, Z. Barsoum, and J. Leander, "Investigation of distortion-induced fatigue cracked welded details using 3D crack propagation analysis," *International Journal of Fatigue*, vol. 64, pp. 54–66, 2014.
- [68] A. H. Noroozi, G. Glinka, and S. Lambert, "A study of the stress ratio effects on fatigue crack growth using the unified two-parameter fatigue crack growth driving force," *International Journal of Fatigue*, vol. 29, no. 9-11, pp. 1616–1633, 2007.
- [69] S. Mikheevskiy, S. Bogdanov, and G. Glinka, "Analysis of fatigue crack growth under spectrum loading – The UniGrow fatigue crack growth model," *Theoretical and Applied Fracture Mechanics*, vol. 79, pp. 25–33, 2015.

- [70] B. Pedrosa, J. Correia, G. Lesiuk, C. Rebelo, and M. Veljkovic, "Fatigue crack growth modelling for S355 structural steel considering plasticity-induced crack-closure by means of UniGrow model," *International Journal of Fatigue*, vol. 164, 2022.
- [71] S. Röscher and M. Knobloch, "Towards a prognosis of fatigue life using a Two-Stage-Model, Application to butt welds," *Steel Construction*, vol. 12, no. 3, pp. 198–208, 2019.
- [72] M. Vormwald and T. Seeger, "THE CONSEQUENCES OF SHORT CRACK CLOSURE ON FATIGUE CRACK GROWTH UNDER VARIABLE AMPLITUDE LOADING," *Fatigue & Fracture of Engineering Materials & Structures*, vol. 14, no. 2-3, pp. 205–225, 1991.
- [73] R. Dekker, F. P. van der Meer, J. Maljaars, and L. J. Sluys, "A cohesive XFEM model for simulating fatigue crack growth under mixed-mode loading and overloading," *International Journal for Numerical Methods in Engineering*, vol. 118, no. 10, pp. 561–577, 2019.
- [74] R. Dekker, F. P. van der Meer, J. Maljaars, and L. J. Sluys, "A cohesive XFEM model for simulating fatigue crack growth under various load conditions," *Engineering Fracture Mechanics*, vol. 248, 2021.
- [75] D. Tchoffo Ngoula, M. Madia, H. T. Beier, M. Vormwald, and U. Zerbst, "Cyclic J-integral: Numerical and analytical investigations for surface cracks in weldments," *Engineering Fracture Mechanics*, vol. 198, pp. 24–44, 2018.
- [76] K. Walker, "The Effect of Stress Ratio During Crack Propagation and Fatigue for 2024-T3 and 7075-T6 Aluminum," in *STP 462: Effects of Environment and Complex Load History on Fatigue Life*, pp. 1–14, ASTM International, 1970.
- [77] O. E. Wheeler, "Spectrum Loading and Crack Growth," *Transactions of the ASME: Journal of Basic Engineering*, vol. 94, no. 1, pp. 181–186, 1972.
- [78] S. G. Russell, "A new model for fatigue crack growth retardation following an overload," *Engineering Fracture Mechanics*, vol. 33, no. 6, pp. 839–854, 1989.
- [79] EN 1993-2, *Eurocode 3 - Design of steel structures - Part 2: Steel Bridges*. European Committee for Standardization, 2006.
- [80] EN 1991-2, *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges*. European Committee for Standardization, 2003.
- [81] M. Li, K. Hashimoto, and K. Sugiura, "Influence of Asphalt Surfacing on Fatigue Evaluation of Rib-to-Deck Joints in Orthotropic Steel Bridge Decks," *Journal of Bridge Engineering*, vol. 19, no. 10, 2014.
- [82] C. Cui, Q. Zhang, H. Hao, J. Li, and Y. Bu, "Influence of Asphalt Pavement Conditions on Fatigue Damage of Orthotropic Steel Decks: Parametric Analysis," *Journal of Bridge Engineering*, vol. 23, no. 12, 2018.

- [83] T. Guo, A. Li, and J. Li, "Fatigue Life Prediction of Welded Joints in Orthotropic Steel Decks Considering Temperature Effect and Increasing Traffic Flow," *Structural Health Monitoring*, vol. 7, no. 3, pp. 189–202, 2008.
- [84] M.-C. Tang, "A new concept of orthotropic steel bridge deck," *Structure and Infrastructure Engineering*, vol. 7, no. 7-8, pp. 587–595, 2011.
- [85] B. Ji, R. Liu, C. Chen, H. Maeno, and X. Chen, "Evaluation on root-deck fatigue of orthotropic steel bridge deck," *Journal of Constructional Steel Research*, vol. 90, pp. 174–183, 2013.
- [86] I. Farreras-Alcover, M. K. Chryssanthopoulos, and J. E. Andersen, "Data-based models for fatigue reliability of orthotropic steel bridge decks based on temperature, traffic and strain monitoring," *International Journal of Fatigue*, vol. 95, pp. 104–119, 2017.
- [87] K. Dudás, G. Jakab, B. Kövesdi, and L. Dunai, "Assessment of fatigue behaviour of orthotropic steel bridge decks using monitoring system," *Procedia Engineering*, vol. 133, pp. 770–777, 2015.
- [88] J. Di, X. Ruan, X. Zhou, J. Wang, and X. Peng, "Fatigue assessment of orthotropic steel bridge decks based on strain monitoring data," *Engineering Structures*, vol. 228, 2021.
- [89] J. W. Fisher and J. M. Barsom, "Evaluation of Cracking in the Rib-to-Deck Welds of the Bronx-Whitestone Bridge," *Journal of Bridge Engineering*, vol. 21, no. 3, 2016.
- [90] T. Attema, A. A. Kosgodagan, O. Morales-Nápoles, and J. Maljaars, "Maintenance decision model for steel bridges: a case in the Netherlands," *Structure and Infrastructure Engineering*, vol. 13, no. 2, pp. 242–253, 2017.
- [91] C. Miki and T. Konishi, "Retrofit Engineering for Steel Bridge Structures in Japan," in *IABSE Symposium: Improving Infrastructure Worldwide*, (Weimar), International Association for Bridge and Structural Engineering, 9 2007.
- [92] T. Yuge, F. Machida, H. Morikawa, C. Miki, T. Kamiki, and T. Masui, "Analysis of Fatigue Damage Patterns in Orthotropic Steel Deck of Tokyo Metropolitan Expressways," in *1st Orthotropic Bridge Conference Proceedings*, (Sacramento), pp. 531–542, 8 2004.
- [93] J. Maljaars, D. Gration, E. Vonk, and F. van Dooren, "Crack growth prediction of deck plate-stiffener joints in orthotropic steel bridge decks," in *IABSE Conference: Assessment, Upgrading and Refurbishment of Infrastructures*, vol. 2, (Rotterdam), pp. 502–503, International Association for Bridge and Structural Engineering, 5 2013.
- [94] M. S. Pfeil, R. C. Battista, and A. J. Mergulhão, "Stress concentration in steel bridge orthotropic decks," *Journal of Constructional Steel Research*, vol. 61, no. 8, pp. 1172–1184, 2005.

- [95] H.-B. Sim and C.-M. Uang, "Stress analyses and parametric study on full-scale fatigue tests of rib-to-deck welded joints in steel orthotropic decks," *Journal of Bridge Engineering*, vol. 17, pp. 765–773, 2012.
- [96] H. Yang, P. Wang, H. Qian, and P. Dong, "Analysis of fatigue test conditions for reproducing weld toe cracking into U-rib wall in orthotropic bridge decks," *International Journal of Fatigue*, vol. 162, 2022.
- [97] H.-B. Sim, C.-M. Uang, and C. Sikorsky, "Effects of Fabrication Procedures on Fatigue Resistance of Welded Joints in Steel Orthotropic Decks," *Journal of Bridge Engineering*, vol. 14, no. 5, pp. 366–373, 2009.
- [98] C. V. Dung, E. Sasaki, K. Tajima, and T. Suzuki, "Investigations on the Effect of Weld Penetration on Fatigue Strength of Rib-to-Deck Welded Joints in Orthotropic Steel Decks," *International Journal of Steel Structures*, vol. 15, no. 2, pp. 299–310, 2015.
- [99] C. Cui, Q. Zhang, Y. Bao, Y. Bu, and Y. Luo, "Fatigue life evaluation of welded joints in steel bridge considering residual stress," *Journal of Constructional Steel Research*, vol. 153, pp. 509–518, 2019.
- [100] J. Chen, C. Wei, and Y. Zhao, "Fatigue resistance of orthotropic steel deck system with double-side welded rib-to-deck joint," *Advances in Structural Engineering*, vol. 26, no. 5, pp. 952–965, 2022.
- [101] M. Li, C. Cui, H. Wang, Q. Zhang, L. Da, and Y. Bu, "Fatigue behavior and remaining life evaluation of rib-to-deck joints using interior repair welds," *Thin-Walled Structures*, vol. 183, 2023.
- [102] Z. G. Xiao, K. Yamada, S. Ya, and X. L. Zhao, "Stress analyses and fatigue evaluation of rib-to-deck joints in steel orthotropic decks," *International Journal of Fatigue*, vol. 30, no. 8, pp. 1387–1397, 2008.
- [103] R. Liu, Y. Liu, B. Ji, M. Wang, and Y. Tian, "Hot spot stress analysis on rib-deck welded joint in orthotropic steel decks," *Journal of Constructional Steel Research*, vol. 97, pp. 1–9, 2014.
- [104] R. Liu, B. Ji, M. Wang, C. Chen, and H. Maeno, "Numerical Evaluation of Toe-Deck Fatigue in Orthotropic Steel Bridge Deck," *Journal of Performance of Constructed Facilities*, vol. 29, no. 6, 2015.
- [105] J. Heng, K. Zheng, C. Gou, Y. Zhang, and Y. Bao, "Fatigue Performance of Rib-to-Deck Joints in Orthotropic Steel Decks with Thickened Edge U-Ribs," *Journal of Bridge Engineering*, vol. 22, no. 9, 2017.
- [106] Q. Wang, B. Ji, Z. Fu, and Z. Ye, "Evaluation of crack propagation and fatigue strength of rib-to-deck welds based on effective notch stress method," *Construction and Building Materials*, vol. 201, pp. 51–61, 2019.

- [107] A. Bignonnet, B. Jacob, J. Caracilli, and M. LaFrance, "Fatigue resistance of orthotropic steel decks," in *Proceedings IABSE Workshop Remaining Fatigue Life of Steel Structures*, (Lausanne), pp. 227–236, International Association for Bridge and Structural Engineering, 1990.
- [108] B. Cheng, X. Ye, X. Cao, D. D. Mbako, and Y. Cao, "Experimental study on fatigue failure of rib-to-deck welded connections in orthotropic steel bridge decks," *International Journal of Fatigue*, vol. 103, pp. 157–167, 2017.
- [109] B. Cheng, X. Cao, X. Ye, and Y. Cao, "Fatigue tests of welded connections between longitudinal stringer and deck plate in railway bridge orthotropic steel decks," *Engineering Structures*, vol. 153, pp. 32–42, 2017.
- [110] M. Li, Y. Suzuki, K. Hashimoto, and K. Sugiura, "Experimental Study on Fatigue Resistance of Rib-to-Deck Joint in Orthotropic Steel Bridge Deck," *Journal of Bridge Engineering*, vol. 23, no. 2, 2018.
- [111] B. Wang, X.-Y. Zhou, and A. Chen, "Probabilistic study on the macro-crack initiation of the rib-to-deck welded joint on orthotropic steel deck," *International Journal of Fatigue*, vol. 139, 2020.
- [112] P. Luo, Q. Zhang, Y. Bao, and Y. Bu, "Fatigue performance of welded joint between thickened-edge U-rib and deck in orthotropic steel deck," *Engineering Structures*, vol. 181, pp. 699–710, 2019.
- [113] D. Wang, C. Xiang, Y. Ma, A. Chen, and B. Wang, "Experimental study on the root-deck fatigue crack on orthotropic steel decks," *Materials and Design*, vol. 203, 2021.
- [114] S. Kainuma, M. Yang, Y.-S. Jeong, S. Inokuchi, A. Kawabata, and D. Uchida, "Experiment on fatigue behavior of rib-to-deck weld root in orthotropic steel decks," *Journal of Constructional Steel Research*, vol. 119, pp. 113–122, 2016.
- [115] S. Kainuma, M. Yang, Y.-S. Jeong, S. Inokuchi, A. Kawabata, and D. Uchida, "Fatigue Behavior Investigation and Stress Analysis for Rib-to-Deck Welded Joints in Orthotropic Steel Decks," *International Journal of Steel Structures*, vol. 18, pp. 512–527, 2018.
- [116] S. Kainuma, Y.-S. Jeong, M. Yang, and S. Inokuchi, "Welding residual stress in roots between deck plate and U-rib in orthotropic steel decks," *Measurement: Journal of the International Measurement Confederation*, vol. 92, pp. 475–482, 2016.
- [117] R. Gadallah, S. Tsutsumi, T. Yonezawa, and H. Shimanuki, "Residual stress measurement at the weld root of rib-to-deck welded joints in orthotropic steel bridge decks using the contour method," *Engineering Structures*, vol. 219, 2020.
- [118] C. Cui, Q. Zhang, Y. Bao, Y. Bu, and Z. Ye, "Fatigue Damage Evaluation of Orthotropic Steel Deck Considering Weld Residual Stress Relaxation Based on Continuum Damage Mechanics," *Journal of Bridge Engineering*, vol. 23, no. 10, 2018.

- [119] A. Bruls, "Measurement and interpretation of dynamic loads in bridges, Phase 3 Fatigue behaviour of orthotropic steel decks," tech. rep., Commission of the European Communities, 1991.
- [120] S. Ya, *Fatigue durability evaluations of trough to deck plate welded details of orthotropic steel deck*. Doctoral dissertation, Nagoya University, Nagoya, 2009.
- [121] H. Yuan, "Optimization of rib-to-deck welds for steel orthotropic bridge decks," master's thesis, Virginia Polytechnic Institute and State University, Blacksburg, 2011.
- [122] Z. Fu, B. Ji, C. Zhang, and Q. Wang, "Fatigue Performance of Roof and U-Rib Weld of Orthotropic Steel Bridge Deck with Different Penetration Rates," *Journal of Bridge Engineering*, vol. 22, no. 6, 2017.
- [123] N. Konda, M. Nishio, M. Ichimiya, T. Kasugai, and S. Kiyokawa, "Development of fatigue test method and improvement of fatigue life by new functional steel plates for welding of trough rib and deck plate of orthotropic decks," *International Journal of Steel Structures*, vol. 13, pp. 191–197, 2013.
- [124] N. Hayashi, Y. Kudo, M. Okumura, D. Uchida, and T. Mori, "Influence of Load Range on Fatigue Crack Propagation Behavior in Orthotropic Steel Deck," in *4th Orthotropic Bridge Conference Proceedings*, (Tianjin), pp. 546–555, 9 2015.
- [125] J. Maljaars and A. C. W. M. Vrouwenvelder, "Probabilistic fatigue life updating accounting for inspections of multiple critical locations," *International Journal of Fatigue*, vol. 68, pp. 24–37, 2014.
- [126] J. Maljaars, E. Bonet, and R. J. M. Pijpers, "Fatigue resistance of the deck plate in steel orthotropic deck structures," *Engineering Fracture Mechanics*, vol. 201, pp. 214–228, 2018.
- [127] W. Nagy, *Fatigue assessment of orthotropic steel decks based on fracture mechanics*. Doctoral dissertation, Ghent University, Ghent, 2017.
- [128] X. Shao, W. Qu, J. Cao, and Y. Yao, "Static and fatigue properties of the steel-UHPC lightweight composite bridge deck with large U ribs," *Journal of Constructional Steel Research*, vol. 148, pp. 491–507, 2018.
- [129] Q. Zhang, Y. Liu, Y. Bao, D. Jia, Y. Bu, and Q. Li, "Fatigue performance of orthotropic steel-concrete composite deck with large-size longitudinal U-shaped ribs," *Engineering Structures*, vol. 150, pp. 864–874, 2017.
- [130] Z. Shi, Q. Su, F. Kavoura, and M. Veljkovic, "Fatigue behavior evaluation of full-scale OSD-UHPC composite bridge deck system," *Engineering Structures*, vol. 275, 2023.
- [131] Z. Feng, C. Li, J. He, L. Ke, Z. Lei, and G. Vasdravellis, "Static and fatigue test on lightweight UHPC-OSD composite bridge deck system subjected to hogging moment," *Engineering Structures*, vol. 241, 2021.

- [132] H. Abdelbaset, B. Cheng, L. Tian, H. Li, and Q. Zhang, "Reduce hot spot stresses in welded connections of orthotropic steel bridge decks by using UHPC layer: Experimental and numerical investigation," *Engineering Structures*, vol. 220, 2020.
- [133] D. C. Wouter, "Renovation techniques for rib to deckplate fatigue cracking in orthotropic bridge decks," *Scientific Research and Essays*, vol. 6, no. 9, pp. 1977–1986, 2011.
- [134] R. C. Battista, M. S. Pfeil, and E. M. L. Carvalho, "Fatigue life estimates for a slender orthotropic steel deck," *Journal of Constructional Steel Research*, vol. 64, pp. 134–143, 2008.
- [135] S. T. de Freitas, *Steel plate reinforcement of orthotropic bridge decks*. Doctoral dissertation, Delft University of Technology, Delft, 2012.
- [136] M. Feldmann, G. Sedlacek, and A. Geßler, "A system of steel-elastomer sandwich plates for strengthening orthotropic bridge decks," *Mechanics of Composite Materials*, vol. 43, no. 2, pp. 183–190, 2007.
- [137] Y. Xia, H. Nassif, E.-S. Hwang, and D. Linzell, "Optimization of Design Details in Orthotropic Steel Decks Subjected to Static and Fatigue Loads," *Transportation Research Record: Journal of the Transportation Research Board*, vol. 2331, no. 1, pp. 14–23, 2013.
- [138] J. S. Leendertz, *Fatigue Behaviour of Closed Stiffener to Crossbeam Connections in Orthotropic Steel Bridge Decks*. Doctoral dissertation, Delft University of Technology, Delft, 2008.
- [139] Z. Xiang and Z. Zhu, "Fatigue behavior of orthotropic composite bridge decks without cutout at rib-to-floorbeam intersection," *Journal of Constructional Steel Research*, vol. 201, 2023.
- [140] L. W. Tong and Z. Y. Shen, "Fatigue Behaviour of L-section Stiffener to Cross-beam Joints in Orthotropic Steel Bridge Decks," *Advances in Structural Engineering*, vol. 2, pp. 265–271, 8 1999.
- [141] P. Croce and D. Pellegrini, "Fatigue Behavior of Stiffener to Cross Beam Joints in Orthotropic Steel Decks," *Procedia Engineering*, vol. 101, pp. 101–108, 2015.
- [142] Y. Deng, T. Liu, H. Ju, and A. Li, "Investigation on fatigue cracks of diaphragm cutout in bridge orthotropic steel deck," *Structure and Infrastructure Engineering*, vol. 20, no. 5, pp. 682–698, 2024.
- [143] H. Yang, P. Wang, and H. Qian, "Fatigue behavior of typical details of orthotropic steel bridges in multiaxial stress states using traction structural stress," *International Journal of Fatigue*, vol. 141, 2020.
- [144] J. Maljaars, J. Leander, A. Nussbaumer, J. D. Sørensen, and D. Straub, "Models and methods for probabilistic safety assessment of steel structures subject to fatigue," *Structural Safety*, vol. 113, 2025.

- [145] T. R. Gurney, "Thickness effect in 'relatively thin' welded joints," tech. rep., Welding Inst., Cambridge (United Kingdom), 1995.
- [146] F. R. Mashiri and X. L. Zhao, "Thickness Effect in Welded Joints-A Review," in *Proceedings of the 15th International Offshore and Polar Engineering Conference*, (Seoul), pp. 325–332, International Society of Offshore and Polar Engineers, 6 2005.
- [147] S. W. Kang, W. S. Kim, and Y. M. Paik, "Fatigue Strength of Fillet Welded Steel Structure Under Out-of-plane Bending Load," *International Journal of Korean Welding Society*, vol. 2, no. 1, pp. 33–39, 2002.
- [148] Q. Zhang, C. Cui, Y. Bu, Y. Liu, and H. Ye, "Fatigue tests and fatigue assessment approaches for rib-to-diaphragm in steel orthotropic decks," *Journal of Constructional Steel Research*, vol. 114, pp. 110–118, 2015.
- [149] Y. Huang, Q. Zhang, Y. Bao, and Y. Bu, "Fatigue assessment of longitudinal rib-to-crossbeam welded joints in orthotropic steel bridge decks," *Journal of Constructional Steel Research*, vol. 159, pp. 53–66, 2019.
- [150] H. Abdelbaset, B. Cheng, L. Tian, H. Li, and J. Zhao, "Enhancing fatigue resistance of rib-to-floorbeam welded connections in orthotropic steel bridge decks by using UHPC layer: An experimental study," *Structures*, vol. 36, pp. 153–167, 2022.
- [151] B. Cheng, H. Abdelbaset, H. Li, L. Tian, and J. Zhao, "Fatigue behavior of rib-to-floorbeam welded connections in UHPC reinforced OSDs subjected to longitudinal flexural," *Engineering Failure Analysis*, vol. 137, 2022.
- [152] J. Maljaars and R. J. M. Pijpers, "Stress intensity factors for fatigue loaded details between crossbeams and trapezoidal stringers," in *3rd Orthotropic Bridge Conference Proceedings* (C. A. Copelan and L. Duan, eds.), pp. 211–221, 6 2013.
- [153] R. J. Connor, "Influence of Cutout Geometry on Stresses at Welded Rib-to-Diaphragm Connections in Steel Orthotropic Bridge Decks," *Transportation Research Record: Journal of the Transportation Research Board*, vol. 1892, pp. 78–87, 2004.
- [154] H. Suganuma and C. Miki, "Full size fatigue tests of the new orthotropic steel deck system," tech. rep., Japan Institute of Welding, 2007.
- [155] S. Roy, J. W. Fisher, N. K. Manandhar, R. S. D. Alapati, and Y. C. Park, "Laboratory fatigue evaluation of replacement orthotropic deck for a signature bridge," in *Bridge Maintenance, Safety, Management, Resilience and Sustainability* (F. Biondini and D. M. Frangopol, eds.), pp. 3769–3775, CRC Press, 1 ed., 7 2012.
- [156] C.-K. Oh and D. Bae, "Fatigue Test of an Advanced Orthotropic Steel Deck System using High Performance Steel for Bridges," *International Journal of Steel Structures*, vol. 13, no. 1, pp. 93–101, 2013.

- [157] J. Di, J. Wang, X. Zhou, X. Peng, and F. Qin, "Fatigue Behavior of Rib-to-Floor Beam Junctions with Separate Inner Stiffeners in Orthotropic Steel Bridge Decks," *Journal of Bridge Engineering*, vol. 27, no. 5, 2022.
- [158] K. Yokozeki, *High fatigue resistant orthotropic steel bridge decks*. Doctoral dissertation, Tokyo City University, Tokyo, 2017.
- [159] J. Saunders, Y. Chen, J. A. Marks, I. Hodgson, R. Sause, and B. M. Kozy, "Finite-Element Fatigue Analysis of a New Rib-to-Floor Beam Connection for Orthotropic Steel Decks," *Journal of Bridge Engineering*, vol. 26, 2021.
- [160] W. De Corte, "Parametric study of floorbeam cutouts for orthotropic bridge decks to determine shape factors," *Bridge Structures*, vol. 5, no. 2-3, pp. 75–85, 2009.
- [161] L. Fryba and S. Urushadze, "Improvement of fatigue properties of orthotropic decks," *Engineering Structures*, vol. 33, pp. 1166–1169, 2011.
- [162] F. Jiang, Y. Ding, Y. Song, F. Geng, and Z. Wang, "CFRP strengthening of fatigue cracks at U-rib to diaphragm welds in orthotropic steel bridge decks: Experimental study, optimization, and decision-making," *Structures*, vol. 43, pp. 1216–1229, 2022.
- [163] X. Qiang, Y. Wu, Y. Wang, and X. Jiang, "Novel crack repair method of steel bridge diaphragm employing Fe-SMA," *Engineering Structures*, vol. 292, 2023.
- [164] X. Qiang, Y. Wu, X. Jiang, and G. Xu, "Fatigue performance of cracked diaphragm cutouts in steel bridge reinforced employing CFRP/SMA," *Journal of Constructional Steel Research*, vol. 211, 2023.
- [165] W. Fricke, "IIW guideline for the assessment of weld root fatigue," *Welding in the World*, vol. 57, pp. 753–791, 2013.
- [166] O. D. Dijkstra and T. Nicolaas, *Vermoeiingsonderzoek reparatiemethode trog dekplaat verbindingen Moerdijkbrug*. TNO report 2000-CON-R4016, TNO Bouw, Rijswijk, Netherlands (in Dutch), 2000.
- [167] P. Wang, X. Pei, P. Dong, and S. Song, "Traction structural stress analysis of fatigue behaviors of rib-to-deck joints in orthotropic bridge deck," *International Journal of Fatigue*, vol. 125, pp. 11–22, 2019.
- [168] W. Wu, M. H. Kolstein, and M. Veljkovic, "Fatigue resistance of rib-to-deck welded joint in OSDs, analyzed by fracture mechanics," *Journal of Constructional Steel Research*, vol. 162, 2019.
- [169] BS 7608:2014+A1:2015, *Guide to fatigue design and assessment of steel products*. British Standards Institution, 2015.
- [170] EN 10025-2, *Hot rolled products of structural steels - Part 2: Technical delivery conditions for non-alloy structural steels*. European Committee for Standardization, 2004.

- [171] B. M. Kozy, R. J. Connor, D. Paterson, and D. R. Mertz, "Proposed Revisions to AASHTO-LRFD Bridge Design Specifications for Orthotropic Steel Deck Bridges," *Journal of Bridge Engineering*, vol. 16, no. 6, pp. 759–767, 2011.
- [172] P. A. Tsakopoulos and J. W. Fisher, *Fatigue Resistance Investigation for the Orthotropic Deck on the Bronx-Whitestone Bridge*. ATLSS report No. 02-05, 117 ATLSS Drive Bethlehem, PA, United States, 2002.
- [173] ISO 5817, *Welding-Fusion-welded joints in steel, nickel, titanium and their alloys (beam welding excluded)-Quality levels for imperfections*. International Organization for Standardization, 3 ed., 2014.
- [174] C. Favi, F. Campi, M. Germani, and M. Mandolini, "A data framework for environmental assessment of metal arc welding processes and welded structures during the design phase," *The International Journal of Advanced Manufacturing Technology*, vol. 105, pp. 967–993, 2019.
- [175] J. Maljaars, R. J. M. Pijpers, W. Wu, and M. H. Kolstein, "Fatigue resistance of rib to deck, crossbeam to deck and deck to deck welds in orthotropic decks using structural stress," *International Journal of Fatigue*, vol. 175, 2023.
- [176] W. Wu, M. H. Kolstein, M. Veljkovic, R. J. M. Pijpers, and J. Vorstenbosch-krabbe, "Fatigue behaviour of the closed rib to deck and crossbeam joint in a newly designed orthotropic bridge deck," *ce/papers*, vol. 1, pp. 2378–2387, 9 2017.
- [177] EN 1990, *Eurocode—Basis of structural design*. European Committee for Standardization, 2002.
- [178] H. Bartsch, K. Drebenstedt, B. Seyfried, M. Feldmann, U. Kuhlmann, and T. Ummenhofer, "Analysis of fatigue test data to reassess EN 1993-1-9 detail categories," *Steel Construction*, vol. 13, no. 4, pp. 280–293, 2020.
- [179] Simulia, *Abaqus User's Manual version 2019*. Dassault Systèmes Simulia Corp., 2019.
- [180] W. Park and C. Miki, "Fatigue assessment of large-size welded joints based on the effective notch stress approach," *International Journal of Fatigue*, vol. 30, no. 9, pp. 1556–1568, 2008.
- [181] M. M. Pedersen, "Multiaxial fatigue assessment of welded joints using the notch stress approach," *International Journal of Fatigue*, vol. 83, pp. 269–279, 2016.
- [182] M. D. Chapetti and C. Steimbregger, "A simple fracture mechanics estimation of the fatigue endurance of welded joints," *International Journal of Fatigue*, vol. 125, pp. 23–34, 2019.
- [183] C. Fischer, O. Feltz, W. Fricke, and P. Lazzarin, "Application of the Notch Stress Intensity and Crack Propagation Approaches to weld toe and root fatigue," *Welding in the World*, vol. 55, pp. 30–39, 2011.

- [184] U. Zerbst, M. Madia, M. Vormwald, and H.Th. Beier, "Fatigue strength and fracture mechanics – A general perspective," *Engineering Fracture Mechanics*, vol. 198, pp. 2–23, 2018.
- [185] M. Aygül, M. Al-Emrani, Z. Barsoum, and J. Leander, "An investigation of distortion-induced fatigue cracking under variable amplitude loading using 3D crack propagation analysis," *Engineering Failure Analysis*, vol. 45, pp. 151–163, 2014.
- [186] R. F. Sanches, A. M. P. de Jesus, J. A. F. O. Correia, A. L. L. da Silva, and A. A. Fernandes, "A probabilistic fatigue approach for riveted joints using Monte Carlo simulation," *Journal of Constructional Steel Research*, vol. 110, pp. 149–162, 2015.
- [187] J. A. Correia, A. M. D. Jesus, and A. Fernández-Canteli, "Local unified probabilistic model for fatigue crack initiation and propagation: Application to a notched geometry," *Engineering Structures*, vol. 52, pp. 394 – 407, 2013.
- [188] M. Lukić and C. Cremona, "Probabilistic optimization of welded joints maintenance versus fatigue and fracture," *Reliability Engineering & System Safety*, vol. 72, no. 3, pp. 253–264, 2001.
- [189] J. Yau, S. Wang, and H. Corten, "A mixed-mode crack analysis of isotropic solids using conservation laws of elasticity," *Journal of Applied Mechanics*, vol. 47, no. 2, pp. 335–341, 1980.
- [190] Simulia, *Abaqus User's Manual version 2014*. Dassault Systèmes Simulia Corp., 2014.
- [191] FRANC3D, *FRANC3D V7.1 ABAQUS Tutorial*. Fracture Analysis Consultants, Inc., 2017.
- [192] C. Newman Jr., J and S. Raju, I, "An empirical stress-intensity factor equation for the surface crack," *Engineering Fracture Mechanics*, vol. 15, no. 1-2, pp. 185–192, 1981.
- [193] P. A. Wawrzynek, B. J. Carter, and L. Banks-Sills, "The M-integral for Computing Stress Intensity Factors in Generally Anisotropic Materials," tech. rep., National Aeronautics and Space Administration, 2005.
- [194] X. B. Lin and R. A. Smith, "Finite element modelling of fatigue crack growth of surface cracked plates: Part II: Crack shape change," *Engineering Fracture Mechanics*, vol. 63, no. 5, pp. 523–540, 1999.
- [195] J. H. Pang, K. S. Tsang, and H. J. Hoh, "3D stress intensity factors for weld toe semi-elliptical surface cracks using XFEM," *Marine Structures*, vol. 48, pp. 1–14, 2016.
- [196] M. Lukić, M. Al-Emrani, M. Aygül, M. Bokesjö, and et al., *Bridge fatigue guidance - meeting sustainable design and assessment (BRiFaG)*. European Commission: Directorate-General for Research and Innovation, 2013.

- [197] C. Cremona, "Reliability updating of welded joints damaged by fatigue," *International Journal of Fatigue*, vol. 18, no. 8, pp. 567–575, 1996.
- [198] T. D. Righiniotis and M. K. Chryssanthopoulos, "Fatigue and fracture simulation of welded bridge details through a bi-linear crack growth law," *Structural Safety*, vol. 26, no. 2, pp. 141–158, 2004.
- [199] W. Wu, M. H. Kolstein, and M. Veljkovic, "Stress intensity factors of the rib-to-deck welded joint at the crossbeam conjunction in OSDs," *Procedia Structural Integrity*, vol. 13, pp. 2017–2023, 2018.
- [200] prEN 1993-1-14, *Eurocode 3 - Design of steel structures — Part 1-14: Design assisted by finite element analysis (final draft)*. European Committee for Standardization, 2021.
- [201] I. Lillemäe, H. Remes, S. Liinalampi, and A. Itävu, "Influence of weld quality on the fatigue strength of thin normal and high strength steel butt joints," *Welding in the World*, vol. 60, pp. 731–740, 2016.
- [202] B. Jonsson, G. Dobmann, A. F. Hobbacher, M. Kassner, and G. Marquis, *IIW Guidelines on Weld Quality in Relationship to Fatigue Strength*. Springer Cham, 1 ed., 2016.
- [203] D. de Bree, "TOFD inspection report on selected areas of the Classification Trough cover plate C33B78 (in Dutch)," tech. rep., Infra Inspectie B.V., 2016.
- [204] J. P. Charlesworth and J. A. G. Temple, *Engineering Applications of Ultrasonic Time-of-Flight Diffraction*. Research Studies Press Ltd., 2 ed., 2002.
- [205] R. J. M. Pijpers, "Structural health monitoring for fatigue life prediction of orthotropic bridge decks," in *3rd Orthotropic Bridge Conference Proceedings* (C. A. Copelan and L. Duan, eds.), pp. 432–445, 6 2013.
- [206] NEN-EN 1993-2+C1:2011/NB:2011, *Dutch National Annex to Eurocode 3 - Design of steel structures - Part 2: Steel bridges (in Dutch)*. Nederlands Normalisatie Instituut (NEN), 2011.
- [207] A. Yazid, N. Abdelkader, and H. Abdelmadjid, "A state-of-the-art review of the X-FEM for computational fracture mechanics," *Applied Mathematical Modelling*, vol. 33, no. 12, pp. 4269–4282, 2009.
- [208] C. I. Kang, S. M. Choi, S. K. Kook, and D. U. Lee, "Analysis of Stress Behavior on Field Welded Joints of U-rib in Steel Bridge," *Journal of Korean Society of Steel Construction*, vol. 16, no. 3, pp. 387–396, 2004.
- [209] J. Saunders, Y. Chen, J. A. Marks, I. Hodgson, and R. Sause, "Finite Element Analysis and Full-Scale Laboratory Tests of Rib-to-Floor Beam Connections with Potential for Automated Fabrication," in *5th Orthotropic Bridge Conference Proceedings*, (Santa Clara), pp. 93–109, 8 2019.

- [210] A. Ahola, R. R. Hamidreza, T. Björk, and O. Kukkonen, "On the interaction of axial and bending loads in the weld root fatigue strength assessment of load-carrying cruciform joints," *Welding in the World*, vol. 66, pp. 731–744, 2022.
- [211] A. Ahola, T. Björk, and Z. Barsoum, "Fatigue strength capacity of load-carrying fillet welds on ultra-high-strength steel plates subjected to out-of-plane bending," *Engineering Structures*, vol. 196, 2019.
- [212] W. Song, X. Liu, and S. M. J. Razavi, "Fatigue assessment of steel load-carrying cruciform welded joints by means of local approaches," *Fatigue & Fracture of Engineering Materials & Structures*, vol. 41, pp. 2598–2613, 2018.
- [213] T. Mori, "Assessment of Fatigue Strength of Cruciform Welded Joints Failing from Weld Roots under Out-of-Plane Bending," tech. rep., International Institute of Welding, 2009.
- [214] R. J. Dexter and J. W. Fisher, "Fatigue cracking of orthotropic steel decks," tech. rep., IABSE reports, Zurich, 1997.
- [215] W. Wu, M. Veljkovic, M. H. Kolstein, R. J. M. Pijpers, and J. Maljaars, "Fatigue behaviour of root crack in stiffener-to-deck plate weld at crossbeam of orthotropic bridge decks," *Engineering Structures*, vol. 306, 2024.
- [216] E. Niemi, W. Fricke, and S. J. Maddox, *Structural Hot-Spot Stress Approach to Fatigue Analysis of Welded Components*. IIW Collection, Springer Singapore, 2 ed., 2018.
- [217] A. A. Bhatti, Z. Barsoum, H. Murakawa, and I. Barsoum, "Influence of thermo-mechanical material properties of different steel grades on welding residual stresses and angular distortion," *Materials and Design*, vol. 65, pp. 878–889, 2015.
- [218] J. Zhu, M. Khurshid, and Z. Barsoum, "Accuracy of computational welding mechanics methods for estimation of angular distortion and residual stresses," *Welding in the World*, vol. 63, pp. 1391–1405, 2019.
- [219] J. Sun, J. Klassen, T. Nitschke-Pagel, and K. Dilger, "Effects of heat source geometric parameters and arc efficiency on welding temperature field, residual stress, and distortion in thin-plate full-penetration welds," *International Journal of Advanced Manufacturing Technology*, vol. 99, pp. 497–515, 2018.
- [220] K. Hemmesi, M. Farajian, and M. Boin, "Numerical studies of welding residual stresses in tubular joints and experimental validations by means of x-ray and neutron diffraction analysis," *Materials and Design*, vol. 126, 2017.
- [221] D. Deng, "FEM prediction of welding residual stress and distortion in carbon steel considering phase transformation effects," *Materials and Design*, vol. 30, no. 2, pp. 359–366, 2009.
- [222] J. A. Goldak and M. Akhlaghi, *Computational Welding Mechanics*. Springer New York, NY, 1 ed., 2005.

- [223] K. Hemmesi, H. Holey, A. Elmoghazy, R. Böhm, M. Farajian, and V. Schulze, "Modelling and experimental validation of material deformation at different zones of welded structural-steel under multiaxial loading," *Materials Science & Engineering A*, vol. 824, 2021.
- [224] L. Feng, X. Qian, and R. Jiang, "A uniform volume-based fatigue indicator for high- and low-cycle assessment of notched components," *International Journal of Fatigue*, vol. 144, 2021.
- [225] Y. Wang, R. Fincato, K. Morita, Q. Wang, and S. Tsutsumi, "Cyclic elastoplasticity-based life assessment of fatigue crack initiation and subsequent propagation in rib-to-deck welded joints," *International Journal of Fatigue*, vol. 173, 2023.
- [226] K. Hussain, "Short fatigue crack behaviour and analytical models: A review," *Engineering Fracture Mechanics*, vol. 58, no. 4, pp. 327–354, 1997.
- [227] B. Marques, M. F. Borges, F. V. Antunes, J. M. Vasco-Olmo, F. A. Díaz, and M. N. James, "Limitations of small-scale yielding for fatigue crack growth," *Engineering Fracture Mechanics*, vol. 252, 2021.
- [228] U. Krupp, *Fatigue Crack Propagation in Metals and Alloys: Microstructural Aspects and Modelling Concepts*. Wiley-VCH; John Wiley, 2007.
- [229] M. Vormwald, P. Heuler, and C. Krae, "Spectrum Fatigue Life Assessment of Notched Specimens Using a Fracture Mechanics Based Approach," *Automation in Fatigue and Fracture: Testing and Analysis*, ASTM STP 1231, pp. 221–240, 1994.
- [230] V. Tvergaard, "On fatigue crack growth in ductile materials by crack-tip blunting," *Journal of the Mechanics and Physics of Solids*, vol. 52, no. 9, pp. 2149–2166, 2004.
- [231] F. Zhu, M. L. Zhang, C. J. Zhang, P. F. He, and Y. Dai, "Experimental investigation and crystal plasticity modeling of combined high and low cycle loading on AISI 4140 steel," *International Journal of Fatigue*, vol. 168, 2023.
- [232] U. Zerbst and J. Hensel, "Application of fracture mechanics to weld fatigue," *International Journal of Fatigue*, vol. 139, 2020.
- [233] P. L. Moore and G. Booth, *The Welding Engineer's Guide to Fracture and Fatigue*. Woodhead Publishing, 1 ed., 2014.
- [234] T. Belytschko and T. Black, "Elastic crack growth in finite elements with minimal remeshing," *International Journal for Numerical Methods in Engineering*, vol. 45, no. 5, pp. 601–620, 1999.
- [235] C. F. Shih and R. J. Asaro, "Elastic-Plastic Analysis of Cracks on Bimaterial Interfaces: Part I—Small Scale Yielding," *Journal of Applied Mechanics, Transactions ASME*, vol. 55, no. 2, pp. 299–316, 1988.
- [236] D. H. Malschaert, "Fracture mechanics: Application on orthotropic steel decks," master's thesis, Delft University of Technology, Delft, 2020.

- [237] A. R. Khoei, *Extended Finite Element Method: Theory and Applications*. John Wiley & Sons, Ltd., 1 ed., 2014.
- [238] K. Tanaka, "Fatigue crack propagation from a crack inclined to the cyclic tensile axis," *Engineering Fracture Mechanics*, vol. 6, no. 3, pp. 493–507, 1974.
- [239] ASTM E647-15, *Standard Test Method for Measurement of Fatigue Crack Growth Rates*. American Society for Testing and Materials, 2015.
- [240] H. Slot, M. Nicoreac, and J. Maljaars, "Influence of material anisotropy on fatigue crack growth in C–Mn steels of existing structures," *Fatigue & Fracture of Engineering Materials & Structures*, vol. 43, pp. 2527–2541, 2020.
- [241] C. F. Shih, B. Moran, and T. Nakamura, "Energy release rate along a three-dimensional crack front in a thermally stressed body," *International Journal of Fracture*, vol. 30, pp. 79–102, 1986.
- [242] V. Tvergaard and K. L. Nielsen, "Relations between a micro-mechanical model and a damage model for ductile failure in shear," *Journal of the Mechanics and Physics of Solids*, vol. 58, no. 9, pp. 1243–1252, 2010.

ACKNOWLEDGEMENTS

Being a doctoral researcher means a role change from a student who finishes assignments using knowledge in textbooks to an explorer who creates new knowledge. Both passion and patience are needed in this journey. Creating reliable and useful knowledge is not an easy task, especially when the topic is related to infrastructure that may have a large societal impact. It is my luck and honour to be able to join an excellent research team and focus on a worldwide challenge of steel bridges.

First, I would like to express my gratitude to promotor Prof. Milan Veljkovic and Prof. Johan Maljaars. Both of you put enormous effort into supervising my research and helped me develop my abilities. While giving sufficient freedom in research, insightful suggestions and feedback were always provided. The rigorous academic attitude and uncompromising spirit you showed in research, and the warm care and the patience you gave in daily cooperation, have had a profound impact on me.

Special acknowledgement goes to my previous co-promotor and daily supervisor Dr. Henk Kolstein. You patiently guided me towards the right and important research questions, and taught me to carry out high-quality experiments. Although it is not possible for you to continue serving as the co-promoter until my defence due to the time limit after your retirement, your contribution to my doctoral study is priceless.

I would like to thank the members of my doctoral committee: Dr. José Correia, Prof. Mohammad al-Emrani, Dr. Richard Pijpers, Prof. Bert Sluys, Prof. Andreas Taras, and Prof. Zili Li (reserve member) for their constructive comments and suggestions.

I am grateful to Ir. Frank van Dooren (Rijkswaterstaat) for the discussions during the project meetings. You always gave valuable advice, while remaining open to new solutions.

I am grateful for the help and support of my colleagues from the Stevinlaboratory: Kevin Mouthaan, John Hermsen, Kees van Beek, Fred Schilperoort, Arjen van Rhijn, and Louis den Breejen in the experiments.

I am very lucky to have such nice office mates. The laughter and discussions we had have created a relaxed and comfortable working atmosphere. Thank you Martin Nijgh, Haohui Xin, and Clarissa Justino!

Friendships in Delft with Weiyuan Zhang, Pingping Wen, Yuguang Yang, Ling Chang, Kaixuan Zhou, Jiapeng Yin, Han Dun, Lele Zhang, Youwei Wang, Ruxin Jing, Shizhe Zhang, Fei Yang, Rui Yan, Pei He, Lu Cheng, Xiuli Wang, Weikang Feng, Abhijith Kamath, and David Malschaert are deeply valued.

Last but not least, I would like to express my gratitude to my family for their endless love and encouragement.

Weijian Wu
2025

LIST OF PUBLICATIONS

Journal papers:

1. **W. Wu***, M.H. Kolstein, M. Veljkovic, *Fatigue resistance of rib-to-deck welded joint in OSDs, analyzed by fracture mechanics*, [Journal of Constructional Steel Research](#), **162**, (2019).
2. J. Maljaars*, R.J.M. Pijpers, **W. Wu**, M.H. Kolstein, *Fatigue resistance of rib to deck, crossbeam to deck and deck to deck welds in orthotropic decks using structural stress*, [International Journal of Fatigue](#), **175**, (2023).
3. **W. Wu***, M. Veljkovic, M.H. Kolstein, J. Maljaars, R.J.M. Pijpers, *Fatigue behaviour of toe and root stiffener cracks in stiffener-to-deck plate weld of orthotropic bridge decks*, [Engineering Structures](#), **305**, (2024) (**Featured Paper Award & Best Paper of the Year 2024 in Bridge Engineering**).
4. **W. Wu***, M. Veljkovic, M.H. Kolstein, R.J.M. Pijpers, J. Maljaars, *Fatigue behaviour of root crack in stiffener-to-deck plate weld at crossbeam of orthotropic bridge decks*, [Engineering Structures](#), **306**, (2024).
5. **W. Wu**, J. Maljaars, D.H. Malschaert*, M. Veljkovic, M.H. Kolstein, R.J.M. Pijpers, *Fatigue behaviour of toe crack at the lower end of crossbeam-to-stiffener weld with cut-outs in OBDs*, [Journal of Constructional Steel Research](#), **222**, (2024).
6. **W. Wu***, M. Veljkovic, J. Maljaars, M.H. Kolstein, *Fatigue evaluation of toe crack at the lower end of crossbeam-to-stiffener weld with cut-out considering welding residual stress*, [in preparation](#).
7. **W. Wu***, M. Veljkovic, J. Maljaars, M.H. Kolstein, R.J.M. Pijpers, *Fatigue behaviour of root crack in stiffener-to-crossbeam fillet weld without cut-out in OBDs*, [in preparation](#).

Conference papers:

1. **W. Wu***, M.H. Kolstein, M. Veljkovic, R.J.M. Pijpers, J. Vorstenbosch-Krabbe, *Fatigue behaviour of the closed rib to deck and crossbeam joint in a newly designed orthotropic bridge deck*, Proceedings of Eurosteel 2017, Copenhagen, Denmark, September 13-15, 2017.
2. **W. Wu***, M.H. Kolstein, M. Veljkovic, *Stress intensity factors of the rib-to-deck welded joint at the crossbeam conjunction in OSDs*, 22nd European Conference on Fracture (ECF22), Belgrade, Serbia, August 26-31, 2018.
3. **W. Wu***, M.H. Kolstein, M. Veljkovic, *Fatigue crack propagation analysis of rib-to-deck welded joint in OSDs using linear elastic fracture mechanics*, 40th IABSE Symposium in Nantes 2018: Tomorrow's Megastructures, Nantes, France, September 19-21, 2018.
4. **W. Wu***, H. Xin, M.H. Kolstein, M. Veljkovic, *3D Crack propagation analysis of longitudinal stiffener to crossbeam welded connection in OSDs*, 6th International Conference on Crack Paths (CP 2018), Verona, Italy, September 19-21, 2018.
5. **W. Wu***, M.H. Kolstein, M. Veljkovic, *Fatigue analysis of rib-to-deck welded joint in OSDs using linear elastic fracture mechanics*, 5th Orthotropic Bridge Conference Proceedings, Santa Clara, California, the United States, August 14-16, 2019.

Technical reports:

1. **W. Wu**, M.H. Kolstein, *Out-of-plane fatigue tests trough-crossbeam connection with 20 mm deck plate*, Stevin report, Delft University of Technology, (2020).
2. R.J.M. Pijpers, **W. Wu**, L. Abspoel, M.H. Kolstein, *Fatigue strength determination of the stiffener-to-deck plate weld, with cracks in the weld toe and/or in the weld root*, TNO report, (2022).
3. R.J.M. Pijpers, M. Nicoreac, **W. Wu**, M.H. Kolstein, *Fatigue strength determination of the stiffener-to-deck plate weld, weld root crack in 10-20mm deck plate at the cross-beam*, TNO report, (2022).

EDUCATION ACTIVITIES

Supervision of master theses:

1. **T. Sharma**, *Fatigue assessment of laser stake welded corrugated core steel sandwich deck under traffic loads*, master's thesis of Delft University of Technology, (2019).
2. **S. Gunecha**, *Corrugated core sandwich steel panel, quantifying performance improvement between empty and filled sandwich panels using J-Integral based local approach*, master's thesis of Delft University of Technology, (2019).
3. **D.H. Malschaert**, *Fracture Mechanics, Application on Orthotropic Steel Decks*, master's thesis of Delft University of Technology, (2020).
4. **S. Pandit**, *Fatigue behaviour of longitudinal open stiffener to crossbeam connections in orthotropic steel deck (OSD) bridges based on finite element modelling (FEM)*, master's thesis of Delft University of Technology, (2020).
5. **D. van der Ende**, *Fatigue behavior of the longitudinal to transverse stiffener connection in an orthotropic steel deck (OSD) based on shell and solid Finite Element Modeling*, master's thesis of Delft University of Technology, (2020).
6. **A. Elkazaz**, *Numerical investigation for cracks of rib-to-deck welded connection at the crossbeam junction in OSD using XFEM, a Crack Propagation Study Based on Linear Elastic Fracture Mechanics*, master's thesis of Delft University of Technology, (2021).

Teaching in master courses:

1. CIEQ5242 - Steel Bridges, 2016 (teaching assistant).
2. CIEQ5243A - Capita Selecta Steel and Aluminium, 2017 - 2021 (teaching assistant).
3. CIEQ5241 - Fatigue, 2017 - 2021 (lecturer for steel bridge details & finite element simulations & teaching assistant).

CURRICULUM VITÆ

Weijian Wu

1990.02.02	Born in Anhui, China
2008.09–2012.07	Bachelor of Engineering in Civil Engineering Hefei University of Technology, China
2012.09–2015.12	Master of Engineering in Architectural and Civil Engineering Tongji University, China
2016.03–now	Doctor in Structural Engineering Delft University of Technology, The Netherlands
2021.07–now	Research assistant (2021.07–2022.03) and postdoctoral researcher Technical University of Denmark, Denmark

