A FRAMEWORK FOR THE ASSESSMENT OF THE REMAINING FATIGUE LIFE OF STEEL BRIDGES WITH WELDED JOINTS

By

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ABSTRACT OF THE DISSERTATION

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In the United States, bridge owners rely on the AASHTO Manual for Bridge Evaluation (MBE) to estimate the remaining fatigue life of steel bridges. Although the manual allows refinements on the fatigue load model by including data from field measurements, the fatigue resistance model, which is based on the AASHTO LRFD S-N curves, has been shown to overestimate fatigue damage. Since the outcomes of the remaining fatigue life assessment may result in costly actions as repairing or replacing the deficient member, refinements on fatigue resistance models are desirable and could benefit bridge owners in making better decisions. Another area in the fatigue evaluation process is to consider the future impact of truck platoons on existing bridges.
A framework for applying deterministic as well as probabilistic fatigue assessment of welded joints is presented in this research considering three different approaches: 1) stress-life, 2) Linear Elastic Fracture Mechanics (LEFM), and 3) the UniGrow approaches. The current AASHTO MBE methodology was used as the benchmark model. The stress-life fatigue resistance model was developed based on the AASHTO fatigue database by refining and reformulating the current S-N curves as bilinear S-N curves for the fatigue detail categories E and E’. Due to the great variability of fatigue lives in the LEFM models caused by different initial crack sizes, this research proposes using the LEFM coupled with the Phased Array Ultrasonic Testing (PAUT) technique. The PAUT has enhanced capabilities of detecting flaws in comparison with other Non-Destructive Testing (NDT) methods. This research also applied the Unigrow model, which is based on the strain-life method and is capable of including the residual stress effects in the fatigue analysis. It was found that the remaining fatigue life obtained from the Unigrow model and from the LEFM coupled with PAUT have similar values. The proposed resistance models were applied to the welded joints of a steel bridge used as a case study. The fatigue loading for the case study was based on a calibrated Finite Element Model integrated with site-specific weigh-in-motion (WIM) measurements.

Additionally, in order to address the impact of truck platooning in the fatigue behavior of steel bridges, this research investigated the cumulative fatigue damage caused by truck platoons made of two, three, and four trucks. It was found that although the load effect caused by truck platoons is higher than a single truck, the number of fatigue stress cycles is reduced in certain cases. Therefore, for some cases, a truck within a platoon causes less fatigue damage than a single truck.
DEDICATION

To my wife.
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LIST OF SYMBOLS

\( A \) fatigue detail category constant.
\( A_{m1} \) S-N curve intercept for the region above the CAFL.
\( A_{m2} \) S-N curve intercept for the region below the CAFL.
\( A_1 \) random S-N curve intercept for the region above the CAFL.
\( A_2 \) random S-N curve intercept for the region below the CAFL.
\( a \) crack size.
\( a_0 \) initial crack size.
\( a_c \) or \( a_{cr} \) critical crack size.
\( a(t) \) crack size as a function of time \( t \).
\( B_F \) random error associate with the calculation of stress intensity factor.
\( b \) fatigue strength coefficient.
\( C \) fatigue crack growth constant.
\( c \) half crack-width (semi-elliptical cracks).
\( c_d \) fatigue ductility exponent.
\( D \) cumulative fatigue damage.
\( D(t) \) cumulative fatigue damage as function of time.
\( D_{AVG_{P,1}} \) average fatigue damage caused by one truck within a platoon.
\( D_F \) fatigue damaged caused by the AASHTO LRFD fatigue load.
\( D_{TRUCK} \) fatigue damage caused by the individual truck.
\( DLA \) dynamic load amplification.
\( E \) modulus of elasticity.
\( e_{WIM} \) random error associated with weigh-in-motion (WIM) measurements.
\( F(a) \) or \( F \) generalized correction factor for the stress intensity factor.
\( F_e \) crack shape correction factor.
$F_g$ stress concentration of stress gradient correction factor.

$F_s$ surface correction factor.

$F_w$ finite plate width or finite plate thickness correction factor.

$F_y$ yield strength of the material.

$f_r$ correction factor for the residual stress intensity factor.

$f_x(x)$ probability distribution function for the random variable $X$.

$G$ strain energy release rate.

$G_c$ correction factor for the stress intensity factor.

$g$ expected annual truck traffic growth.

$g(X)$ limit state function.

$J$ J-integral.

$j$ number of trucks within a platoon.

$K$ strength coefficient.

$K_c$ fracture toughness.

$K'$ cyclic strength coefficient.

$K_t$ stress concentration factor.

$K_{tm}$ maximum stress concentration factor.

$K_{max, appl}$ maximum applied stress intensity factor.

$K_{min, appl}$ minimum applied stress intensity factor.

$K_{max, tot}$ total maximum stress intensity factor.

$K_{min, tot}$ total minimum stress intensity factor.

$K_r$ residual stress intensity factor range.

$K_I$ stress intensity factor for the crack opening mode I.

$K_{I, a}$ stress intensity factor at the crack depth, $a$.

$K_{I, c}$ stress intensity factor at the half crack-width, $c$.

$L$ position along the crack path.

$M$ correction factor for the stress intensity factor.

$M_l$ moment range $i$. 
$M_{Fi}$ $i$th moment range caused by the AASHTO LRFD fatigue load.

$M_{Pi}$ moment range caused by the truck platoon.

$M_{Ti}$ $i$th moment range caused by the individual truck.

$m$ material constant.

$m_1$ S-N curve slope above the CAFL (finite life region).

$m_2$ S-N curve slope below the CAFL.

$n$ number of cycles per truck passage on the bridge.

$n_{a,i}$ number of cycles corresponding to stress range $S_{a,i}$.

$n_{b,j}$ number of cycles corresponding to stress range $S_{b,j}$.

$n_h$ strain hardening exponent.

$n'$ cyclic strain hardening exponent.

$n_i$ number of cycles produced by stress range $i$.

$n_{Fi}$ number of stress cycles for the stress range caused by the moment range $M_{Fi}$.

$n_{Pi}$ number of stress cycles for the stress range caused by $M_{Pi}$.

$n_{Ti}$ number of stress cycles for the stress range caused by the moment range $M_{Ti}$.

$N_{av}$ number of available cycles at the first service year of the detail.

$N_1$ number of cycles consumed over the present age of the detail.

$N_i$ number of cycles to failure corresponding to the stress range $i$.

$N$ or $N_f$ number of cycles to failure.

$N(t)$ deterministic number of cycles to failure as a function of time $t$.

$p$ material parameter for UniGrow model.

$Q$ square root of the complete elliptical integral of the second kind.

$R$ stress ratio.

$r_y$ plastic zone radius.

$S$ nominal stress.

$S_{a,i}$ stress range cycles above the CAFL.

$S_{b,j}$ stress range cycles below the CAFL.
$S_f$ fatigue damage parameter.

$S_{\text{max, appl}}$ maximum applied nominal stress.

$S_{\text{min, appl}}$ minimum applied nominal stress.

$S_{re}$ effective stress range.

$S'_{re}$ effective stress range according to bilinear S-N curve.

$S_{ri}$ stress range $i$.

$S_X$ elastic section modulus.

$t$ plate thickness.

$Y_{REM}$ remaining fatigue life in years.

$(Y_{ADTT})_{\text{LIMIT}}$ remaining fatigue life in years according to the $(ADTT_{SL})_{\text{LIMIT}}$.

$W$ strain energy density.

$w$ plate width or half-width.

$\beta$ reliability index.

$\beta_T$ target reliability index.

$\Delta$ critical fatigue damage accumulation.

$(\Delta f)_{\text{eff}}$ factored effective stress range according to AASHTO MBE notation.

$(\Delta f)_{\text{max}}$ factored maximum stress range according to AASHTO MBE notation.

$\Delta F_{TH}$ constant amplitude fatigue threshold or constant amplitude fatigue limit (CAFL).

$\Delta K_I$ stress intensity factor range for crack opening mode I.

$\Delta K_{\text{appl}}$ applied stress intensity factor range.

$\Delta K_{th}$ threshold stress intensity factor range.

$\Delta K_{tot}$ total stress intensity factor range.

$\Delta S$ nominal stress range.

$\Delta \sigma$ nominal normal stress range.

$\Delta \sigma_a$ actual stress range.

$\Delta \sigma^a$ actual stress range – UniGrow notation.

$\Delta \sigma^e$ elastic stress range – UniGrow notation.

$\Delta \varepsilon$ actual strain range.
$\Delta \varepsilon_e$ elastic strain range.
$\Delta \varepsilon_p$ plastic strain range.
$\Delta \varepsilon^a$ actual strain range – UniGrow notation.
$\varepsilon$ actual strain.
$\varepsilon_a$ total strain amplitude.
$\varepsilon'_f$ fatigue ductility coefficient.
$\bar{\varepsilon}^a_{\text{max}}$ maximum actual strain – UniGrow notation.
$\sigma$ nominal normal stress.
$\sigma_0$ maximum value for the tensile residual stress.
$\sigma_{\text{max}}$ maximum nominal normal stress.
$\sigma_{\text{min}}$ minimum nominal normal stress.
$\sigma_{r,T}$ distribution of transversal residual stress.
$\sigma_a$ actual stress.
$\sigma'_f$ fatigue strength coefficient.
$\bar{\sigma}^{a}_{\text{max}}$ actual maximum stress – UniGrow notation.
$\bar{\sigma}^e_{\text{max}}$ maximum elastic stress – UniGrow notation.
$\bar{\sigma}^e_{\text{min}}$ minimum elastic stress – UniGrow notation.
$\gamma$ material parameter for UniGrow model.
$\nu$ Poisson’s ratio.
$\theta$ weld flank angle.
$\rho$ weld toe radius.
$\rho^*$ elementary material block size – UniGrow notation.
$\psi_{y,1}$ average constant for the first elementary material block – UniGrow notation.
1.1 Background

The evaluation of fatigue damage or the prediction of the remaining fatigue life of a steel bridges is still a challenging and unsolved issue (Ye et al. 2014). Although the stress-life approach is the most popular method used for the fatigue assessment, it is well accepted in the literature that crack-based fatigue resistance model provides a more assertive predictions of the remaining fatigue life (Tsiatas et al. 2002 and Delkhosh et al. 2020).

Currently in the United States, the evaluation of the remaining fatigue life of steel bridges is based on the AASHTO Manual for Bridge Evaluation (MBE) provisions. On the load side, the manual allows different alternatives for the fatigue load models that varies from the application of the AASHTO LRFD BDS Fatigue Truck, to more advanced methods such as the use of weight-in-motion (WIM) measurements, or the implementation of structural health monitoring (SHM). On the resistance side, the MBE relies on the AASHTO LRFD BDS S-N curves, e.g., the stress life approach, as the single alternative for the fatigue resistance models. Since the outcomes of the remaining fatigue life assessment may result in costly actions as repairing or replacing the deficient member,
improvements on the fatigue models are desirable and could benefit bridge owners to make better decisions regarding the repair or replacement of fatigue critical weld details.

1.2 Problem Statement

The current AASHTO S-N curves were derived from linear regression models employed on a database with fatigue tests collected from various NCHRP projects and world-wide research (Keating and Fisher, 1986), under the assumption of constant standard error at different stress levels. The design S-N curve, referred as Minimum Fatigue Life curve in MBE, was formulated using a 95% lower confidence limit from the mean S-N curve (Bowman et al., 2012). Yet, many researchers have found that AASHTO S-N curves may lead to over conservative estimation of the remaining fatigue life of welded joints. One of the criticisms of the current AASHTO stress-life method is the extrapolation of the S-N curve below the constant amplitude fatigue limit (CAFL) with the same slope \( m = -3 \) of the finite life region that leads to overestimated fatigue damage. In order to address the over conservative predictions of remaining fatigue lives, MBE allows the extension of finite fatigue lives based on S-N curves with higher probability of failures, e.g., lower confidence intervals than the Minimum Fatigue Life. However, it is stressed that shifting the S-N curves under the assumption of constant standard error, is not reasonable since the number of cycles to failure at different stress levels have different standard deviation values.

It is also emphasized that failure definition attributed to the stress-life method varies among different fatigue tests which contributes to the scatter of S-N curves. According to ASTM E466 (2015), specimen failure for S-N tests may be defined as complete separation,
visible crack, crack of a specific dimension or by some other criterion. According to Hirt (1971) the failure criterion used in the steel beams tested in NCHRP Project 12-7 (Reports 102 and 147) was set as the increase of the midspan deflection of 0.02 inches which corresponded to a crack propagation between 15% to 75% of the flange area. Hence, the number of cycles to failure obtained from AASHTO S-N curves is not capable of informing the critical crack sizes attributed to the component failure and it does not differentiate crack initiation of crack propagation. It is noted that including crack information in fatigue resistance models, such as Linear Elastic Fracture Mechanics (LEFM) and UniGrow model, could provide a more objective and standardized criterion to define failure as well as extension of fatigue life up to a predefined critical crack size. Furthermore, this type of model is applicable for more complex details that might not have been categorized per AASHTO LRFD BDS classification details.

From the load side perspective, MBE recognizes that more accurate ways to calculate the number of cycles per truck passage on a bridge, could be used to improve the prediction of fatigue remaining life. Even though the Manual provides a factor to account for the multiple presence case, there is no discussion regarding truck platooning. Yarnold and Weidner (2019) stated that it is not a question of whether platooned trucks will impact our nation’s infrastructure, but when. The authors identified that pavement and bridges will be the most affected infrastructure due to truck platooning and research is needed.

1.3 Research Significance

This research presents a framework for applying deterministic as well as probabilistic fatigue assessment of welded joints according to the stress-life, Linear Elastic...
Fracture Mechanics (LEFM), and the UniGrow approaches. Focus is given to the component remaining fatigue life (fatigue detail) rather than the whole system fatigue life (whole bridge). Furthermore, based on the fatigue damage quantification, appropriate truck platooning configuration is recommended aiming to minimize the cumulative fatigue damage in steel bridges.

The database used for the formulation of the current AASHTO S-N curves was re-assessed and new S-N curves with a less steel slope below the CAFL were proposed based on the piecewise regression model. The variability of the number of cycles below and above the CAFL load level was considered.

As an alternative to the stress-life method, LEFM approach concerning the crack propagation phase was applied at different welded joints of a steel bridge used as case study. Since the variability on the assumptions of initial crack sizes found in the literature highly affects the prediction of the remaining fatigue life, this research proposes the application of LEFM model coupled with Phased Array Ultrasonic Testing (PAUT) technique. The PAUT technique has enhanced capabilities of detecting flaws in comparison with other Non-Destructive Testing methods and it was applied on critical weld details of the studied bridge.

In order to address some of the limitations of the LEFM models, such as the prediction of the crack growth near the threshold stress intensity factor region, the UniGrow approach was employed for the fatigue assessment of welded joints. This method is based on the local stress-strain relationship at the crack tip and is capable of addressing crack initiation and propagation phase. Other important aspects of the fatigue behavior such as the residual stresses and crack closure effects are also considered in the model. It
is noted that there is limited literature regarding the use of local-global approaches for the fatigue assessment of steel bridges.

On the fatigue load side, analytical investigation on the number of fatigue stress cycles produced by the passage of a truck platooning was developed. The cumulative fatigue damage was evaluated based on the size of the truck platooning, the steel bridge boundary condition, and the span length. Appropriate platooning configurations were recommended in order to minimize the cumulative fatigue damage.

The research scope is summarized in Figure 1.1.

**1.4 Objectives**

The main objective of this research is to propose a refined framework for the assessment of the remaining fatigue life of steel bridges with welded joints. Additionally, the impact of truck platooning on the fatigue damage of steel bridges is evaluated. In order to accomplish the main objective, the partial objectives are defined as following:
1. Re-evaluation of experimental fatigue test data used to formulate the current AASHTO S-N curves and the development of S-N curves based on Piecewise Regression model in order to assess the curve slope below the CAFL;

2. Assessment of the remaining fatigue life of a steel bridge with welded joints according to the stress-life and LEFM methodologies, using the deterministic and probabilistic approaches;

3. Application of LEFM models by considering initial crack sizes based on PAUT results;

4. Application of the UniGrow model for the assessment of the remaining fatigue life;

5. Comparison of results for the remaining fatigue lives among the different methodologies and recommendation of different tiers of analysis according to each model capability;

6. Estimation of the cumulative damage caused by trucks platoons and scenarios when truck platoon minimizes the fatigue damage.

1.5 Organization of the Thesis

This thesis contains eight chapters. This first chapter serves as an introduction outlining the problem statement, research significance and definition of the objectives.

Literature review is presented in Chapter two. Particularities of fatigue in welded joints are presented and the AASHTO LRFD and MBE stress-life methodology are revised. The background for the crack-based fatigue resistance models, such as the LEFM and UniGrow model are discussed. The prevailing Non-Destructive Testing techniques to detect fatigue cracks in steel bridges were compared with an enhanced ultrasonic method called Phased Array Ultrasonic Testing (PAUT). Additionaly, the background of the
reliability theory was presented. Lastly, previous research regarding the effects of truck platooning on steel bridges was reviewed.

Chapter three describes the steel bridge used as a case study and the implementation of WIM and SHM. Also, the calibration of the Finite Element model was presented.

Chapter four presents the fatigue assessment performed in the studied bridge based on the stress-life method. In this chapter the experimental fatigue test data used to formulate the AASHTO S-N curves was revised, and new S-N curves were proposed for AASHTO detail categories E and E’.

Chapter five focused on crack-based fatigue models to assess the remaining fatigue life. Four tiers of analysis were performed: 1) LEFM with a range of initial crack sizes; 2) LEFM with PAUT; 3) UniGrow with a range of initial crack sizes; and 4) UniGrow with PAUT.

The reliability-based fatigue assessment considering the stress-life and LEFM approaches was developed in chapter six.

Chapter seven contains the investigation regarding the cumulative fatigue damage produced by truck platoons.

Chapter eight contains the summary and conclusions of this thesis.
CHAPTER 2

LITERATURE REVIEW

The literature review presents the methodologies used to assess the remaining fatigue life of welded joints in steel bridges based on the stress-life method, LEFM and Unigrow model. The prevailing NDT methods to detect fatigue cracks in steel bridges are revised and the advantages of PAUT technique are discussed. The background of the reliability theory is revised. Lastly, previous research on the impact of truck platooning on bridges and its effects on the fatigue behavior of steel bridge is presented.

2.1 Singularities of fatigue in welded joints

Fatigue resistance of welded joints is not as good as other general metals due to three factors: 1) high tensile residual stresses; 2) severe notch effect due to the weld geometry; and 3) presence of imperfections or crack-like defects resulted from the welding process. To illustrate how these factors affect the fatigue resistance of a welded joint, the S-N curve of a smooth specimen, a plate with a bore hole and a plate with fillet welds are depicted in Figure 2.1. It is observed that the plate with the weld attachment has significantly shorter fatigue life than the other cases.
Figure 2.1. Effect of welding in fatigue strength (Maddox, 2000).

The residual stresses found in welded joints are caused by the different thermal strain induced in the weld and base metal after the welding process. An example of longitudinal residual stresses is shown in Figure 2.2.

Figure 2.2. Residual stresses due to welding (Maddox, 2011).

Since there is great uncertainty related to the distribution of residual stresses (Chyssanthopoulos and Righiniotis, 2006), usually it is conservatively assumed as the yield strength of the material for fatigue assessment purposes (Gurney 1979, Fisher 1984 and
Maddox 1991). As a result, the fatigue life of welded joints is independent of the mean stress and depends only on the applied stress range, (Gurney 1979, Maddox 1991 and Fisher 1997).

Welds introduce discontinuities in plate geometries that causes local stress concentration. The stress concentration factor, $K_t$, is defined as the ratio between the peak stress at the root of the notch and the nominal stress which would be present if a stress concentration did not occur, considering linear elastic material behavior (Schijve, 2009). According to BS 7608:2014, nominal stress is the structure stress that would exist in the absence of the structural discontinuity being considered. One example of stress concentration due to double-sided full penetration butt weld is depicted in Figure 2.3. In this case, the stress rise occurs at the weld toe. It is noted that the severity of notch stress concentration is governed by the notch geometry, for example the flank angle $\theta$ and toe radius $\rho$ (see Figure 2.3).

![Figure 2.3. Stress concentration in a double-sided full penetration butt weld detail.](image)

Severe stress concentration combined with the inherent flaws from the welding process yield to fatigue cracks that are already nucleated (Maddox, 2011). As a result, crack initiation life is zero and fatigue life is attributed to the propagation phase. Examples of discontinuities that are treated as initial crack sizes are related to lack of fusion, partial penetration, porosity or slag inclusions, as shown in Figure 2.4. It is noted
that discontinuities are commonly found in welded joints of steel bridges built prior to 1970 that lacked proper quality control (Russo et al. 2016), and especially in secondary structural members such as longitudinal stiffener that did not require nondestructive test to comply with an established weld quality criterion (Fisher, 1977).

However, much debate exists in the literature regarding the boundary that differentiate crack initiation than propagation (Stephens et al., 2001). Schilling and Klippstein (1978) concluded that crack initiation phase is an important part of fatigue life for cover plate details in beams made of A514-steel. Pereira Batista (2016) found that neglecting the initiation life under variable amplitude fatigue loading is a conservative approach. Lassen and Recho (2006) stated that initiation phase is present in the fatigue damage process. Thus, when crack-like discontinuities do not exist in welded joints, crack initiation and propagation phase might be included in the fatigue life assessment (Stephens et al., 2001).
2.2 Stress-life method to assess remaining fatigue life

Fatigue in steel bridges is classified by AASHTO LRFD BDS into load-induced fatigue and distortion-induced fatigue. Load-induced fatigue is defined as the fatigue in a structural component caused by in-plane stresses whereas distortion-induced fatigue occurs due to out-of-plane stresses. Usually the design is performed for the load-induced case and proper structural detailing is intended to avoid distortion-induced fatigue. The revision on fatigue design and evaluation herein is focused on load-induced fatigue of welded joints.

2.2.1 AASHTO LRFD BDS S-N curves

The current AASHTO S-N curves were derived from simple linear regression models employed on a database with several fatigue tests datapoints collected from various NCHRP projects and world-wide research (Keating and Fisher, 1986). For instance, NCHRP Reports 102, 147, 188, 206, 227, 267 and 354 show the fatigue test results performed in full scale beams under constant and variable amplitude fatigue loading. For tests performed under VAL, the number of cycles to failure, $N$, was converted to an equivalent constant amplitude effective stress range, $S_{re}$, that produces the same fatigue damage as the variable amplitude stress spectrum. In NCHRP Report 188, for instance, the correlation of variable-amplitude data points to the constant amplitude S-N curves was performed by calculating the effective stress range given in Equation 2.1, where $\alpha_i$ is the frequency of occurrence of the stress range $S_{ri}$.

$$S_{re} = \left( \sum \alpha_i \times S_{ri}^B \right)^{1/B}$$  \hspace{1cm} (Equation 2.1)
If $B$ is taken as 2, the Root-Mean-Square (RMS) of the stress range in the spectrum is obtained. If $B$ is equal to 3, the Root-Mean-Cube (RMC) stress range is obtained. The RMC stress-range calculation is equivalent to applying the cumulative fatigue damage proposed by Palmgren and Miner’s, which is still the dominant rule applied for fatigue analysis (Stephens et al., 2001). The Palmgren-Miner’s rule is shown in Equation 2.2, where $n_i$ is the number of cycles produced by the stress range $i$ ($S_{ri}$) and $N_i$ is the number of cycles to failure to the corresponding $S_{ri}$.

$$D = \frac{\sum n_i}{N_i} = 1$$  \hspace{1cm} (Equation 2.2)

After the fatigue database was compiled, eight fatigue detail categories (A, B, C, C’, D, E and E’) were formulated based on the similarities of fatigue lives found in the experimental results. Category A represents the most resistant fatigue details, whereas Category E’ represents the weakest details. Figure 2.5 illustrates the experimental data for coverplated beams that are classified as category E as well as all the design S-N curves provided in AASTHO LRFD. It is noted that the design S-N curve, referred as Minimum Fatigue Life curve in MBE, were formulated using a 95% lower confidence limit from the mean S-N curve (Bowman et al., 2012).

![Fatigue data for Coverplated beams](image1)

![AASHTO S-N curves](image2)

(a) Fatigue data for Coverplated beams  \hspace{1cm} (b) AASHTO S-N curves

Figure 2.5. Development of AASHTO S-N curves (Keating and Fisher, 1986).
The finite life region of the AASHTO S-N curves (continuous line in Figure 2.5b), follows Basquin equation as shown in Equation 2.3, where \( S_r \) is the constant amplitude nominal stress range, \( m \) is the empirical material constant set as \( m = -3 \) for all category details, and \( A \) is the detail category constant that represents the curve intercept in log-log scale.

\[
N \times S_r^m = A
\]  
(Equation 2.3)

The horizontal dashed lines in in Figure 2.5b represent the constant amplitude fatigue threshold, \( \Delta F_{TH} \), called elsewhere as constant amplitude fatigue limit (CAFL). If \( S_r < \Delta F_{TH} \) the fatigue detail has infinite fatigue life. In the case of variable amplitude loading though, if the maximum stress range observed in the spectrum is higher than the CAFL, the detail has finite fatigue life, even though \( S_{re} < \Delta F_{TH} \) (Russo et al., 2016). In this scenario, the finite region of the S-N curve is extended below the CAFL with the same slope of \( m = -3 \), as seen in Figure 2.5b.

Previous research focused on AASHTO S-N curves suggest that slope below the fatigue limit should be less steep than the finite life region, i.e., \( m < -3 \). Crudele and Yen (2006) analytically examined the extension of S-N lines below the CAFL for AASHTO detail categories B, C, D and E. The analysis was based on the crack propagation caused by different stress spectrum in the region of the stress intensity factor threshold \( \Delta K_{th} \). The authors concluded that a slope of \( m = -4 \) below the CAFL appears to be suitable for the assessment of fatigue life. Various researchers adopted Crudele and Yen recommendation for fatigue assessment of steel bridges. Kwon et al. (2012) concluded that the bilinear AASHTO S-N curve with slopes of \( m = -3 \) and \( m = -4 \), below the fatigue limit, can effectively be used to estimate fatigue life. Yet, the authors suggested experimental
validation of bilinear S-N curves. Yen et al. (2013) developed an expression for the calculation of the effective stress range considering a bilinear S-N curve and concluded that this methodology is less conservative than the current AASHTO S-N curves. Wang et al. (2015) evaluated the remaining fatigue life of welded joints of a railway bridge and concluded that AASHTO S-N curves lead to more conservative estimation than other bilinear S-N curves proposed elsewhere. Lou et al. (2017) proposed improvements in S-N curve by adding new experimental points that had not been used before and applying a Piecewise Regression rather than the Simple Linear Regression model.

Although bilinear AASHTO S-N curves have been proposed in the literature, the lack of experimental validation is an obstacle for its application. Moreover, data to support a less steep S-N curve in the region below the CAFL is expensive and demands fatigue tests at the very-high cycle fatigue (VHCF) behavior ($N = 1 \times 10^7$ to $N = 1 \times 10^{10}$). According Wang et al. (2012), fatigue testing more than $N = 1 \times 10^7$ is not a standard practice due to time and cost constraints. The author mentioned that accelerated fatigue testing through more modern machines is applicable for testing specimens at VHCF. However, it is stressed that most of AASHTO S-N curves were based on full scale steel beams, since the scale factor is determinant on fatigue lives. Therefore, besides the time and cost, the availability of a proper machine to perform VHCF might be an additional constraint.

As a conclusion, the extrapolation of the AASHTO S-N curve below the CAFL with the same slope of the finite life region has been criticized to be conservative.
2.2.2 Remaining Fatigue Life based on MBE

According to MBE provisions, if the maximum nominal stress range, \((\Delta f)_{max}\), is larger than the constant amplitude fatigue threshold, \((\Delta F)_{TH}\), the detail has finite fatigue life. In this case, the estimation of fatigue life in terms of number of cycles is performed based on the difference between the number of initially available fatigue stress cycles, \(N_{av}\), and the number of consumed fatigue stress cycles, \(N_1\), over the present age of the detail. The AASHTO S-N curves are employed to compute \(N_{av}\) where different levels of safety can be used by considering 5%, 15.9%, 32.9% and 50% probability of failure that corresponds to Minimum, Evaluation 1, Evaluation 2 and Mean Life curves, respectively (Bowman M., et al., 2012). The remaining fatigue life in terms of years is then accounted based on the number of cycles created per truck passage on the bridge, \(n\); the average daily truck traffic in a single lane in the present age, \((ADT_{SL})_{PRESENT}\); the average daily truck traffic in a single lane in the first year the detail was in service \((ADT_{SL})_0\); and the expected annual truck traffic growth rate, \(g\) (see Equation 2.4).

\[
Y_{REM} = \frac{\log \left( \frac{g}{1+g} \left( \frac{N_{av} - N_1}{365n(ADT_{SL})_{PRESENT}} \right) + 1 \right)}{\log (1 + g)} \quad \text{(Equation 2.4)}
\]

The expected annual truck traffic growth, \(g\), should be constrained by the physical limits of the bridge under investigation. According to AASHTO LRFD Commentary C.3.6.1.4.2, research has shown that the average daily traffic \((ADT)\), including all vehicles, e.g., cars and trucks, is physically limited to about 20,000 vehicles per lane per day under normal conditions. Therefore, whenever non-zero growth rates are assumed in the calculations of the remaining fatigue life, there is a need to check whether \((ADT_{SL})_{FUTURE} \leq (ADT_{SL})_{LIMIT}\) according to the Equation 2.5, where:
\((ADTT_{SL})_{FUTURE}\) is the average number of trucks per day in a single lane in the year when the life corresponding to year \(Y_{REM}\) is reached; and \((ADTT_{SL})_{LIMIT}\) is the highway design maximum average number of trucks per day in a single lane for the roadway under consideration.

\[
(ADTT_{SL})_{FUTURE} = [(ADTT_{SL})_{PRESENT}](1 + g)^{Y_{REM}} \tag{Equation 2.5}
\]

If \((ADTT_{SL})_{FUTURE} > (ADTT_{SL})_{LIMIT}\), there is a need to calculate \((Y_{ADTT})_{LIMIT}\), as presented in Equation 2.6, which is the number of years from the present day until \((ADTT_{SL})_{LIMIT}\) is reached.

\[
(Y_{ADTT})_{LIMIT} = \frac{\log \left( \frac{(ADTT_{SL})_{LIMIT}}{(ADTT_{SL})_{PRESENT}} \right)}{\log(1 + g)} \tag{Equation 2.6}
\]

In this scenario, the estimation of the remaining fatigue life, \(Y_{REM}\), needs to be updated as Equation 2.7.

\[
Y_{REM} = \frac{N_{av} - N_1}{365n(ADTT_{SL})_{LIMIT}} - \frac{(1 + g)^{(Y_{ADTT})_{LIMIT}} - 1}{g(1 + g)^{(Y_{ADTT})_{LIMIT}} - 1} + (Y_{ADTT})_{LIMIT} \tag{Equation 2.7}
\]

Remaining fatigue life assessment based on MBE stress life method might result in negative lives due to over conservative assumptions. In order to address this issue MBE allows the extension of finite fatigue lives based on S-N curves with higher probability of failures, e.g., lower confidence intervals than the Minimum Fatigue Life. Other alternatives include improvements on fatigue load models such as: field measurements of stress range at the fatigue prone detail; 3D Finite Element analysis of stresses at the fatigue prone detail; weigh-in-motion data at or near the bridge site; site-specific data on \(ADTT\) at or near the bridge site; calculation of the number of cycles per truck passage on the bridge, \(n\), using influence lines or field measurements. Additionally, MBE provides updating equations for
the remaining fatigue life if previous field inspections show no evidence of fatigue cracking at the detail.

2.3 Linear Elastic Fracture Mechanics approach

The LEFM approach is concerned with the macrocrack propagation phase, where the elastic stress field in front of a crack tip is characterized through the stress intensity factor parameter. After the proper stress intensity factor is defined, a crack propagation law is used to evaluate the number of cycles for the crack to propagate from its initial size to the critical size.

2.3.1 Stress Intensity Factors

The stress intensity factor (SIF) is dependent on the crack opening direction and other correction factors that account for the specimen geometry and crack size and shape. According to Hobbacher (2011) and Lassen and Recho (2006), the crack opening perpendicular to the main stress, called mode I, is the most important mode for welded joints. Therefore, only the stress intensity factors for mode I are discussed herein.

For a body subjected to distributed stresses a general expression for the stress intensity factor is presented in Equation 2.8 (Albrecht and Yamada, 1977), where \(a\) is the crack size, \(\sigma\) is the tension stress, and \(F\) is the generalized correction factor that is dependent on the crack size and the specimen geometry.

\[
K = F \cdot \sigma \sqrt{\pi a}
\]  
(Equation 2.8)
According to Zettlemoyer et al. (1977), Albrecht and Yamada (1977), and Fisher (1984), the factor $F$ from Equation 2.8 can be subdivided into four correction factors that account for: surface crack (factor $F_s$), finite plate width/thickness (factor $F_w$), crack shape correction (factor $F_e$) and stress concentration correction (factor $F_g$). Hence, for the crack opening mode I, Equation 2.8 is rewritten in Equation 2.9.

$$K_I = F_s F_w F_e F_g \sigma \sqrt{\pi a}$$

(Equation 2.9)

It is noted that the crack size $a$ in Equation 2.9 have different meanings according to the type of crack. For instance, Figure 2.6 shows three cases of crack in a plate subjected to tension stress. For the semi-elliptical crack, the semi-axes $a$ represents the crack depth and the semi-axes $c$ represents the crack half-width. Although the crack-front is a semi-ellipse, usually the highest SIF occurs at crack tips $a$ and $c$. The angle $\phi$ is the parametric angle for the ellipse and it is useful to calculate the different SIFs along the crack front. Since points $a$ and $c$ are the most important ones, crack propagation is investigated coupling the growth of the crack depth $a$ and crack half-width $c$ (Figure 2.6a depicts the crack growth in dashed lines). Figure 2.6b presents a through-thickness where the crack propagation is considered by the growth of the half of the crack width $a$ along the plate width, as shown in the dashed lines. Lastly, a single edge crack is shown in Figure 2.6c. It is important to observe that the notation for plate width is $2w$ for cases (a) and (b), and $w$ for case (c). This notation matters when calculating stress intensity factors.
Various stress intensity factors expressions for plates under tension/bending and other cases can be found in Tada et al. (1973, 1985), Paris and Sih (1965), Hobbacher (2016) and Newman and Raju (1979, 1981). The latter authors have developed a set of SIF equations based on numerical analysis which have been used as a reliable reference. For instance, BS7910 (2005) and the International Institute of Welding refer to Newman and Raju expressions.

2.3.1.1 Surface correction factor $F_s$

According to Zettlemoyer (1976), the surface correction factor is dependent whether the displacements at the free boundary where the crack is located are indeed not restrained by any type of attachments or stiffeners. When the effect of displacement restriction is not considered, the free edge correction factor $F_s$ for a single or double edge crack in a finite plate is assumed to be 1.12 (Anderson 2005, Barsom el al. 1999 and
According to Fisher (1984), a common expression to correct the front free surface of semi-elliptical cracks is given in Equation 2.10.

\[ F_s = 1.211 - 0.186 \sqrt{\frac{\alpha}{c}} \]  

(Equation 2.10)

where \( \alpha \) is the crack depth and \( c \) the crack half-width.

2.3.1.2 Finite plate width or back surface correction factor \( F_w \)

Paris and Sih (1965) derived the \( F_w \) factor expression considering a center cracked specimen, according to Equation 2.11a.

\[ F_w = \sqrt{\frac{2w}{\pi a} \tan \left( \frac{\pi a}{2w} \right)} \]  

(Equation 2.11a)

Many other solutions for \( F_w \) were compared to Equation 2.11a in Tada et al. (1973, 1985). One of the most accurate expressions within 0.5% error when \( \alpha/c \leq 0.7 \) was proposed by Brown (1966), as given in Equation 2.11b.

\[ F_w = \sqrt{\sec \left( \frac{\pi \alpha}{2w} \right)} \]  

(Equation 2.11b)

In both Equations 2.11a and 2.11b, \( w \) is half of the plate width. In the cases where the crack propagates along the plate thickness, the half-width \( w \) shall be substituted by the plate thickness \( t \). According to Zettlemoyer (1976), these two expressions are often cited in the literature as the finite plate width, or back surface, correction factor. The author also discussed that bending amplifies \( F_w \) effect, especially when the crack has propagated more than half of the plate thickness. For instance, for an infinite strip \( (L/w > 1) \) of width \( w \) with an edge crack \( a \) (as Figure 2.6c), the SIF expression considering bending effects is
given in Equation 2.11c (Tada et al. 973, 1985). It is noted that this expression includes the free surface as well as the back surface corrections.

\[
K_I = \sigma \sqrt{\pi a} \times \left\{ 0.752 + 2.02 \left( \frac{a}{w} \right) + 0.37 \left[ 1 - \sin \left( \frac{\pi a}{2w} \right) \right]^3 \right\} \sec \left( \frac{\pi a}{2w} \right) \sqrt{2w} \tan \left( \frac{\pi a}{2w} \right)
\]  

(2.11c)

Leander et al. (2016) used Equation 2.11c to assess the crack propagation of an edge crack on the flange of a steel I-girder. The crack initiated due to a lateral gusset plate connection to the girder bottom flange.

The bending effects on \( F_w \) amplification may be neglected in some cases, such as the web plates of girders where the bending is constraint by stiffeners.

2.3.1.3 Crack shape correction factor \( F_e \)

The crack shape correction factor \( F_e \) is given in Equation 2.12. This factor adjusts the shape of the crack front which is often assumed to be elliptical (Zettlemoyer et al. 1977). The term \( \sqrt{Q} \) is the complete elliptical integral of the second kind.

\[
F_e = \frac{1}{\sqrt{Q}}
\]  

(Equation 2.12)

According to Newman and Raju (1981) a useful approximation for \( Q \) is given in Equation 2.13.

\[
Q = \begin{cases} 
1 + 1.464 \left( \frac{a}{c} \right)^{1.65} & \text{for } \frac{a}{c} \leq 1 \\
1 + 1.464 \left( \frac{c}{a} \right)^{1.65} & \text{for } \frac{a}{c} > 1 
\end{cases}
\]  

(Equation 2.13)

2.3.1.4 Surface semi-elliptical crack by Newman and Raju
As mentioned before, the developed SIF by Newman and Raju are often referred as reliable references. Equations 2.14 and 2.15 present the SIF for a surface semi-elliptical crack in a finite plate of of width $2w$ and thickness $t$, where $2c < 2w$ and $a < t$, (as shown in Figure 2.7). It is noted that the correction factors corresponding to $F_s$, $F_w$ and $F_e$ are embedded in the following expression.

![Figure 2.7. Semi-elliptical surface crack in a finite rectangular plate.](image)

$$K_{t,a} = F \cdot \frac{M}{\sqrt{Q}} \cdot \sigma \sqrt{\pi a}$$ \hspace{1cm} (Equation 2.14)

$$K_{t,c} = F \cdot \frac{M}{\sqrt{Q}} \cdot G_c \cdot \left(\frac{a}{c}\right) \sigma \sqrt{\pi c}$$ \hspace{1cm} (Equation 2.15)

The term $1/\sqrt{Q}$ is equal to the factor $F_e$, which is given in Equation 2.12. The factors $F$, $M$ and $G_c$ are given in Equations 2.16, 2.17 and 2.18, respectively.

$$F = \sqrt{\sec \left(\frac{\pi c}{2w} \cdot \sqrt{\frac{a}{t}}\right) \cdot \left[1 - 0.025 \left(\frac{c}{w} \sqrt{\frac{a}{t}}\right)^2 + 0.06 \left(\frac{c}{w} \sqrt{\frac{a}{t}}\right)^4\right]}$$ \hspace{1cm} (Equation 2.16)

$$M = M_1 + M_2 \left(\frac{a}{t}\right)^2 + M_3 \left(\frac{a}{t}\right)^4$$ \hspace{1cm} (Equation 2.17)

$$G_c = \begin{cases} 
1.1 + 0.35(a/t)^2 & \text{for } a \leq c \\
1.1 + 0.35(c/a)(a/t)^2 & \text{for } a > c 
\end{cases}$$ \hspace{1cm} (Equation 2.18)

The terms $M_1$, $M_2$ and $M_3$ are provided below, for cases when $a \leq c$ and $a > c$. 
For $a \leq c$:

$$M_1 = 1.13 - 0.09 \left( \frac{a}{c} \right) \quad \text{for } a \leq c \quad \text{(Equation 2.19)}$$

$$M_2 = -0.54 + \frac{0.89}{0.2 + \left( \frac{a}{c} \right)} \quad \text{for } a \leq c \quad \text{(Equation 2.20)}$$

$$M_3 = 0.5 - \frac{1}{0.65 + \left( \frac{a}{c} \right)} + 14 \left( 1 - \frac{a}{c} \right)^{24} \quad \text{for } a \leq c \quad \text{(Equation 2.21)}$$

For $a > c$:

$$M_1 = \left( \frac{c}{a} \right) + 0.04 \left( \frac{c}{a} \right)^2 \quad \text{for } a < c \quad \text{(Equation 2.22)}$$

$$M_2 = 0.2 \left( \frac{c}{a} \right)^{4.5} \quad \text{for } a < c \quad \text{(Equation 2.23)}$$

$$M_3 = -0.11 \left( \frac{c}{a} \right)^{4.5} \quad \text{for } a < c \quad \text{(Equation 2.24)}$$

2.3.1.5 Stress gradient correction factor $F_g$

The stress gradient correction factor $F_g$ accounts for the variation of stresses or stress concentrations caused by the detail geometry. According to Fisher (1984), for bridge applications $F_g$ is often given in terms of the crack size, $a$, and the maximum stress concentration factor, $K_{tm}$. A practical method to determine $F_g$ was proposed by Albrecht and Yamada (1977) by the application of the Green’s function or SIF superposition approach. This method utilizes the stress distribution of the uncracked body on the straight direction where the crack is expected to grow. Considering a through-thickness crack in an infinite plate under uniform tension stress, the stress intensity factor based on the Green’s function is given in Equation 2.25.
\[ K = \sigma \sqrt{\pi a} \times \frac{2}{\pi} \int_{0}^{a} \frac{K_t}{\sqrt{a^2 - L^2}} \, dL \]  
(Equation 2.25)

where \( K_t \) is the stress concentration factor of the uncracked body at position \( L \) and \( a \) is the crack size. The position \( L \) is given along the expected straight crack path where the crack origin is taken as \( L = 0 \). Since \( \sigma \sqrt{\pi a} \) is the stress intensity factor for the through-thickness crack in an infinite plate, the stress gradient correction factor results as Equation 2.26.

\[ F_g = \frac{2}{\pi} \int_{0}^{a} \frac{K_t(L)}{\sqrt{a^2 - L^2}} \, dL \]  
(Equation 2.26)

A numerical solution for Equation 2.26 was developed by Albrecht and Yamada (1977) which is presented herein in Equation 2.27 (Zettlemoyer et al. 1977).

\[ F_g = \frac{2}{\pi} \sum_{i=1}^{m} K_{t,i} \left[ \arcsin \left( \frac{L_{i+1}}{a} \right) - \arcsin \left( \frac{L_i}{a} \right) \right] \]  
(Equation 2.27)

where:

- \( K_{t,i} \) is the stress concentration factor in the element \( i \) from the Finite Element Model analysis, or the average between two adjacent elements with same edge size along the crack path;
- \( L_i \) is the distance from the crack origin to the near side of the finite element \( i \);
- \( L_{i+1} \) is the distance from the crack origin to the far side of the finite element \( i \);
- \( a \) is the crack size considered for the calculation of the stress gradient correction factor.

As an alternative, the code BS7910 (2005) provide a series of solutions for the stress concentration correction factor, named in the code as \( M_k \) in lieu of \( F_g \). According to Hobbacher (2011), \( M_k \) is determined based on the solution of Equation 2.26, where the non-linear speak stress is taken from the surface through the plate thickness along the
anticipated crack path, which is analogous to $F_g$ calculation. This method is also known as weight function approach.

2.3.1.6 Stress Concentration Factor Effects on $F_g$

The normal stresses distribution in plates at weldments location is highly affected by the weld geometry that causes local stress concentration. Therefore, in order to properly compute $F_g$ in SIF expression, the stress concentration analysis along the expected crack path is required.

Zettlemoyer et al. (1977) developed specific two-dimensional FE model according to the detail geometry in order to extract the stress concentration factors at each element located in the anticipated crack path. According to Pang (1991), when 2D models are used rather than 3D models, comparative calculations show that there could be an error up to 9% in terms of SIF, but on the conservative side.

Based on NCHRP Report 227 findings, the stress concentration factor for the fillet welded web attachments varies between 7.0 and 8.0. According to Norris et al. (1981) the stress concentration factor for this type of detail was found to vary between 5.5 and 7.0. It is observed that both studies did not discuss the effect of the weld toe radius $\rho$. Various publications show that as the weld toe radius increase the stress concentration factor decrease, and therefore, the fatigue life is improved (Pang 1994; Lee et al. 2009, Molski et al. 2020).

Recently, Molski et al. (2020) investigated the stress concentration factor for a cruciform welded joint under axial loading through finite element modeling. The authors stated that the weld toe radius $\rho$ is the most important parameter influencing the stress
concentration in a welded joint. Figure 2.8a shows the geometry and boundary conditions used in the numerical analysis and Figure 2.8b depicts how the stress concentration factor decreases as the weld toe radius increase. Molski et al. (2020) defined a parametric weld toe radius $X$ according to Equation 2.28, where $a_t$ is the weld throat thickness. Additionally, the parametric plate thickness $Y$ was considered as Equation 2.29, where $t$ is the plate thickness and $a_t$ is the weld throat thickness. It was concluded that as the weld toe radius increases the stress concentration factor rapidly decreases.

$$X = \frac{\rho}{(\rho + a_t)} \quad \text{(Equation 2.28)}$$

$$Y = \frac{a_t}{(a_t + t)} \quad \text{(Equation 2.29)}$$

(a) Geometry and Boundary Conditions of the cruciform joint.  
(b) Influence of the relative weld toe radius $X$ on the stress concentration factor $k_{tt}$

Figure 2.8. Investigation of the weld toe radius on the stress concentration factor (Molski et al. 2020)

According to Hobbacher (2016), if no weld toe radius $\rho$ was specified or determined by measuring, it is recommended to assume a sharp corner, i.e., a toe radius of $\rho = 0$ to $\rho = 2 \text{ mm} \ (0.008 \text{ inches})$. 
2.3.1.7 Numerical Stress Intensity Factors based on J-integral

Rice (1968) developed a path-independent line integral to determine the concentrated strain fields near notches and cracks. The integral is known as $J$-integral and it is performed around the contour of a crack tip, as shown in Equation 2.30 and Figure 2.9.

$$J = \int \left( Wdy - \overrightarrow{T} \cdot \frac{d\overrightarrow{u}}{dx} ds \right) d\Gamma$$  \hspace{1cm} (Equation 2.30)

where $W = \int_{0}^{em} \sigma_{ij} deij$ is the strain energy density (loading work), $\overrightarrow{T}$ is the traction vector of a contour element $ds$ defined by the normal vector $\overrightarrow{n}$ along the contour $\Gamma$ and $\overrightarrow{u}$ is the displacement vector at $ds$.

Figure 2.9. J-integral representation on a flat plate with a notch in 2D deformation field (Rice, 1968)

Rice (1968) showed that for nonlinear elastic behavior the $J$-integral could be used to describe a cracked body in terms of energy release during crack extension (Stephens et al. 2001). If the material behavior is under the linear-elastic regime, the $J$-integral value is identical to the strain energy release rate, $G$. According to Irwin (1948), the relationship
between $G$ and $K_I$ for the plane-strain and plane-stress condition are given in Equations 2.31 and 2.32, respectively.

\[
G = \frac{K_I^2}{E} (1 - \nu^2) \quad \text{(Equation 2.31)}
\]

\[
G = \frac{K_I^2}{E} \quad \text{(Equation 2.32)}
\]

Therefore, for LEFM applications, the calculation of $J$-integral is another way of obtaining stress intensity factors.

2.3.1.7.1 Calculation of $J$-integral based on FEM

Many authors have determined SIF based on the numerical solutions for the $J$-integral values. Lin and Smith (1999) modeled surface cracked plates to determine the SIF according to $J$-integral calculation. Hrnjica et al. (2016) found that the numerically determined $J$-integral of single-edge notched bend followed the values from the experimental results. Souto et al (2020) confirmed the accuracy of the numerical SIF solution by comparing the FE results with the classical analytical expression for a through-thickness crack in a finite plate. The latter author also stated that in order to guarantee that the contour $\Gamma$ is continuous in the FE model, the elements must be oriented in a “polar” way around the crack tip, as shown in Figure 2.10.
Figure 2.10. Definition of the $J$-integral contours in FE mesh (Souto et al. 2020).

The LEFM expressions for the stress field ahead the crack tip contains a singularity of $1/\sqrt{r}$ that needs to be considered in FE models in order to obtain accurate results of SIFs. The classical formulation of finite elements under linear-elastic behavior does not account for this type of singularity. In order to address this limitation, Henshell and Shaw (1975) and Barsoum (1976) developed the so-called quarter-point element that incorporates the $1/\sqrt{r}$ stress/strain singularity for linear-elastic field around the crack tip. The method consists of moving the mid-side node from the edge of the isoparametric element to a position closer to the crack tip, at a distance of one-quarter of its edge size (Castro and Meggiolaro, 2016), as shown in Figure 2.11. According to Barsoum (1986), the accuracy of results in a reasonable mesh is very high.
In this research, some of the stress intensity factors were calculated based on $J$-integral values obtained from FE models, accordingly to the procedure presented above. In Appendix A, the analytical SIF given in Equation 2.11c is compared to a numerical SIF obtained based on $J$-integral technique. The results show that the numerical procedure is a reliable way to obtain the SIF. One of the disadvantages of this method, though, is the need for remeshing the model for different crack sizes. Additionally, one should know the expected path for the crack to grow, so the correct crack size can be inputted in the FE model.

2.3.2 Crack Propagation Law

After defining the appropriated stress intensity factor for the fatigue-prone detail being investigated, a crack-growth law needs to be considered for the evaluation of the fatigue crack propagation life. Typically, crack growth under constant amplitude loading is characterized by three regions, as shown in Figure 2.12.
• Region I: region near the stress intensity factor threshold ($\Delta K_{th}$), below where there is no crack growth.

• Region II: stable macroscopic crack growth region, usually related to a crack-growth law.

• Region III: high crack growth rate approaching instability. Fracture occurs when the stress intensity factor range ($\Delta K_f$) reaches a material property known as fracture toughness ($K_C$).

![Figure 2.12. Regions for crack propagation.](image)

The most common crack growth law for constant amplitude loading is known as Paris’s law, where $\frac{da}{dN}$ is the crack growth rate and $C$ and $m$ are empirical material constants and (see Equation 2.33).

$$\frac{da}{dN} = C. (\Delta K_f)^m$$  \hspace{1cm} (Equation 2.33)

The number of cycles for the crack to propagate from the initial crack size $a_i$ to the critical crack size $a_c$ is calculated by solving Equation 2.34.

$$N = \int_{a_i}^{a_c} C(\Delta K_f)^m \, da$$  \hspace{1cm} (Equation 2.34)
When analytical solution for Equation 2.34 is not feasible numerical methods must be used. For instance, the numerical method based on incremental crack growth, $\Delta a$, provides accurate solutions for $\Delta N$ when $\Delta a$ is small enough.

Some models were developed to predict the crack growth rates under variable amplitude loading, as listed below.

- **Direct summation**: a crack growth increment is associated for each stress cycle (or block cycles). The size of the crack in the $n^{th}$ cycle is calculated by Equation 2.35, where $a_0$ is the initial crack size.

$$a_N = a_0 + \sum_{i=1}^{N} f(\Delta K_i) \quad \text{(Equation 2.35)}$$

- **Equivalent K methods**: equivalent stress intensity factor range is estimated in order to produce the same amount of fatigue crack growth as in the variable amplitude load history. For this purpose, the equivalent stress range can be determined by the RMC or RMS methods.

- **Wheeler model**: modified direct summation where the empirical retardation parameter ($C_i$) is included in the calculation (see Equation 2.36). Crack growth acceleration is not considered.

$$a_N = a_0 + \sum_{i=1}^{N} C_i \times f(\Delta K_i) \quad \text{(Equation 2.36)}$$

- **Willenborg model**: considers crack growth retardation by using an effective stress concept to reduce the applied stresses and hence the stress intensity factor. Any tensile load greater than the preceding tensile overload creates a new retardation. The model operates in terms of total crack length rather than increments (see Equation 2.37).
\[
\frac{da}{dN} = \frac{C.(K_{eff})^m}{(1 - R_{eff}).K_c - \Delta K_{eff}}
\]  
(Equation 2.37)

where \(R_{eff}\) is the effective load ratio and \(\Delta K_{eff}\) is the effective stress intensity factor range.

2.3.2.1 Crack Aspect Ratio of Semi-Elliptical Cracks

The aspect ratio of a semi-elliptical crack is defined by the ratio of the crack depth, \(a\), and half of the crack width, \(c\). It is reported in the literature that the shape of a crack during fatigue crack propagation does not remain constant (Wu, 1985). In other words, the crack aspect ratio \(a/c\) varies at each growth step. Moreover, it is well recognized that the variation of the aspect ratio \(a/c\) has a paramount influence on the prediction of the fatigue life of a component (Wu, 1985, Lu 1995, Leander et al. 2016).

According to Ravichandran (1996), the main factors that affect the variation in the crack aspect ratio are the type of loading in the crack plane, e.g., whether the remote loading is tension or bending, the presence of residual stresses and the initial crack sizes \(a_0\) and \(c_0\). Additionally, the empirical material constant \(m\) used in the crack propagation laws was found to affect the crack shape during fatigue crack growth.

Wu (1985) derived analytical expressions for the variation of the crack aspect ratio in metal plates by evaluating the crack propagation in both direction \(a\) and \(c\), and relating them as depicted in Equation 2.38. Thus, the main parameters affecting the shape of the crack are the stress intensity factor in each principal direction, as well as the crack growth constant \(C\). Wu (1985) considered that \(C_a = 1.1^m C_c\) based on Newman and Raju (1979) assumption that the crack growth rate \(C\) is different among the surface and the plate...
thickness due to the change of the stress state (plane stress in the surface and plane strain at the maximum depth point). The predicted aspect ratios proposed by the author were in good agreement with experimental data for various metal plates under tension and bending loads.

\[ \frac{da}{dc} = \frac{C_a}{C_c} \left( \frac{\Delta K_a}{\Delta K_p} \right)^m \]  

(Equation 2.38)

Likewise, Lu (1995) developed crack aspect development curves (CADC) for semi-elliptical surface cracks at weld toes, by relating the crack front propagation \( a \) and \( c \) accordingly to Equation 2.38. Lu (1995) included residual stresses in the calculation of stress intensity factors (SIF) and assumed that \( C_a = C_c \). After defining the appropriate SIFs, for a given increment of crack depth \( \Delta a \), the corresponding increment of half crack width \( c \), \( \Delta c \), and the number of stress cycles \( \Delta N \) were computed. This procedure continued until the fracture toughness of the material was reached or the crack depth penetrated the plate thickness.

According to Hobbacher (2016), if the stress intensity factors equations are available for both directions \( a \) and \( c \), the aspect ratio would be determined directly from the crack propagation calculations, as performed by Wu (1985) and Lu (1995). Maljaars et al. (2012) followed this procedure in the fatigue assessment of a cover plate detail on a bridge girder. Alternatively, Hobbacher (2016) recommends the use of experimental data whenever available. As the last alternative, the author recommends the use of a constant value of \( a/c = 0.1 \) as a conservative assumption.

The methods found in the literature to predict the aspect ratio during fatigue crack growth assume that shape of the crack will remain semi-elliptical. Although the crack may present an irregular shape, or the shape is not strictly semi-elliptical during propagation,
there is evidence in the literature that treating the crack as semi-ellipse is reasonable for engineering assessment (Lu, 1995). In order to verify such behavior, Castro et al. (1998) investigated the crack propagation from a forced initial rectangular notch. The fractographic examination showed that the crack quickly changed to a roughly semi-elliptical shape, as shown in Figure 2.13.

![Crack](image)

Figure 2.13. Crack that initiated from a rectangular notch and propagated to a semi-elliptical shape (Meggiolaro and Castro, 2001).

2.3.3 Initial crack sizes

The assessment of the remaining fatigue life based on LEFM models require knowledge on the initial crack size. When information provided by NDT inspection is not available, assumptions must be made. It is noted though, the total fatigue life computed based in LEFM model is very sensitive to different initial crack sizes. Various authors have considered different initial crack sizes for the fatigue assessment of steel bridges. Table 2.1 summarizes some of the most common initial crack sizes found in the literature. The distribution and the coefficient of variation for the probabilistic analyses are also provided. In the case of deterministic approach, the initial crack size is informed in the “Mean size” column.
Table 2.1 – Initial crack sizes in literature.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Distribution</th>
<th>Mean size</th>
<th>COV</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fisher J.W. et al. – NCHRP 147 (1974)</td>
<td>-----</td>
<td>0.001 to 0.02</td>
<td>-----</td>
<td>Steel Bridge (transverse stiffener details)</td>
</tr>
<tr>
<td>Fisher J.W. et al. – NCHRP 227 (1980)</td>
<td>-----</td>
<td>0.02</td>
<td>-----</td>
<td>Steel Bridge (longitudinal stiffener details)</td>
</tr>
<tr>
<td>Norris, S.N. et al. (1981)</td>
<td>-----</td>
<td>0.02</td>
<td>-----</td>
<td>Steel Bridge (longitudinal stiffener details)</td>
</tr>
<tr>
<td>Fisher, J.W. (1984)</td>
<td>-----</td>
<td>0.02</td>
<td>-----</td>
<td>Steel Bridge – various details</td>
</tr>
<tr>
<td>JCSS Probabilistic Model Code (2011)</td>
<td>Lognormal</td>
<td>0.006</td>
<td>0.66</td>
<td>Various</td>
</tr>
<tr>
<td>Zhao et al. (1994)</td>
<td>Lognormal</td>
<td>0.02</td>
<td>0.50</td>
<td>Steel Bridge</td>
</tr>
<tr>
<td>Cheung, M.S. et al. (2003)</td>
<td>Lognormal</td>
<td>0.03</td>
<td>0.50</td>
<td>Steel Bridge</td>
</tr>
<tr>
<td>Guo, T. et al. (2011)</td>
<td>Lognormal</td>
<td>0.02</td>
<td>0.50</td>
<td>Steel Bridge</td>
</tr>
<tr>
<td>Maljaars, J. et al. (2012)</td>
<td>Lognormal</td>
<td>0.006</td>
<td>0.66</td>
<td>Cover plate on steel bridge girder</td>
</tr>
<tr>
<td>Kim, J-H. et al. (2017)</td>
<td>-----</td>
<td>0.20</td>
<td>-----</td>
<td>Steel Bridge (edge crack on plain steel)</td>
</tr>
<tr>
<td>Hobbacher (2016)</td>
<td>-----</td>
<td>0.004</td>
<td>-----</td>
<td>Welded Joints</td>
</tr>
<tr>
<td>Leander, J. et al. (2016)</td>
<td>Lognormal</td>
<td>0.006</td>
<td>0.66</td>
<td>Transverse stiffener and gusset plate-girder connection</td>
</tr>
<tr>
<td>Hashemi, B. et al. (2017)</td>
<td>Lognormal</td>
<td>0.006</td>
<td>0.66</td>
<td>Welded Joint (transverse butt weld joint)</td>
</tr>
</tbody>
</table>

Usually the probabilistic-based models assume a lognormal distribution for the initial crack size variable. According to Zhao (1994), the lognormal distribution tends to be more conservative than a Weibull distribution since it predicts higher probability of larger cracks. Moreover, the coefficient of variation (COV) was found to be between 0.50 and 0.66. The mean values varied from 0.006 inches and 0.03 inches. For the deterministic analysis, the crack size varied from 0.001 inches to 0.2 inches. It is stressed that the initial crack size highly affects the estimation of the remaining fatigue life of a component or detail prone to fatigue.
Based on the wide range of initial cracks for bridge applications found in previous research, in this dissertation initial cracks from 0.006 inches to 0.2 inches are considered in the preliminary LEFM assessment.

2.3.4 Applicability of LEFM

Before applying LEFM approach, three conditions must be satisfied in order to not violate the assumptions regarding LEFM theory. Firstly, the plastic zone radius, \( r_p \), at the crack tip must be small in comparison to the local geometry and crack size, so the linear elastic behavior of the material can be considered. This condition can be verified based on Equation 2.39.

\[
\begin{align*}
\frac{r_p}{a} &= \frac{1}{2\pi} \left( \frac{K_{II}}{F_y} \right)^2 \leq \frac{a}{4} & \text{under Plane Stress} \\
\frac{r_p}{a} &= \frac{1}{6\pi} \left( \frac{K_{I}}{F_y} \right)^2 \leq \frac{a}{4} & \text{under Plane Strain}
\end{align*}
\]  

(Equation 2.39)

Secondly, the applied nominal stress should be less than 0.8\( F_y \) in order to limit the plasticity created at the crack tip. Thirdly, the stress intensity factor range, \( \Delta K_I \), should be larger than the threshold stress intensity factor range, \( \Delta K_{th} \), and smaller than the fracture toughness of the material, \( K_c \).

2.3.4.1 Threshold Stress Intensity Factor Range

Below a certain value of stress intensity factor range, the crack growth does not occur. The term crack growth herein implies the propagation of macrocracks where the so-called Paris region governs the crack growth behavior. This limit for the crack to grow is
called threshold stress intensity factor range, $\Delta K_{th}$ and it is obtained experimentally as per ASTM E647 “Standard Test Method for Measurement of Fatigue Crack Growth Rates”. Various parameters affect $\Delta K_{th}$ values, such as, mean stress, or stress ratio $R$ ($R = \sigma_{min}/\sigma_{max}$), maximum stress intensity factor $K_{max}$, grain size of the material, stress-history, residual stress and environmental effects (Barsom et al. 1999, Anderson 2005, BS7910 2005, Manson et al. 2006). Figure 2.14 shows fatigue crack growth test results for various steels published by Barsom et al. (1999) and Anderson (2005). These tests were performed under constant amplitude loading (CAL). It is observed that that $\Delta K_{th}$ decreases as the stress ratio $R$ increases. The $\Delta K_{th}$ for a mild steel (A36 steel) at stress ratio $R = 0.8$ under CAL results in to $3.5 \text{ ksi} \sqrt{\text{in}}$ ($4 \text{ MPa} \sqrt{\text{m}}$).

![Graph of Fatigue Crack Growth Test Results](image1)

![Graph of Threshold Stress Intensity Factor Range](image2)

(Barsom and Rolfe, 1999) (Anderson, 2005)

Figure 2.14. Effect of $R$ ratio of the threshold stress intensity factor range.

It has been shown that $\Delta K_{th}$ under variable amplitude loading (VAL) can be extrapolated below the limit established for the constant amplitude loading (Fisher et al.
Fatigue crack growth tests under random block loading were performed in the NCHRP Project 12-15(4) for A36 steel. At a stress ratio \( R = 0.8 \), the reduction in the \( \Delta K_{th} \) was found to be from 3.5 ksi \( \sqrt{\text{in}} \) to 2.0 ksi \( \sqrt{\text{in}} \) (see Figure 2.15a). Figure 2.15b compares both the constant amplitude tests reported in Barsom et al. (1999) and the variable amplitude tests published by Fisher et al. (1983) for a mild steel at different stress ratios. It is concluded that under the variable amplitude loading the \( \Delta K_{th} \) is significantly reduced. These results provide good reference for \( \Delta K_{th} \) estimation of welded joints in steel bridges, since welded structures tend to have high stress ratio \( R \) due to the presence of high residual stresses (Norris 1979, Fisher 1984).

(a) Fatigue crack growth test for A36 steel under variable amplitude loading (VAL).

(b) Comparison of tests under VAL and CAL.

Figure 2.15. Test for fatigue crack growth threshold under VAL and CAL (Fisher et al. 1983 – NCHRP Report 267).
Recommendations on the threshold stress intensity factor range found in the literature are provided below.

- Schijve (2009) questioned whether there is a unique threshold level for crack growth since $\Delta K_{th}$ is dependent on many variables and the definition for the transition between microcracks to macrocrack is not straightforward. Small cracks (microcracks) are defined by the ASTM E647 as: “1) their length being small compared to relevant microstructure dimension; 2) their length is small compared to the scale of local plasticity; 3) they are physical small (< 1mm)”. In conclusion, Schijve (2009) recommends caution when defining the crack growth threshold. For the crack growth resistance of a material the author recommends the extrapolation of the $da/dN$ vs. $\Delta K$ curve further down from the Paris regime, to obtain lower values of $\Delta K_{th}$ as represented in Figure 2.16. It is noted that according to ASTM E647, the definition of $\Delta K_{th}$ for most materials is the corresponding $\Delta K$ value at an operational and arbitrary crack growth rate ($da/dN$) of $10^{-10}$ m/cycle ($3.9 \times 10^{-8}$ in/cycle).
Figure 2.16. Extrapolation of crack growth rate curve further down from the Paris regime (Schijve, 2009).

- According to Barsom et al. (1999), a conservative estimation of \( \Delta K_{th} \) for \( R \geq 0.1 \) is given by Equation 6.1. A crack growth threshold of 2.048 \( ksi\sqrt{in} \) is obtained when a stress ratio \( R = 0.8 \) is inputted in Equation 2.40. This result is in good agreement to the fatigue crack growth tests presented in NCHRP Project 12-15(4) (see Figure 2.15a).

\[
\Delta K_{th} = 6.4(1 - 0.85R) ksi\sqrt{in} \quad \text{(Equation 2.40)}
\]

- The recommended values for \( \Delta K_{th} \) presented in the BS7910 (2005) to assess crack growth in welded joints, are shown herein in Table 2.2. For steels in the air environmental, \( \Delta K_{th} = 1.82 ksi\sqrt{in} \) (2 MPa\( \sqrt{m} \)).
Table 2.2 – Recommended $\Delta K_{th}$ for assessing welded joints (BS7910, 2005).

<table>
<thead>
<tr>
<th>Material</th>
<th>Environment</th>
<th>$\Delta K_{th}$ N/mm$^{3/2}$ (MPa$\sqrt{m}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steels, including austenitic</td>
<td>Air or other non-aggressive environments up to 100°C</td>
<td>63 (2)</td>
</tr>
<tr>
<td>Steels, excluding austenitic</td>
<td>Marine with cathodic protection, up to 20°C</td>
<td>63 (2)</td>
</tr>
<tr>
<td>Steels, including austenitic</td>
<td>Marine, unprotected</td>
<td>0 (0)</td>
</tr>
<tr>
<td>Aluminum alloys</td>
<td>Air or other non-aggressive environments up to 20°C</td>
<td>21 (0.7)</td>
</tr>
</tbody>
</table>

- Schroeder (2018) found that BS7910 (2013) assumes a $\Delta K_{th}$ between 1.5 ksi$\sqrt{in}$ and 2.0 ksi$\sqrt{in}$ to determine the critical sizes of slag inclusions in a welded joint.

2.4 UniGrow model

Although it is accepted that fatigue crack growth depends on local stresses and strains at the crack tip, most of the crack growth models developed so far does not account for this effect. The Unigrow model developed by Noroozi et al. (2005) addresses this limitation. The actual elastic-plastic stress-strain field at the crack tip is correlated to a unified two-parameter crack growth model based on the total maximum stress intensity factor, $K_{max,tot}$, and the total stress intensity factor range, $\Delta K_{tot}$. Thus, Unigrow integrates the strain-life method for notch strain analysis with LEFM into a singular model that is able to predict fatigue crack nucleation and propagation.

The next sections present a review of the strain-life method and the detailed assumptions regarding the Unigrow model.
2.4.1 Strain-life and notch strain analysis

Crack initiation life is usually associated to the local plastic straining at notch tips. The notch strain analysis is performed based on the elastic-plastic monotonic and cyclic stress-strain curves of the material along with an analytical model, such as Neuber’s rule, that correlates the nominal stress-strain to the actual stress-strain at the notch. It is recalled that it is not practical to directly measure strains histories at notch tips, so analytical models should be used (Castro and Meggiolaro, 2016a). Then, fatigue initiation life is estimated according to some damage parameter such as the Smith-Watson-Topper parameter.

The monotonic and cyclic elastic-plastic stress-strain curve of a material can be represented by the Ramberg-Osgood Relationship, as presented in Equation 2.41 and 2.42, respectively.

\[ \varepsilon = \frac{\sigma_a}{E} + \left(\frac{\sigma_a}{K}\right)^{1/n_h} \]  
\[ \Delta \varepsilon = \frac{\Delta \sigma_a}{E} + 2 \left(\frac{\Delta \sigma_a}{2K'}\right)^{1/n'} \]

(Equation 2.41)  
(Equation 2.42)

where \( \varepsilon \) is the actual strain, \( \sigma_a \) is the actual stress, \( K \) is the strength coefficient, \( n_h \) is the strain hardening exponent, \( \Delta \varepsilon \) is the actual strain range, \( \Delta \sigma_a \) is the actual stress range, \( K' \) is the cyclic strength coefficient and \( n' \) is cyclic strain hardening exponent. The parameters for the monotonic curve are defined according to the simple tension test as indicated in ASTM E8/E8M whereas, the cyclic stress-strain curve parameters are based on strain-controlled tests according to ASTM E606/E606M. Both tests utilize smooth axial specimens.

During the cyclic loading the material can harden or soften depending on its dislocation state (Manson and Halford, 2006). The cyclic hardening of a material is
characterized by the increase of resistance to deformation, whereas cyclic softening is the opposite phenomenon (see Figure 2.17). Thus, the comparison between the monotonic and cyclic stress-strain curves in terms of strain-hardening exponents and strength coefficient gives useful information regarding the stability of the material (ASTM E606/E606M).

Figure 2.17. Strain cycling hardening and softening examples (Stephens et al., 2001).

From the strain-controlled tests the total strain amplitude, $\varepsilon_d$, is correlated to the number of cycles to failure $N_f$ (or number of reversals $2N_f$ to failure) forming the strain-life curve, known as Coffin-Manson equation, as presented in Equation 2.43, where: $\Delta \varepsilon_e / 2$ is the elastic strain amplitude, $\Delta \varepsilon_p / 2$ is the plastic strain amplitude, $\sigma_f$ is the fatigue strength coefficient, $b$ is the fatigue strength exponent, $\varepsilon_f'$ is the fatigue ductility coefficient and $c_d$ is the fatigue ductility exponent. The stabilized cyclic elastic-plastic stress-strain curve, referred as stabilized hysteresis loop, is used to characterize the strain-life curve of a material.
\[ \varepsilon_a = \frac{\Delta \varepsilon_e}{2} + \frac{\Delta \varepsilon_p}{2} = \frac{\sigma_f'(2N_f)^b + \epsilon_f'(2N_f)^c}{E} \]  \hspace{1cm} \text{(Equation 2.43)}

Once the fatigue properties are determined as discussed above, based on the similitude concept it is assumed that the fatigue life at the notch tip of a component is similar to the life of the smooth axial specimen used in the strain-life method (Pereira Batista, 2016). The main idea is to extract the actual stress-strain at the notch tip according to an analytical model and estimate fatigue life based on the corresponding strain-life curve of the smooth axial specimen made of same material of the component, as shown in Figure 2.18.

![Image](image.png)

**Figure 2.18.** Similitude concept (Pereira Batista, 2016).

The Neuber’s rule that correlates the nominal elastic stress-strain to the actual (or local) stress-strain according for the monotonic and cyclic loading are given in Equations 2.44 and 2.45, respectively.

\[ \varepsilon \sigma_a = \frac{(K_t S)^2}{E} \]  \hspace{1cm} \text{(Equation 2.44)}

\[ \Delta \varepsilon \Delta \sigma_a = \frac{(K_t \Delta S)^2}{E} \]  \hspace{1cm} \text{(Equation 2.45)}
where $K_t$ is the stress concentration factor, $S$ is the nominal stress and $\Delta S$ is the nominal stress range.

Solving Neuber’s rule Equations with the monotonic and cyclic stress-strain relationship, one can obtain the strain amplitude, $\varepsilon_a$, and the maximum, $\sigma_{max}$, and predict the fatigue life for crack initiation based on the Smith-Watson-Topper (SWT) parameter, as shown in Equation 2.46. This parameter assumes that the term $\sigma_{max}\varepsilon_a$ remains constant for a given life.

$$\sigma_{max}\varepsilon_a = \frac{(\sigma_f)^2 (2N_f)^{2b}}{E} + \sigma'_f \varepsilon'_f (2N_f)^{b+c+d} \quad \text{(Equation 2.46)}$$

Four steps are required to calculate fatigue life using this approach: 1) from the constant amplitude load history, extract the maximum nominal stress, $S_{max}$, and calculate the maximum actual stress, $\sigma_{max}$, and the maximum actual strain, $\varepsilon_{max}$, for the $1^{st}$ reversal based on the solution of the monotonic Ramberg-Osgood and Neuber’s rule equations; 2) from the constant amplitude load history, extract the nominal stress range, $\Delta S$, and calculate the actual stress range, $\Delta \sigma$, and the actual strain range, $\Delta \varepsilon$, based on the cyclic Ramberg-Osgood and Neuber’s rule equations; 3) compute the residual stress and strain based on the results of item 1) and 2), e.g., $\sigma_{res} = \sigma_{max} - \Delta \sigma$ and $\varepsilon_{res} = \varepsilon_{max} - \Delta \varepsilon$; 4) Estimate the fatigue life in terms of number of cycles to failure, $N_f$, according to the Smith-Watson-Topper equation. Table 2.3 summarizes the equations for the computation of items 1) and 2).

This method is suitable to predict fatigue life to formation of small cracks on the order of 0.04 inches (1 mm) (Stephens et al. 2001).
Table 2.3 – Ramberg Osgood and Neuber’s rule equations.

<table>
<thead>
<tr>
<th>Loading</th>
<th>Stress-strain relationship</th>
<th>Neuber’s rule</th>
<th>Local stresses and strains</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monotonic</td>
<td>$\varepsilon = \frac{\sigma_a}{E} + \left(\frac{\sigma_a}{K}\right)^{1/n_h}$</td>
<td>$\varepsilon\sigma_a = \frac{(K\delta S)^2}{E}$</td>
<td>$\begin{cases} \frac{(K\delta S_{\text{max}})^2}{E} = \frac{\sigma_{\text{max}}^2}{E} + \frac{\sigma_{\text{max}}^2}{K}^{1/n_h} \ \varepsilon_{\text{max}} = \frac{\sigma_{\text{max}}}{E} + \left(\frac{\sigma_{\text{max}}}{K}\right)^{1/n_h} \end{cases}$</td>
</tr>
<tr>
<td>Cyclic</td>
<td>$\Delta \varepsilon = \frac{\Delta \sigma_a}{E} + 2 \left(\frac{\Delta \sigma_a}{2K}\right)^{1/n'}$</td>
<td>$\Delta \varepsilon \Delta \sigma_a = \frac{(K\delta S)^2}{E}$</td>
<td>$\begin{cases} \frac{(K\delta S)^2}{E} = \frac{\Delta \sigma_a^2}{E} + 2\Delta \sigma_a \left(\frac{\Delta \sigma_a}{2K}\right)^{1/n'} \ \Delta \varepsilon = \frac{\Delta \sigma_a}{E} + 2 \left(\frac{\Delta \sigma_a}{2K}\right)^{1/n'} \end{cases}$</td>
</tr>
</tbody>
</table>

2.4.2 UniGrow model

The UniGrow model is a unified crack growth model for fatigue crack initiation and propagation. It considers the elastic-plastic stress-strain at the crack tip and the residual stress due to cyclic plastic deformation to formulate the instantaneous fatigue crack growth rate in terms of the total maximum stress intensity factor, $K_{\text{max, tot}}$, and the total stress intensity factor range, $\Delta K_{\text{tot}}$. The model uses similar concepts from the strain-life method to compute the elastic-plastic stress-strain relationship at the crack tip. Yet, the definition of the elastic stresses (nominal stresses) are based on fracture mechanics principles. For instance, the stress intensity factor solutions are used to define the elastic stresses around the crack tip.

The main assumptions regarding the UniGrow model are presented below:

- The material is assumed to be composed of identical elementary material blocks of dimension $\rho^*$ where the classical mechanics of continuum can still be applied (see Figure 2.19).
The fatigue crack is considered to be a deep sharp notch with a finite notch/crack tip radius $\rho^*$ (blunt crack). The advantage of this assumption is that the notch theory can be applied, and it avoids the unrealistic assumptions of having high stresses in the vicinity of the crack tips. In this way the concept of crack closure behind the crack tip does not need to be introduced in the model (Noroozi et al. 2007).

The material properties can be described by the Ramberg-Osgood relationship and the corresponding strain-life curve. The estimation of the actual stresses-strains ahead of the blunt crack can be performed by using Neuber’s rule.

The Fatigue crack growth is considered as successive crack increments due to crack re-initiation over the distance $\rho^*$ (Noroozi et al. 2007).

The number of cycles $N$ necessary to fail the material over the distance $\rho^*$ can be obtained from the Coffin-Manson curve and the Smith-Watson-Topper (SWT) fatigue damage parameter (Noroozi et al. 2007).

The fatigue crack growth rate is determined based on Equation 2.47.
\[ \frac{da}{dN} = \rho^* N \]  

(Equation 2.47)

The correlation between the actual stresses-strains to the elastic stresses (nominal stresses), under the monotonic and cyclic loading are presented in Equation 2.48 and 2.49, respectively.

\[
\left\{ \begin{aligned}
\frac{\left(\bar{\sigma}_{max}^e\right)^2}{E} &= \frac{\left(\bar{\sigma}_{max}^a\right)^2}{E} + \bar{\sigma}_{max}^a \left(\frac{\bar{\sigma}_{max}^a}{K'}\right)^{1/n'} \\
\bar{\varepsilon}_{max}^a &= \bar{\sigma}_{max}^a + \left(\frac{\bar{\sigma}_{max}^a}{K'}\right)^{1/n'} \\
\frac{(\Delta\bar{\sigma}^e)^2}{E} &= \frac{(\Delta\bar{\sigma}^a)^2}{E} + 2(\Delta\bar{\sigma}^a) \left(\frac{\Delta\bar{\sigma}^a}{2K'}\right)^{1/n'} \\
\Delta\bar{\varepsilon}^a &= \frac{\Delta\bar{\sigma}^a}{E} + 2 \left(\frac{\Delta\bar{\sigma}^a}{2K'}\right)^{1/n'} 
\end{aligned} \right. \]  

(Equation 2.48)

where: \( \bar{\sigma}_{max}^e \) is maximum elastic stress, \( \Delta\bar{\sigma}^e \) is the elastic stress range, \( \bar{\varepsilon}_{max}^a \) is the maximum actual strain and \( \Delta\bar{\varepsilon}^a \) is the actual strain range. All of these variables are calculated in first elementary block \( \rho^* \) ahead of the crack tip.

For a stress ratio greater than zero, the average linear-elastic stresses over the first elementary material block induced by the applied maximum and minimum nominal stresses (\( S_{max,appl} \) and \( S_{min,appl} \)), can be calculated as Equations 2.50 and 2.51, respectively, where \( \psi_{y,1} \) is the average constant for the 1st elementary material block defined as 1.633 in Noroozi et al. (2005), \( K_{max,appl} \) is the maximum applied stress intensity factor and \( K_{min,appl} \) is the minimum applied stress intensity factor. The elastic stress range over the first elementary material block is given in Equation 2.52 and the expressions for \( K_{max,appl}, K_{min,appl} \) and \( \Delta K_{appl} \) are provided in Equation 2.53 to 2.55, where \( F(a) \) is the generalized correction factor.
\[ \tilde{\sigma}_{\text{max}} = \frac{\psi_y K_{\text{max,appl}}}{\sqrt{2\pi \rho^*}} \]  
(Equation 2.50)

\[ \tilde{\sigma}_{\text{min}} = \frac{\psi_y K_{\text{min,appl}}}{\sqrt{2\pi \rho^*}} \]  
(Equation 2.51)

\[ \Delta \tilde{\sigma} = \frac{\psi_y (\Delta K_{\text{appl}})}{\sqrt{2\pi \rho^*}} \]  
(Equation 2.52)

\[ K_{\text{max,appl}} = S_{\text{max,appl}} F(a) \sqrt{\pi a} \]  
(Equation 2.53)

\[ K_{\text{min,appl}} = S_{\text{min,appl}} F(a) \sqrt{\pi a} \]  
(Equation 2.54)

\[ \Delta K_{\text{appl}} = K_{\text{max,appl}} - K_{\text{min,appl}} \]  
(Equation 2.55)

Once \( \tilde{\sigma}_{\text{max}} \) and \( \Delta \tilde{\sigma} \) are determined, \( \tilde{\sigma}_{\text{max}} \) and \( \Delta \tilde{\sigma} \) can be calculated based on the Ramberg-Osgood and Neuber’s rule equations (Equations 2.48 and 2.49).

The residual stress field induced by the loading and unloading stress reversals is found by subtracting the local stress range \( \Delta \tilde{\sigma} \) from the maximum local stress \( \tilde{\sigma}_{\text{max}} \). This calculation is needed at several locations ahead the crack tip, resulting in a distribution of the residual stress versus the distance from the crack tip. Then, the residual stress effect should be quantified in terms of stress intensity factor, \( K_r \), based on the weight function method or Green’s function (Stephens et al. 2001). Lastly, \( K_{\text{max,appl}} \) and \( K_{\text{min,appl}} \) should be modified in order to account for the effect of residual stress on the fatigue crack growth (see Equations 2.56 to 2.58), as shown in Figure 2.20 for the case of stress ratio greater than zero and tension residual stresses.

\[ K_{\text{max,tot}} = K_{\text{max,appl}} + K_r \]  
(Equation 2.56)

\[ K_{\text{min,tot}} = K_{\text{min,appl}} + K_r \]  
(Equation 2.57)

\[ \Delta K_{\text{tot}} = \Delta K_{\text{appl}} \]  
(Equation 2.58)
The minimum applied stress intensity factor, $K_{\text{min,appl}}$, is not affected by compression residual stresses. Thus, Equations 2.57 and 2.58 are updated to Equation 2.59 and 2.60.

$$K_{\text{min,tot}} = K_{\text{min,appl}} \quad \text{(Equation 2.59)}$$

$$\Delta K_{\text{tot}} = \Delta K_{\text{appl}} + K_r \quad \text{(Equation 2.60)}$$

After the stress intensity factors are updated with $K_r$ values, $K_{\text{max,tot}}$ and $\Delta K_{\text{tot}}$ are assigned to recalculate the elastic stresses over the 1st material block $\rho^*$ ahead of the crack tip. Therefore, Equation 2.50 and 2.52 are updated to Equation 2.61 and 2.62, respectively. Likewise, the actual maximum stress $\bar{\sigma}_{\text{max}}^a$ and the actual stress range $\Delta\bar{\varepsilon}^a$ are also recalculated (Equations 2.48 and 2.49) considering the updated $\bar{\sigma}_{\text{max}}^e$ and $\Delta\bar{\sigma}^e$.

$$\bar{\sigma}_{\text{max}}^e = \frac{\psi_{y,1} K_{\text{max,tot}}}{\sqrt{2\pi \rho^*}} \quad \text{(Equation 2.61)}$$

$$\Delta\bar{\sigma}^e = \frac{\psi_{y,1} (\Delta K_{\text{tot}})}{\sqrt{2\pi \rho^*}} \quad \text{(Equation 2.62)}$$

With the known values of $\bar{\sigma}_{\text{max}}^a$ and $\Delta\bar{\varepsilon}^a$ the number of cycles to fail the first elementary block $\rho^*$ is calculated based on the SWT and Coffin-Mason Equation, repeated herein in Equation 2.63 with updated notation for the Unigrow model.
\[
\frac{\Delta \bar{\epsilon}}{2} = \frac{(\sigma_f')^2 (2N_f)^{2b}}{E} + \sigma_f' \varepsilon_f' (2N_f)^{b+c_d} \quad \text{(Equation 2.63)}
\]

As a result, the crack growth is considered as successive crack increments due to crack re-initiations over the distance \( \rho^* \), as expressed before in Equation 2.47.

Based on the presented methodology, Noroozi et al. (2005) derived the fatigue crack growth equation in terms of a unified two-parameter driving force \( K_{\text{max,tot}} \) and \( \Delta K_{\text{tot}} \), as depicted in Equation 2.64.

\[
\frac{da}{dN} = C \left[ \left( K_{\text{max,tot}} \right)^p \left( \Delta K_{\text{tot}} \right)^{0.5} \right]^\gamma \quad \text{(Equation 2.64)}
\]

The definition of the parameters \( C, p \) and \( \gamma \) depend on whether the behavior of the material at the crack tip is predominantly plastic, elastic or elastic-plastic. Equations 2.65 to 2.67 present the computation of these parameters for each case.

**Predominantly Plastic behavior at the crack tip**

\[
C = 2 \rho^* \left[ \frac{\left( \psi_{y,1} \right)^2}{\left[ \frac{(n' + 1)^2}{2} \right]} \right]^{\left( \frac{1}{b+c_d} \right)} ; p = \frac{n'}{n' + 1} ; \gamma = \frac{2}{b + c_d} \quad \text{(Equation 2.65)}
\]

**Predominantly Elastic behavior at the crack tip**

\[
C = 2 \rho^* \left[ \frac{\left( \psi_{y,1} \right)^2}{4 \pi \rho^* \left( \sigma_f' \right)^2} \right]^{-1/2b} ; p = 0.5 ; \gamma = \frac{1}{b} \quad \text{(Equation 2.66)}
\]

**Elastic-Plastic behavior at the crack tip**

\[
C = 2 \rho^* \left\{ \frac{1}{2 \left( \sigma_f' \right)^2} \times \left[ \left( \frac{\psi_{y,1}}{\sqrt{2 \pi \rho^*}} \right)^{3n' + 1} \times \frac{K'}{E n'} \right] \right\}^{-1/2b} ; p = \frac{n'}{n' + 1} ; \gamma = \frac{1}{b} \quad \text{(Eq. 2.67)}
\]
The latter case was developed based on the assumption that the strain at the crack tip induced by the maximum load were predominantly plastic, while the strain ranges were predominantly elastic (Noroozi et al., 2007).

Thus, the driving force, \( \Delta \kappa \), that induce fatigue crack growth is defined by two the two parameters, as depicted in Equation 2.68.

\[
\Delta \kappa = \left( K_{\text{max, tot}} \right)^p (\Delta K_{\text{tot}})^{1-p}
\]  

(Equation 2.68)

It is observed that Croft et al. (2007), found strong quantitative support for a crack growth rate driving force as presented in Equation 2.68, based on the correlation of X-ray strain profiling results with the crack growth rate \( da/dN \), indicating the high accuracy of the model.

As a result, the Unigrow model is capable to correlate the maximum stress intensity factor and the stress intensity factor range to the local actual elasto-plastic stresses and strains at the crack tip.

2.5 Prevailing Non-Destructive-Testing to Detect Cracks in Welded Joints

The remaining fatigue life assessment is very sensitive to different initial crack size assumptions. In order to address this issue, the initial crack size must be determined by the available techniques of non-destructive inspection and must be done as accurately as possible (Hobbacher, 2011). The prevailing NDT methods to detect fatigue cracks in steel bridges are revised in this section. Moreover, the advantages of the phased array ultrasonic
testing, which was recently included in the AASHTO/AWS D1.5 Bridge Welding Code as an additional NDT method to inspect steel bridges, is discussed.

2.5.1 Radiographic Testing (RT)

Radiographic Testing (RT) method is used to evaluate complete joint penetration (CJP) since 1930s. The welds are inspected based on a source of radiation that penetrates the specimen and a two-dimensional projection of the weld is produced in the photographic film. The radiograph image has a varying density according to the radiation reaching each area of the photographic film (Wilkinson, 2014). The flaw is detected when a lack of density (lack of material) is presented in the film as darker areas on the image (see Figure 2.21).

![Radiographic Testing (RT) method](image)

(a) RT Technique  
(b) Typical RT Film for a Weld with flaws

Figure 2.21 Radiographic Testing (RT) example (Wilkinson, 2014).

The advantages of using RT in comparison to UT and PAUT methods are summarized as:

- A highly skilled operator is not required.
- With film the radiograph is in real scale and the discontinuity can be measured with a ruler.
- The results are recorded in a film or on a digital image and can be re-interpreted by a second party.
The disadvantages of RT method are:

- RT requires special safety arrangements due to the strong radiation source.
- The cost of RT can be significant. According to Wilkinson (2014), Florida Department of Transportation (FDOT) would be able to save 2-4 million a year in RT expenses if other NDT technique could be used instead.
- Delaminations and planar cracks are difficult to detect. Moreover, very thin discontinuities such as lack of fusion may not create enough of void to show up on a radiograph (Medlock et al. 2020).

2.5.2 Ultrasonic Testing (UT)

The Ultrasonic Testing was introduced in the AWS D1.5 Bridge Welding Code in 1969. It uses a single element transducer to create an ultrasound in a particular direction that is transmitted into the specimen. The same transducer reads the reflected sound (pulse and echo technique) which is evaluated by a plot known as A-scan that shows the sound amplitude in decibels (dB) along the time. If a discontinuity is found, an anomalous signal is returned to the transducer. Figure 2.22 exemplifies the UT method.

![UT technique](a) UT technique (Medlock et al. 2020).
![Transducer placed in the testing area](b) Transducer placed in the testing area (Hopwood et al. (2016)).
![A-scan plot](c) A-scan plot (Hopwood et al. (2016)).

Figure 2.22. Ultrasonic Testing example.
The advantages of UT in relation to RT are: i) the system is nonhazardous and easier to transport and implement in the testing area since it requires only one surface to perform the test; ii) UT is more capable than RT of determining the depth location of the flaws. As drawbacks, UT requires considerable operator skill to manipulate the transducer and interpret the received signals (Wilkinson, 2014). Moreover, the results are not permanently recorded, and the interpretation of results must be in real time. This means that the bridge owners must trust the operator conclusions when performing UT, since the results cannot be revisited.

2.5.3 Phased Array Ultrasonic Testing (PAUT)

This method was added to AWS D1.5 code in 2015 revision and it has the same principles of UT. PAUT is known as enhanced UT due its many advantages in comparison to UT method. Firstly, rather than a single transducer the method possesses a multiple element transducer (16 to 256 elements) that produces ultrasonic waves with different or same direction angles that are swept through a wider volume of material. The generated waves can be linear or sectorial as shown in Figure 2.23. It is noted that the multiple pulses can be controlled and transmitted in phases at separated time-shifts (phased array). Thus, one position of the transducer result in a greater volume of material tested, meaning that PAUT application is much faster and efficient than UT. Secondly, PAUT produces inspection results in multiple formats such as: A-scan, B-scan (side-view of the tested cross-section), C-scan (top view of the scanned region), D-scan (end-view). Figure 2.24 exemplifies and explains the various results format. Thirdly, the inspection results are
recorded, so it can be re-interpreted by a second party or revisited other time rather than the real-time inspection.

(a) Linear Scan
(b) Sectorial Scan
Figure 2.23. PAUT multiple ultrasounds generated (Medlock et al., 2020).

(a) PAUT various results format
(b) PAUT images explained
(Medlock et al., 2020)  
(AASHTO/AWS D1.5, 2015)
Figure 2.24. PAUT results format.

Florida Department of Transportation (FDOT) recognized the superiority of PAUT method, and it is allowing its application in lieu of the traditional RT even in Fracture Critical Members (Medlock et al., 2020). Hence, the use of PAUT for detecting realistic initial crack sizes would highly benefit the analytical model based on LEFM.

The major disadvantage of the PAUT is that the method requires a high-skilled operator. Moreover, careful PAUT calibration must be performed to assure reliable results.
2.5.4 Magnetic Particle (MT)

The principle of Magnetic Particle test is based on the interruption of a magnetic field by the presence of a discontinuity. Iron powder (magnetic particles) are dusted in the inspection area and a magnetic field is introduced by placing a permanent magnet or electromagnet on the material, or by passing a current through the material (Medlock et al. 2020). The weld to be inspected must be cleaned before the test. The magnetic particles may be applied with a bulb applicator (dry method) or by aerosol cans of powder suspension (wet method) (Hopwood II et al. 2016). If a discontinuity disturbs the field, a concentration of magnetic lines of force will occur, and thus a concentration of iron powder (MBE 3rd Edition). Usually, for field application, electromagnetic yokes are preferable to induce the electromagnetic field.

As advantages, this test is easy to be performed and the results are quickly available. Moreover, the equipment is portable and does not require skilled operators. On the other hand, MT is efficient only to detect surface and sub-surface cracks.

2.5.5 Acceptance Criteria for Discontinuities found in NDT

When the discontinuity found in NDT does not comply with the minimum acceptance criteria established by a certain recognized construction code, the discontinuity is said to be “rejected” and it is classified as a defect. The bridge industry in United States uses the AASHTO/AWS D1.5 Bridge Welding Code for such verifications. According to AWS D1.5, defects are unacceptable and require repair or replacement of the defective component. According to the Bridge Welding Code it is important to distinguish “Discontinuity”, “Flaw”, and “Defect”. Discontinuity is defined as an interruption of the
typical material structure and not necessarily is malign to the structure; Flaw is defined as an undesired discontinuity; Defect is defined as a discontinuity that fails to comply with the acceptance criteria.

Historically, the AWS D1.5 acceptance criteria is based on workmanship rather than fitness-for-service (Schroeder, 2018). That means that rejectable flaws are based on traditional values rather than an objective criterium founded on analytical or experimental investigation. Medlock et al. 2020 recommends that the AWS D1.5 code criteria should not be considered as lower bounds for acceptable service due to the lack of engineering critical assessment (ECA) for its definition, meaning that there is room for technical assessment before rejecting a discontinuity. For PAUT applications, AWS D1.5 considers the maximum sound amplitude and the discontinuity length as parameters for the acceptance criteria (Connor et al., 2019).

It is noted that the Bridge Welding Code is intended to assure the quality of welds in steel bridges before they are put in service. A discontinuity rejection would be easily repaired in the fabrication shop before the plate steel girder is assembled on the field. It is evident that the scenario of a rejection becomes much more challenging when the bridge is in service. In this case, appropriate fitness-for-service criterium based on fracture mechanics analysis would be suitable for the decision making of repairing or replacing the rejected component.

2.5.5.1 PAUT discontinuities rejection rate

One of the concerns of applying PAUT in steel bridge welds is that the flaws rejection rate might be higher than other methodologies due its enhanced detection
capabilities. Research published by Florida Department of Transportation (FDOT) compared NDT results among RT, UT and PAUT methods. It was concluded that although PAUT was able to capture minor discontinuities that RT and UT did not detect, the rate of rejection among the three methods were very similar (Wilkinson, 2014).

2.6 Background of Structural Reliability Theory

Due to the fact that fatigue loading as well as fatigue resistance have intrinsic uncertainties, one can state that the probability of failure of any fatigue sensitive detail is always greater than zero. The reliability-based fatigue assessment aims to investigate the probability that the detail will not fail to perform its intended function. Therefore, based on the random variables involved in the fatigue problem, e.g., number of cycles to failure, fatigue detail constant, effective stress range, initial crack size, etc., the accumulation of damage or the propagation of cracks are assessed in a probabilistic manner to compute the corresponding reliability of the detail being investigated.

In order to assess the probability of failure of a structure, or a structure component, limit states functions must be properly defined according to a specific failure mode. In other words, the safety of structure against collapse, limitations on fatigue damage, or on excessive deflections, are evaluated based on the violation of its corresponding limit state functions (Melchers and Beck, 2018). As an example, the basic limit state function is formulated in terms of only one load effect, $Q$, and one resistance, $R$, as shown in Equation 2.69, where $g(X)$ represents the limit state function for the vector $X$ of random variables.

$$g(X) = R - Q$$  \hspace{1cm} (Equation 2.69)
The failure of a structure component occurs whenever the resistance $R$ is less than the load demand $Q$. Therefore, in the space of random variables the safe domain is defined as $g \geq 0$ and the failure domain (or limit state violation) is defined as $g < 0$. Assuming that $R$ and $Q$ are continuous random variables and statistically independent, $R$, $Q$ and $g$ can be described by its probability density function (PDF), as shown in Figure 2.25. As a result, the probability of failure in terms of the limit state function is defined as Equation 2.70.

\[ P_f = P(g < 0) = \int f_g(g) \, dg \quad \text{(Equation 2.70)} \]

In general, though, the limit state function has many variables to represent resistance and load effect, which makes the calculation of the probability of failure much more complicated than the basic case shown in Figure 2.25 and Equation 2.70. Thus, the variables are described by its marginal density functions and the probability of failure is computed based on the integration of the generalized joint probability density function for the n-dimensional vector $X$ of basic variables, $f_X(x)$, over the failure domain in the 3D random variable space, as shown in Equation 2.71 (Melchers and Beck, 2018). Since the solution of this integration is very difficult to evaluate, the reliability index, $\beta$, is used as an alternative to quantify the structure safety.
\[ P_f = P(g(X) < 0) = \int \ldots \int f_X(x) \, dx \quad \text{(Equation 2.71)} \]

In order to clarify the definition of \( \beta \), the fundamental case, where \( R \) and \( Q \) are independent and normally distributed, is assumed. In addition, the reduced variables \( Z_R = (R - \mu_R)/\sigma_R \) and \( Z_Q = (Q - \mu_Q)/\sigma_Q \) are used to express the limit state function previously defined in Equation 2.69 (see Equation 2.72).

\[ g(Z_R, Z_Q) = (\mu_R - \mu_Q) + Z_R \sigma_R - Z_Q \sigma_Q \quad \text{(Equation 2.72)} \]

According to Hasofer and Lind (1974), the reliability index \( \beta \) is defined as the shortest distance from the origin of the reduced variables space to the line \( g(Z_R, Z_Q) = 0 \) (Nowak and Collins, 2013), as shown in Figure 2.26. It is noted that for this special case, the integration of Equation 2.71 has an analytical closed-form solution, and the probability of failure can be defined as in Equation 2.73, where \( \Phi \) is the standard normal distribution function, \( \mu \) is the mean and \( \sigma \) is the variance.

\[ P_f = P(g(Z_R, Z_Q) < 0) = \Phi \left[ \frac{-(\mu_R - \mu_Q)}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \right] = \Phi(-\beta) \quad \text{(Equation 2.73)} \]

Figure 2.26. Definition of the reliability index \( \beta \) (Nowak and Collins, 2013).
2.6.1 Reliability Methods

If the limit state function of the n-dimensional vector $X$ is linear with uncorrelated variables, the reliability index can be calculated by the First Order Reliability Method (FORM) expressed in Equation 2.74. In this method, the PDFs of the random variables are not required for the calculation of $\beta$, yet, if all the variables are normally distributed, the solution of $\beta$ is exact.

$$\beta = \frac{a_0 + \sum_{i=1}^{n} a_i \mu_{X_i}}{\sqrt{\sum_{i=1}^{n} (a_i \sigma_{X_i})}}$$  \hspace{1cm} (Equation 2.74)

where $a_i$ are the constant terms in the limit state function.

For more complex limit state functions, the reliability index may be calculated using simulation techniques such as the Monte Carlo Simulation and the Latin Hypercube Sampling. Both methods are briefly described below.

2.6.1.1 Monte Carlo Simulation

The basic steps to calculate the probability of failure using Monte Carlo (MC) simulation are summarized below:

i. Define the limit state function;

ii. Determine the PDF of each random variable on the limit state function;

iii. Determine the mean (or the shape parameter) and the standard deviation (or the scale parameter) of each random variable.

iv. For each random variable, obtain uniformly distributed random values, $u_i$, varying from 0 to 1.
v. The simulation value of the random variable, $x_i$, is obtained by calculating the inverse of the cumulative density function, $F_X^{-1}$, of $u_i$, as shown in Equation 2.75. This technique is known as the “inverse transform” method.

$$x_i = F_X^{-1}(u_i) \quad \text{(Equation 2.75)}$$

vi. For each set of the simulated values $x_i$, the corresponding limit state function value, $g_i$, is calculated. The estimated probability of failure is calculated by dividing the number of times that $g_i \leq 0$ by the total number of simulation.

2.6.1.2 Latin Hypercube Sampling

The Latin Hypercube Sampling (LHS) is an alternative technique of random sampling that is useful in calculating probability of failures. The main advantage of this method in comparison to the Monte Carlo is the reduced number of simulations needed to obtain reasonable results (Carneiro et al., 2021).

The LHS consists of partitioning the random variable domain in stripes, which are represented once, and only once, by some selected random value within the stripe (Santos and Beck, 2015). Considering $m$ the number of random variables and $n$ the number of samples, a matrix $P(n \times m)$ is created where each column is a random permutation of 1, $\ldots$, $n$, and a matrix $R(n \times m)$, of independent uniformly distributed random numbers from 0 to 1, is created (Santos and Beck, 2015). These matrices form the basic sampling plan, represented by the matrix $S$ (Olsson et al., 2003) in Equation 2.76.

$$S = \frac{1}{n} (P - R) \quad \text{(Equation 2.76)}$$
Finally, the simulated values $x_{ij}$ of the random variable is obtained by Equation 2.77, where $s_{ij}$ are the elements from the matrix $S$.

$$x_{ij} = F_X^{-1}(s_{ij}) \quad \text{(Equation 2.77)}$$

2.6.2 Fatigue Limit State Functions

Depending on the approach used for the fatigue assessment, the limit state function can be defined in terms of fatigue damage accumulation (stress-life approach) or in terms of crack size (LEFM approach).

In the stress-life method approach, the critical fatigue damage accumulation, $\Delta$, represents the resistance side of the limit state function, whereas the fatigue damage accumulation along the time, $D(t)$, represents the load side, as shown in Equation 2.78.

$$g(X, t) = \Delta - D(t) \quad \text{(Equation 2.78)}$$

Both parameters, $\Delta$ and $D$, are based on the linear damage accumulation proposed by Palmgren-Miner’s rule. It has been observed in the literature that the critical damage accumulation can vary from 0.5 and 2.0 (Zhao et al., 1994). Thus, in order to account for the uncertainty values of $\Delta$, Wirsching (1984) proposed a lognormally distributed critical damage accumulation with mean of 1.0 and coefficient of variation (CV) of 0.30. The damage accumulation, $D$, is calculated by the number of cycles of the stress range $i$ over the number of cycles to failure obtained from the appropriate S-N curve (see Equation 2.2).

Therefore, the uncertainties in $D$ are associated with the fatigue loading (stress range spectrum or effective stress range) and the definition of the corresponding S-N curve, e.g., slope and intercept.
In the LEFM approach, the limit state function can be defined by the critical crack size, $a_{cr}$, and the current crack size $a(t)$ corresponding to the number of cycles, $N$ at the time $t$. (Zhao et al., 1994, Cheung et al., 2003, Wang et al., 2009 and Kwon et al., 2012).

$$g(X, t) = a_{cr} - a(t) \quad \text{(Equation 2.79)}$$

### 2.7 Fatigue Behavior Due to Truck Platooning

Truck platooning technology is considered by various researchers to yield positive results in terms of fuel consumption, decrease in CO$_2$ emissions, and increase in highway safety and capacity (Tsugawa et al., 2016, Birgisson, et al., 2020 and Hassan et al., 2020). In order to implement this technology in the future and better understand the effects of truck platooning on bridges, further analysis regarding the safety, serviceability and remaining life is needed.

Previous research regarding the impact of truck platoons on bridges were focused on the live load demands and load ratings factors. Yarnold and Weidner (2019) evaluated the bending moments and shear forces caused by truck platoons made of Florida C5 legal load. Tohme and Yarnold (2020) investigated the load ratings due to truck platoon loading in a range of bridge configuration and span lengths to cover the behavior of multi-girders bridges in US. Sunna and moore (2020) investigated the effects of truck platoons in terms of reactions, bending moments and shear forces in order to evaluate the superstructure and substructure load ratings. Birgisson et al. (2020) and Thulaseedharam (2020) studied the live load demands of truck platoons and the load ratinhs in Texas bridges invetory. Although the live load demands and load ratings were investigated in previous research, from the authors knowledge there is no study that elucidated the impact of truck platooning
on the fatigue behavior of steel bridges. It is stressed that in spite of the higher load effects that truck platooning may cause on the bridge, the number of cycles per truck passage is another variable that must be investigated in order to quantify the cumulative fatigue damage on steel bridges.
CHAPTER 3

FIELD INSTRUMENTATION AND FINITE ELEMENT MODEL OF THE CASE STUDY BRIDGE

3.1 Introduction

A three-span simply supported steel-plate girder bridge located in New Jersey was used as a case study for the remaining fatigue life assessment. The bridge was originally built in 1966 and widened in 1996 when two steel girders were added in the west fascia, and the overhang was doubled on east fascia to accommodate the new exit lane. The fatigue assessment was developed for the bridge center-span, where a diagnostic load test was performed to calibrate the FE model of the superstructure. The short-term monitoring system consisted of the main processing unit that samples data, junction boxes, and strain transducers and accelerometers that were mounted on the girders bottom flange and web. In order to capture the site-specific live load on the bridge, weigh-in-motion sensors, e.g., piezoelectric sensors and inductive loops, were installed in the exit lane and in the right lane, prior to the South Abutment of the bridge. The PAUT was the third field testing performed on the bridge aiming to enhance the fatigue resistance models. Figure 3.1 presents the framing plan of the superstructure as well as the implementation of SHM, WIM and PAUT.
Fatigue analysis was focused on the fascia girder B8 due to the presence of longitudinal stiffener as well as the fact that this girder is underneath the right lane where most of the truck traffic is observed. The PAUT inspection results are presented in Chapter 5.

3.2 Weigh-in-Motion (WIM) System

In order to capture the site-specific live load on the bridge, weigh-in-motion system was implemented prior to bridge structure on May 11th and 12th, 2018. The WIM system consists of WIM sensors installed in the pavement that are connected to a portable WIM data collection unit via a junction node and a terminal block device. Two piezoelectric sensors type piezopolymer (PVDF) and one inductive loop were installed in the exit lane and right lane, prior to the South Abutment of the bridge, as presented in Figure 3.2. The PVDF sensor cables were connected to the junction node that was linked to the data.
collection unit, whereas the loop sensor cables were connected to the terminal block that was also linked to the data collection unit.

Figure 3.2. WIM sensors installation and layout.

A Class 6 truck (according to FHWA classification) with known gross vehicle weight (GVW) and axle spacing was used to calibrate the WIM system, as shown in Figure 3.3. The GVW and axle weights were certified based on a static scale.

Figure 3.3. WIM calibration truck.
The calibration truck run multiple times on the exit lane and right lane, so the calibration factors were defined according to the ASTM E1318-09 performance requirements for Type I and II WIM systems. The results in terms of gross vehicle weight and axle weight are presented in Table 3.1 and Table 3.2.

Table 3.1 – WIM calibration of Gross and Axle Weights – Right Lane.

<table>
<thead>
<tr>
<th>Exact Run #</th>
<th>66,780 lbs</th>
<th>19,820 lbs</th>
<th>44,080 lbs</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>GVV</td>
<td>Steer</td>
<td>Tandem</td>
<td>GVV</td>
</tr>
<tr>
<td>1</td>
<td>60,251</td>
<td>17,066</td>
<td>43,185</td>
<td>9.8%</td>
</tr>
<tr>
<td>2</td>
<td>65,046</td>
<td>18,580</td>
<td>46,465</td>
<td>2.6%</td>
</tr>
<tr>
<td>3</td>
<td>71,308</td>
<td>21,103</td>
<td>50,204</td>
<td>6.8%</td>
</tr>
<tr>
<td>4</td>
<td>72,854</td>
<td>20,552</td>
<td>52,302</td>
<td>9.1%</td>
</tr>
<tr>
<td>5</td>
<td>72,712</td>
<td>20,772</td>
<td>51,939</td>
<td>8.9%</td>
</tr>
<tr>
<td>6</td>
<td>65,238</td>
<td>19,511</td>
<td>45,726</td>
<td>2.3%</td>
</tr>
<tr>
<td>7</td>
<td>69,007</td>
<td>19,669</td>
<td>49,338</td>
<td>3.3%</td>
</tr>
<tr>
<td>8</td>
<td>62,019</td>
<td>19,906</td>
<td>42,113</td>
<td>7.1%</td>
</tr>
<tr>
<td></td>
<td>Average Error</td>
<td>6.2%</td>
<td>4.7%</td>
<td>9.7%</td>
</tr>
<tr>
<td></td>
<td>Std. Dev. Error</td>
<td>2.9%</td>
<td>4.1%</td>
<td>6.2%</td>
</tr>
<tr>
<td></td>
<td>Max. Error</td>
<td>9.8%</td>
<td>13.9%</td>
<td>18.7%</td>
</tr>
<tr>
<td></td>
<td>Min. Error</td>
<td>2.3%</td>
<td>0.4%</td>
<td>2.0%</td>
</tr>
<tr>
<td></td>
<td>ASTM E1318 Standard Type I</td>
<td>10%</td>
<td>20%</td>
<td>15%</td>
</tr>
<tr>
<td></td>
<td>ASTM E1318 Standard Type II</td>
<td>15%</td>
<td>30%</td>
<td>20%</td>
</tr>
<tr>
<td>* Not complying with Type I; # Not comply with Type II</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2 – WIM calibration of Gross and Axle Weights – Exit Lane.

<table>
<thead>
<tr>
<th>Exact Run #</th>
<th>66,780 lbs</th>
<th>19,820 lbs</th>
<th>44,080 lbs</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>GVV</td>
<td>Steer</td>
<td>Tandem</td>
<td>GVV</td>
</tr>
<tr>
<td>1</td>
<td>56,182</td>
<td>14,573</td>
<td>41,609</td>
<td>15.9%#</td>
</tr>
<tr>
<td>2</td>
<td>59,830</td>
<td>17,727</td>
<td>42,103</td>
<td>10.4%#</td>
</tr>
<tr>
<td>3</td>
<td>67,482</td>
<td>18,642</td>
<td>48,840</td>
<td>1.1%</td>
</tr>
<tr>
<td>4</td>
<td>68,888</td>
<td>19,703</td>
<td>49,185</td>
<td>3.2%</td>
</tr>
<tr>
<td>5</td>
<td>72,067</td>
<td>19,777</td>
<td>52,290</td>
<td>7.9%</td>
</tr>
<tr>
<td>6</td>
<td>72,125</td>
<td>19,883</td>
<td>52,242</td>
<td>8.0%</td>
</tr>
<tr>
<td>7</td>
<td>71,582</td>
<td>19,161</td>
<td>52,420</td>
<td>7.2%</td>
</tr>
<tr>
<td>8</td>
<td>75,052</td>
<td>19,644</td>
<td>55,408</td>
<td>12.4%*</td>
</tr>
<tr>
<td></td>
<td>Average Error</td>
<td>8.2%</td>
<td>6.0%</td>
<td>14.3%</td>
</tr>
<tr>
<td></td>
<td>Std. Dev. Error</td>
<td>4.5%</td>
<td>8.4%</td>
<td>6.9%</td>
</tr>
<tr>
<td></td>
<td>Max. Error</td>
<td>15.9%</td>
<td>26.5%</td>
<td>25.7%</td>
</tr>
</tbody>
</table>
It is noted that Type I and Type II WIM Systems are identified as suitable for traffic data collection where the vehicle speed range is required to be between 10 mph to 80 mph (FHWA, 2018). The ASTM E1318-09 performance criteria for different WIM systems is given in Table 3.3.

### Table 3.3 – ASTM E1318-09 Performance criteria.

<table>
<thead>
<tr>
<th>Function</th>
<th>Type I</th>
<th>Type II</th>
<th>Type III</th>
<th>Type IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wheel Load</td>
<td>±25 %</td>
<td>±20 %</td>
<td>±15 %</td>
<td>±10 %</td>
</tr>
<tr>
<td>Axle Load</td>
<td>±20 %</td>
<td>±30 %</td>
<td>±16 %</td>
<td>±10 %</td>
</tr>
<tr>
<td>Axle-Group Load</td>
<td>±15 %</td>
<td>±20 %</td>
<td>±10 %</td>
<td>±6 %</td>
</tr>
<tr>
<td>Gross-Vehicle Weight</td>
<td>±10 %</td>
<td>±15 %</td>
<td>±6 %</td>
<td>±6 %</td>
</tr>
<tr>
<td>Speed</td>
<td>±1 mph (2 km/h)</td>
<td>±0.5 ft (0.15 m)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Higher values are not usually a concern in enforcement.

3.2.1 WIM data filtering criteria

Weigh-in-Motion data was collected from October 2018 to September 2019, contemplating more than 4 million vehicles. In order to discard erroneous weigh-in-motion measurements and to identify the qualified truck population, the data was filtered according to NCHRP 12-83 criteria (Wassef et al., 2014), which is presented in Figure 3.4. The qualified trucks for the regular truck traffic population are obtained after the filtering criteria 1 and 2 were applied. For fatigue analysis purposes a third filter is apply to exclude trucks with less than 20 kips which do not cause considerable fatigue damage on the bridges. This threshold of 20 kips was established by Fisher (1977).
3.2.2 WIM data quality assurance

The quality assurance of the WIM data was verified based on the procedure proposed by Southgate (2000). The author used a logarithmic regression of the steering axle spacing and weight to check the quality of WIM data. The concept of verifying whether the collected WIM data is realistic requires comparison against an outside source of data. Therefore, according to Southgate (2000) the following criteria provide firm trends and boundary conditions. The criteria was developed based on the following:

a) 12000 lb legal limit load for a steering axle, also manufacture’s maximum load;
b) analyses of static scale data for over 4000 5-axle semi-trailer trucks;
c) published data from truck-tractor manufactures based on vehicle as it sits on the dealer’s lot – no driver, no fuel, and no trailer;
d) providing a tolerance of 50 lb to a) and c) above in an attempt to account for differences between dynamic and static conditions. The “50lb” is an arbitrary number.

Since WIM sensors are installed per lane, the evaluation for quality assurance is performed per lane. The application of the Southgate method has the following step-by-step:

1. One day of raw data containing only 5-axle semi-trailer trucks is used to represent the corresponding set of raw data;
2. For each truck recorded, divide the steering axle weight (AW1) by the axle spacing between the front axle and the following axle (AS1);
3. Take the base 10 logarithm of the ratio (AW1)/(AS1);
4. Calculate the base 10 logarithm of (AS1);
5. Plot the log(AS1) in the x-axis and log((AW1)/(AS1)) on the y-axis and find the coefficients (slope M and intercept B) for the linear regression line;
6. Calculate the “log regression R” of each data point such that 
   \[ R = 10^{[B+M \times \log(AS1)]}; \]
7. Calculate the “reference regression E” by 
   \[ E = 10^{[3.925361−0.952182\times \log(AS1)]}; \]
8. Define the upper bound of the regression as the maximum steering axle weight to spacing ratio (by manufactures specifications) as 
   \[ MAX = [12000/(AS1)] + 50; \]
9. Define the lower bound of the regression of the steering axle as 
   \[ MIN = 10^{[3.942369−1.075085\times \log(AS1)]}; \]
10. Plot: (AW1)/(AS1), the regression R, the reference E, the upper bound MAX and the lower bound MIN for each truck.

In this study, it is considered that each month is a set of WIM data. Thus, the quality assurance following the Southgate procedure is performed twelve times for the exit lane (Lane 1) and right lane (Lane 2). The quality assurance procedure was applied in WIM data
after erroneous readings were removed based on the NCHRP 12-83 filtering criteria 1. The results corresponding to September 2019 is presented in Figure 3.5 Error! Reference source not found. The analysis for the other months are presented in the Appendix B.

![Figure 3.5. WIM quality control for the studied bridge – September 2019.](image)

The overall results show that the regression line falls in between the upper and lower bound and it is very close to the reference line, meaning that the quality of the WIM data is acceptable.

### 3.2.3 WIM data counts and statistics

The counts of trucks per FHWA Classification and per number of axles are summarized in Table 3.4 and Table 3.5. It is observed that more than 50% of the trucks are Class 9, whereas Class 5 and Class 6 are the second and third most frequent trucks.

<table>
<thead>
<tr>
<th>Truck Class</th>
<th>Number of occurrences</th>
<th>Frequency (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class 4</td>
<td>5375</td>
<td>2.0%</td>
</tr>
<tr>
<td>Class 5</td>
<td>60813</td>
<td>22.4%</td>
</tr>
<tr>
<td>Class 6</td>
<td>31753</td>
<td>11.7%</td>
</tr>
<tr>
<td>Class 7</td>
<td>8296</td>
<td>3.1%</td>
</tr>
<tr>
<td>Class</td>
<td>Count</td>
<td>Frequency (%)</td>
</tr>
<tr>
<td>---------</td>
<td>-------</td>
<td>---------------</td>
</tr>
<tr>
<td>Class 8</td>
<td>12257</td>
<td>4.5%</td>
</tr>
<tr>
<td>Class 9</td>
<td>146069</td>
<td>53.8%</td>
</tr>
<tr>
<td>Class 10</td>
<td>2786</td>
<td>1.0%</td>
</tr>
<tr>
<td>Class 11</td>
<td>2710</td>
<td>1.0%</td>
</tr>
<tr>
<td>Class 12</td>
<td>1444</td>
<td>0.5%</td>
</tr>
<tr>
<td>Class 13</td>
<td>213</td>
<td>0.1%</td>
</tr>
</tbody>
</table>

Table 3.5 – Counts of trucks per number of axles.

<table>
<thead>
<tr>
<th>Number of axles</th>
<th>Number of occurrences</th>
<th>Frequency (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-axle</td>
<td>63743</td>
<td>23.46%</td>
</tr>
<tr>
<td>3-axle</td>
<td>36448</td>
<td>13.41%</td>
</tr>
<tr>
<td>4-axle</td>
<td>18041</td>
<td>6.64%</td>
</tr>
<tr>
<td>5-axle</td>
<td>149036</td>
<td>54.85%</td>
</tr>
<tr>
<td>6-axle</td>
<td>3802</td>
<td>1.40%</td>
</tr>
<tr>
<td>7-axle</td>
<td>497</td>
<td>0.18%</td>
</tr>
<tr>
<td>8-axle</td>
<td>113</td>
<td>0.04%</td>
</tr>
<tr>
<td>9-axle</td>
<td>36</td>
<td>0.01%</td>
</tr>
<tr>
<td>10-axle</td>
<td>0</td>
<td>0%</td>
</tr>
<tr>
<td>11-axle</td>
<td>0</td>
<td>0%</td>
</tr>
</tbody>
</table>

The GVW histogram plotted in Figure 3.6 presents a bi-modal distribution that might be attributed to the quantity of lighter trucks as 2-axle and 3-axle trucks that travel in the site, as indicated in Table 3.3. To corroborate this idea the GVW histograms shown in Figure 3.7 is divided in two groups, trucks with 2 and 3 axles (Figure 3.7a); and trucks with 4 to 9 axles (Figure 3.7b). It is observed that for the first group (2 and 3 axles truck) the distribution is not normal and resembles the first modal in the histogram presented in Figure 3.6, whereas for the second group (4 to 9 axles truck) the histogram looks like a normal distribution.
Figure 3.6. GVW Histogram for the WIM data collected from October 2018 to September 2019.

(a) Trucks with 2 and 3 axles                     (b) Trucks with 4 to 9 axles

Figure 3.7. GVW histogram divided in two groups.

The mean and maximum GVW are presented in each plot by the dashed blue and red lines, respectively. The maximum recorded GVW was 235.5 kips and the average GVW is 47 kips. The percentage of overweight trucks is calculated based on the legal gross weight limit of 80 kips. It was found that 8.58 % of the trucks are above the legal limit.
3.3 Structural Health Monitoring (SHM)

Structure Health Monitoring (SHM) was implemented in the center span of the studied bridge. For the short-term monitoring, the Structural Testing System (STS) manufactured by Bridge Diagnostics Inc. (BDI) was used to perform the diagnostic load test. The system consists of the main processing unit that samples data, junction boxes, strain transducers, and accelerometers, as shown in Figure 3.8.

![Figure 3.8. Structural Testing System (STS) from BDI.](image)

The installation plan for the short-term strain transducers and accelerometers is presented in Figure 3.1. The detailed location for the short-term sensors is presented in Table 3.6 and also in Appendix C.

<table>
<thead>
<tr>
<th>Girder</th>
<th>SENSOR #</th>
<th>Distance from South Pier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fascia Girder (B8)</td>
<td>B2044</td>
<td>29’</td>
</tr>
<tr>
<td></td>
<td>B3231</td>
<td>49’-1”</td>
</tr>
<tr>
<td></td>
<td>B3678</td>
<td>49’-1”</td>
</tr>
<tr>
<td></td>
<td>A3008</td>
<td>48’-4”</td>
</tr>
<tr>
<td></td>
<td>B2492</td>
<td>72’</td>
</tr>
<tr>
<td>Web</td>
<td>B2060</td>
<td>28’-8”</td>
</tr>
<tr>
<td></td>
<td>B3683</td>
<td>42’-8”</td>
</tr>
<tr>
<td></td>
<td>B3680</td>
<td>68’</td>
</tr>
<tr>
<td>Adjacent Interior Girder (B9)</td>
<td>B3682</td>
<td>27’-10”</td>
</tr>
<tr>
<td></td>
<td>B3227</td>
<td>45’-3”</td>
</tr>
</tbody>
</table>

![a) Junction node   (b) Main unit   (c) Strain transducer   (d) Accelerometer](image)
For the long-term monitoring, strain gages, tiltmeters and accelerometers were installed in the east fascia girder B8 and in its adjacent interior girder B9, as shown in Figure 3.1. The detailed location for the long-term sensors are presented in Table 3.7 and also in Appendix C.

Table 3.7 – Sensor Longitudinal Position for Long-Term Monitoring

<table>
<thead>
<tr>
<th>Girder</th>
<th>SENSOR #</th>
<th>Distance from South Pier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fascia Girder (B8) BF</td>
<td>B1002</td>
<td>45’-3”</td>
</tr>
<tr>
<td></td>
<td>A1196</td>
<td>45’-3”</td>
</tr>
<tr>
<td>Adjacent Interior Girder (B9)</td>
<td>B1001</td>
<td>45’-7”</td>
</tr>
<tr>
<td></td>
<td>A1197</td>
<td>46’-0”</td>
</tr>
</tbody>
</table>

The long-term sensors were wire-connected to the data logger where the BDI software was installed for the management of the long-term data collection. One modem was connected to the data-logger for the purpose of remote communication. The whole system was located underneath the bridge within an aluminum white enclosure. The electric power source was provided by two 12V batteries that were being charged by two solar panels whenever there was enough sunlight. The configuration of the long-term SHM system is shown in Figure 3.9.
3.3.1 Diagnostic Load Test

In order to investigate the bridge response subjected to live load, a diagnostic load test was performed. Since the main girder to be investigated was the east fascia girder B8, the calibration truck, with known weight and configuration (see Figure 3.10) run on the exit lane, right lane and middle lane. Table 3.8 summarizes all the runs performed on the bridge indicating the speed used in each run, e.g., 20 mph, 30 mph or 40 mph.

![Calibration truck weight and configuration.](image)

**Figure 3.10.** Calibration truck weight and configuration.

<table>
<thead>
<tr>
<th>Run</th>
<th>Lane</th>
<th>Speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Run 1</td>
<td>Exit Ramp Lane</td>
<td>20 mph</td>
</tr>
<tr>
<td>Run 2</td>
<td>Exit Ramp Lane</td>
<td>30 mph</td>
</tr>
<tr>
<td>Run 3</td>
<td>Exit Ramp Lane</td>
<td>40 mph</td>
</tr>
<tr>
<td>Run 4</td>
<td>Exit Ramp</td>
<td>20 mph</td>
</tr>
<tr>
<td>Run 5</td>
<td>Right Lane</td>
<td>20 mph</td>
</tr>
</tbody>
</table>
The strain data recorded in Run 1 for the bottom flange of girders B8, B9, B11 and B14 are presented in Figure 3.11. All the other results are presented in the Appendix C.

![B8 Bottom Flange Strain - Run 1](image1)
(a) Girder B8 Bottom Flange Strain

![B9 Bottom Flange Strain - Run 1](image2)
(b) Girder B9 Bottom Flange Strain

![B11 Bottom Flange Strain - Run 1](image3)
(c) Girder B11 Bottom Flange Strain

![B14 Bottom Flange Strain - Run 1](image4)
(d) Girder B14 Bottom Flange Strain

Figure 3.11. Bottom flange strain on Girders B8, B9, B11 and B14 yielded by Run 1.
3.4 Finite Element Model of the Bridge

The FE model of the bridge was developed in ABAQUS/CAE 2017 considering linear elastic behavior of the structure. All the structural member dimensions were extracted from the bridge design plans, as detailed in Appendix D. The girder webs and the concrete deck were modeled using a four-node quadrilateral shell element while the girder flanges, cross-frame members, diaphragms, and stiffeners were modeled with two-node beam elements. The concrete material with $E = 3450 \text{ ksi}$ and $\nu = 0.20$ was assigned in the deck and the steel material with $E = 29000 \text{ ksi}$ and $\nu = 0.30$ was assigned to all steel members.

According to the bridge design-plans, shear studs were distributed on the girder top flanges to assure the composite behavior between the concrete deck and the steel girders. Although the shear studs were not considered in the FE model, multi point constraint was employed to guarantee the composite behavior. This connection provides a rigid beam between two nodes to constraint the displacement and rotation at the first node to the displacement and rotation at the second node, corresponding to the presence of a rigid beam between the two nodes (Abaqus/CAE User’s Guide). Therefore, the MPC-beam constraint was applied to link the nodes of the girder top flanges to the corresponding aligned nodes in the concrete deck, as shown in Figure 3.12. This type of connection was also applied to link the cross-frames and diaphragms to their respective connection plates.
The boundary conditions used in the model followed the information provided in the design plans, e.g., pin connections at the South Pier (fixed bearings) and roller connections at the North Pier (expansion bearings).

For the girders mesh, two hundred and seventy-six elements were used throughout the girder’s length. Therefore, both the beam elements for the flanges and the shell elements for the webs were 4-inch long. For girders B8, B14, S3 and S4, twelve elements were used through the depth of the web resulting in shell elements of 4 inches by 4 inches. Likewise, for girders B9 to B13, nine elements were used through the depth of the web, resulting in shell elements of 4 inches by 5 inches. The size of the beam element assigned to stiffeners followed the same pattern of the web mesh. Lastly, all the cross-frame and diaphragm members were discretized with beam elements less than 6-inch long. Figure 3.13 shows the mesh applied in the steel structural members. The coincident nodes from the girders flanges and webs were merged. Similarly, coincident nodes among stiffeners and webs were merged.
The concrete deck mesh followed similar pattern as the girder mesh. In the longitudinal direction 4-inch-long shells were used. Thus, shell elements of 4 inches by 4 inches and 4 inches by 3 inches were used accordingly to the different girder spacings and overhangs. Further, in both deck ends, a irregular mesh was applied due to the skewed piers.

3.4.1 Calibration of the FE model based on the Diagnostic Load Test

Unit loads were applied on the concrete deck in the regions corresponding to the lanes used in the Diagnostic Load Test, e.g., exit lane, right lane, and middle lane. The influence surfaces were extracted at the points where the sensors were installed. The strain response obtained in the FE model on several locations were compared to the strain measurements obtained in the field. It was observed that the real structure did not behave as a simply supported span. Figure 3.14 shows the strain in the bottom flange of girder B8 recorded by sensors B3678 and B2492, for Run 1 on exit lane compared with the FE model results.
The magnitude of strain obtained in the FE model was almost twice the strain recorded in the field. For the case of sensor B3678, the FE model resulted in a maximum static strain of 117.5 με whereas the field measurement resulted in a maximum of 63.9 με. Moreover, the shape of the strain curve obtained in the field test indicate that there is additional constraint in the bearings, as it can be seen by the zero-strain result when the front-axle of the truck was located at 20 ft for the case of sensor B3678 and 40 ft for the case of sensor B2492. Therefore, a sensitivity analysis regarding the boundary conditions was performed in order to calibrate the FE model. It was concluded that the numerical simulation considering fix supports at the South Pier and pin supports at the North Pier presented the closest behavior to the real strain measurement obtained in the diagnostic load test.

Figure 3.15 to Figure 3.19 shows the results for the calibration of girders B8, B9, B11 and B14, based on the strain transducers installed on the bottom flange. For girders B8 and B9, the analysis was developed considering the calibration truck running on exit
lane and right lane. Likewise, for girders B11 and B14, the analysis considered the truck running on the middle lane. Further results are presented in the Appendix D.

Figure 3.15. Strain response in the bottom flange of girder B8 – truck on the Exit Lane.
Figure 3.16. Strain response in the bottom flange of girder B8 – truck on the Right Lane.

Figure 3.17. Strain response in the bottom flange of girder B9 – truck on the Exit Lane.
For all cases, the Pin-Roller model (red curve) provides higher strain responses than the Field Test. In general, the Pin-Pin model (blue curve) provides similar results to the
Fixed-Pin model (magenta curve). However, the Fix-Pin model presented the most consistent results with the field measurements.

The strain response from the sensors installed in the mid-web of fascia girder B8 is presented in Figure 3.20.

![Strain response graphs](image)

Figure 3.20. Strain response in the mid-web of girder B8 – sensors B2060 and B3680.

The Fixed-Pin model matches well with the field test measurement for the case of the strain transducer B2060, as it can be seen in Figure 3.20a and Figure 3.20c. However, for the case of the strain-transducer B3680, the Fixed-Pin model provided much lower strain than the
one recorded in the field test. It is worth mentioning that visible scratches caused by the impact of over-height truck in the bottom flange of the fascia girder B8 were observed nearby the midspan. Therefore, the high strain values observed in the mid-web provided by sensor B3680 could be related to an initial deformation caused by the hit of the truck in that region which yields secondary effects in the web. In addition to the scratches a bent cross-frame that braces girder B8 to girder B9 was observed, as depicted in Figure 3.21.

![Scratches on the bottom flange of B8](image1.png)  ![Bent cross-frame that braces B8 and B9](image2.png)

(a) Scratches on the bottom flange of B8        (b) Bent cross-frame that braces B8 and B9

Figure 3.21. Scratches on the bottom flange of girder B8 and bent cross-frame.

The strain response from the sensors installed in the mid-web of the adjacent interior girder B9 is presented in Figure 3.22.
Figure 3.22. Strain response in the mid-web of girder B9.

For both sensors, the maximum strain value as well as the shape of the curve obtained in the Fix-Pin model matched well with the field test results. For the test performed on the exit lane, the maximum strain recorded by sensor B3238 was 18 με whereas the Fix-Pin model resulted in maximum strain of 15 με (difference of 16%). For the case of sensor B3219, the high strain obtained in the web could also be related to the hit of the truck in the bottom flange of fascia girder B8, since both girders are braced by cross-frames.

3.4.2 Calibration of the FE model based on long-term strain gages

The original diagnostic load test was performed in October 2018. In order to confirm the structural behavior of the bridge, an additional diagnostic load test was performed two years later, on October 14th, 2020. The strain measurement was recorded by the long-term strain gages B1002 and B1001 that remained installed on the bottom
flanges of girders B8 and B9, respectively. Table 3.5 presented the exact location of the strain gages.

A three-axle dump truck with gross vehicle weight of 72700 lb was used as the calibration truck in the new test. The truck weight and configuration are shown in Figure 3.23.

![Three-axle dump truck](image1)

(a) Three-axle dump truck           (b) Axle weights and spacing       (c) Truck width

Figure 3.23. Calibration truck used in the new diagnostic load test.

A total of eighteen runs were performed (twelve runs on the exit lane and six runs on the right lane). The truck speed among all runs varied from 37.5 mph (minimum speed recorded in Run 5) and 42 mph (maximum speed recorded in Run 16). The comparison of the strain data collected in the field with the FE model are presented in Figure 3.24 for Run 3 on exit lane and in Figure 3.25 for Run 2 on right lane.

![Calibration truck on Exit Lane – Run 3](image2)
(b) Girder B8 strain response  

(c) Girder B9 strain response  

Figure 3.24. Strain response on girders B8 and B9 – truck on Exit Lane  

(a) Calibration truck on Right Lane – Run 2  

(b) Girder B8 strain response  

(c) Girder B9 strain response  

Figure 3.25. Strain response on girders B8 and B9 – truck on Right Lane
Based on the results of Figure 3.24 and Figure 3.25 it is confirmed that the FE model with the fix supports at the South Pier and pin supports at the North Pier is the best model that captures the real behavior of the structure.

3.4.3 Verification of the FE model based on WIM data

As described before, WIM sensors were installed to monitor the real live load on the bridge. The sensors were installed only on the exit lane and on the right lane since the main objective was to investigate the structural behavior of the east fascia girder B8. Since the long-term strain gages remained installed on the bottom flanges of girders B8 and B9, further verification of the FE model was performed using random trucks recorded in the WIM data and the strains recorded in the long-term SHM system.

The procedure consisted in extracting the truck weight and configuration from the raw WIM data and applying it in the influence surfaces extracted from the FE model for the points where the strain gages B1002 and B1001 were installed. The simulated strains in the FE model were compared to the strain recorded in SHM system. Figure 3.26 presents an example for a five-axle truck captured on the Exit Lane. Other results are shown in the Appendix D.

(a) Five-axle truck (Class9)  (b) Truck weight and configuration based on WIM data
3.4.4 Unreliable Mechanism

Although the investigated superstructure was designed as a simply supported span, the diagnostic load test with the FE model simulations indicate that the bridge is behaving as a fixed-pinned span. This condition could have happened due to frozen bearings (Transportation Research Circular E-C257, 2019) or lack of maintenance in the bearings. According to the MBE 3rd Edition (2019 with Errata), simply supported bridges are assumed to not carry any moment at the supports. However, tests have shown that there can be significant end moments attributable to frozen bearings. One of the consequences of this condition is that the magnitude of stresses and strains at midspan for example, are much lower than the simply supported case. The discussed example given in Figure 3.14...
showed that the strain magnitude between the simply-supported model and the field measurement could be as high as two times in different location in the girder bottom flange.

According to (Transportation Research Circular E-C257, 2019) it is important to remove unreliable mechanisms such as frozen bearings that would affect the structure evaluation at the ultimate limit state. Furthermore, for load rating purposes, the MBE 3rd Edition do recommend avoiding such restraints detected during the load test.

Since the time when the unintended fixity started is unknown, the fatigue assessment performed in this research considered the original bearing design, e.g., the simply-supported condition.

3.5 Field Testing and FEM for the remaining fatigue life assessment.

The field testing performed on the investigated bridge and its Finite Element Model were employed for the remaining fatigue life assessment, as shown in Figure 3.27.

![Diagram of Fatigue Assessment Methodologies](image)

Figure 3.27. Fatigue Assessment Methodologies.
CHAPTER 4

FATIGUE DETAILS AND ASSESSMENT BASED ON THE STRESS-LIFE METHOD

The critical fatigue prone details were identified based on the bridge design plans and visual inspection. The details were classified according to AASHTO LRDF provisions and the remaining fatigue life was evaluated following the MBE 3rd Edition (2019) approach, using the current AASHTO S-N curves. Alternatively, the database used for the formulation of AASHTO S-N curves was revisited and new curves based on Piecewise Regression were developed and employed for the fatigue assessment. The fatigue loading considered for this analysis was based on the site-specific truck population measured in the WIM system coupled with the FE model of the bridge. Alternatively, the AASHTO Fatigue design truck was also employed in the assessment.

4.1 Weld details identified in the bridge design plans

According to the original design plan Bridge Sheet № B13/B28, the welded joints were designed either as fillet welds or groove/butt welds. Figure 4.1 and Figure 4.2 show the welding details for web – flanges connections as well as web – stiffeners connections.
The weld symbols follow the definitions given in the American Welding Society A2.4, Standard Symbols for Welding, Brazing and Nondestructive Examination (AWS, 2012). Therefore, according to the information provided in the design plans, it can be stated that:

- Double fillet weld was used to connect web to flanges.
- X-groove butt weld was used to splice web plates.
- X-groove butt weld was used to connect bottom flange plates with different thickness.
- Double fillet weld was used to connect transverse stiffeners to the web and to the flanges.
- Double fillet weld was used to connect longitudinal stiffeners to the web.

![Figure 4.1. Welding details for girder web and flanges.](image)

![Figure 4.2. Welding details for girder stiffeners.](image)
Figure 4.2b also shows that the longitudinal stiffener was designed with plates of 6 inches by 0.5 inches along the full length (“F.L.” stands for full length) of the fascia girders. It is noted that although the design plans did not provide any information regarding longitudinal stiffener splices, it is common practice to use plates of 20 ft long and splice them through groove butt welds. The longitudinal stiffener weld splices were confirmed by visual inspection as depicted in Figure 4.3.

![Figure 4.3. Longitudinal stiffener weld splice](image)

In general, the welding details for the new girders added in the west fascia follow the same pattern of the original girders. However, the detail used for the flange thickness transition was different. As mentioned before the X-groove weld was used to connect the flanges with different thickness in the original girders. The detail presented in the widened design plans indicates a V-groove with partial joint penetration, as shown in Figure 4.4.
4.2 Fatigue-prone details

Based on the revision of the design plans, fatigue details were identified and classified per AASHTO LRFD Table 6.6.1.2.3-1, whenever possible. Mainly, the midspan portion of the girders were considered due to the higher stresses. Also, the analysis focused on fascia girder B8.

Detail 1 – Web-to-flange connections

According to the design plans, the flanges were welded to the girder webs through double fillet welds, as depicted in Figure 4.2. The fatigue category detail per AASHTO LRFD classification, is defined as category B. The corresponding constant amplitude fatigue threshold \((\Delta F)_{TH}\) is 16 ksi.

Detail 2 – Transverse stiffeners connections

The transverse stiffeners are double fillet welded to the girder web and flanges, according to Figure 4.2. The fatigue category detail per AASHTO LRFD classification, is defined as category C’. The corresponding constant amplitude fatigue threshold \((\Delta F)_{TH}\) is 12 ksi.

Detail 3 – Bracing Connections
From the widened design plans, the cross-frame members were bolted to the girder connection plates. The fatigue category detail per AASHTO LRFD classification, is defined as category B with constant amplitude fatigue threshold $(\Delta F)_TH$ of 16 ksi.

Detail 4 – Shear Connectors

From the original design plans, bulb angles were welded on the girder top flanges to act as shear connectors between the reinforced concrete deck and the steel plate girders. The fatigue category detail per AASHTO LRFD classification, is defined as category D, considering that the length $L$ of the bulb angle is in between 2 inches and 4 inches. The corresponding constant amplitude fatigue threshold $(\Delta F)_TH$ is 7 ksi.

Detail 5 – Longitudinal stiffener

In terms of fatigue assessment, three cases are possible for the longitudinal stiffener weld details. From the three possible details, only Detail 5A and 5B are classified by AASHTO LRFD.

- Detail 5A: considering that the stiffener-to-web weld is continuous, the corresponding detail classification per AASHTO LRFD is category B. The constant amplitude fatigue threshold $(\Delta F)_TH$ is 16 ksi for this category.

- Detail 5B: considering that the stiffener-to-web weld is not continuous, the corresponding detail classification per AASHTO LRFD is category E. The constant amplitude fatigue threshold $(\Delta F)_TH$ is 4.5 ksi for this category.

- Detail 5C: longitudinal stiffener weld splice. Although this detail was not informed in the design plans, from visual inspection it was observed that the segments of the longitudinal stiffener plates were connected through butt welds, as depicted in Figure 4.3. According to NCHRP Report 227 (Fisher et al., 1980) no direct
comparison between this detail and the other AASHTO details (A to E’) could be made. This is because the severity of this detail is highly dependent on the degree of the unfused weld that splices the longitudinal stiffener plates. Therefore, no classification was possible in this case.

Detail 6 – Bottom flange thickness transition of the new girders S3 and S4

Two cases were considered for the bottom flange thickness transition of the new girders S3 and S4.

- Detail 6A: assuming that the flange thickness transition followed the recommendations given in the original design plans, e.g., double-sided X-groove weld with complete joint penetration (see Figure 4.1), the detail classification per AASHTO LRFD is category B. The constant amplitude fatigue threshold \( (\Delta F)_{TH} \) is 16 ksi for this category.

- Detail 6B: assuming that the flange thickness transition complies with the detail provided in the widened design plans, e.g., single-sided V groove weld with partial joint penetration (see Figure 4.4), there is no suitable classification in AASHTO LRFD for this case. In fact, transversely loaded partial penetration groove welds were prohibited in the release of AASHTO Standard Specification 1989 (see note c below the fatigue detail classification table from AASHTO Standard Specification 1996 in Figure 4.5). Therefore, further investigation is needed to assess the fatigue behavior.
Figure 4.5. Note in AASHTO Standard Specification about transversely loaded groove welds.

Appendix E shows the actual details related to the corresponding classification in AASHTO LRFD Table 6.6.1.2.3-1.

Detail 7 – Web splice

From the original design plans, the web splices were made by double-sided X-groove weld with complete joint penetration, as depicted in Figure 4.1. According to AASHTO LRFD Table 6.6.1.2.3-1 and Fisher (1977), when the weld reinforcement at the groove weld is removed from the web splice, the fatigue category is improved from category C to category B. Based on visual inspection one can state that the groove weld at the web splice was not ground flush to surface, as depicted in Figure 4.6. Thus, the fatigue category detail per AASHTO LRFD classification, is defined as category C. The corresponding constant amplitude fatigue threshold \((\Delta F)_{TH}\) is 10 ksi.

Figure 4.6. Groove weld at the web splice on east fascia girder B8.
4.3 Investigation on the fatigue quality of Detail 6B

According to Lassen and Recho (2006), a one-side V-groove weld has much poorer fatigue quality than a double-sided X groove weld. To corroborate this idea, the stress concentration at the weld toe or weld root of the flange transition is investigated through finite element modeling. Three different weld configurations were studied:

a) Double-sided X groove – complete joint penetration (X-groove – detail 6A)
b) Single-sided V groove - partial joint penetration (V-groove – detail 6B)
c) Double-sided V groove - partial joint penetration (Double V-groove – International Institute of Welding detail category 36 - Hobbacher, 2016)

The finite element model was developed considering the plane strain condition. Triangle shell elements CPS3 from Abaqus library were employed. The boundary conditions and the pressure of 1 ksi applied at one end of the detail are presented in Figure 4.7.

![Figure 4.7. Boundary conditions and load applied – single-sided V groove case.](image)

The stress concentration results for each case are presented below in Figure 4.8. Among the investigated weld configurations, it is confirmed that the best scenario, e.g., the configuration that produces the least stress concentration, is the double-sided X groove with complete joint penetration (case a – detail 6A: yielded $k_t = 4.62$ at the weld toe). The highest stress concentration was observed in case b ($k_t = 16.57$ at weld root), confirming
that the detail 6B has poor fatigue quality. The double-sided V-groove with partial joint penetration (case c) is in between cases a) and b).

(a) Double-sided X-groove (Detail 6A): stress concentration $k_t = 4.62$ at weld toe.

(b) Single-sided V-groove (Detail 6B): stress concentration $k_t = 16.57$ at weld root.

(c) Double-sided V-groove: stress concentration $k_t = 8.52$ at weld root.

Figure 4.8. Stress concentration $k_t$ for each notch types a), b) and c).

These results are in agreement to the detail categories and its fatigue limits presented in Hobbacher (2016), depicted herein in Figure 4.9. Analyzing the fatigue strength for the steel material (column FAT St. in Figure 4.9), the fatigue limit of 112 MPa for the detail N° 211 (equivalent to case b) is much higher than the fatigue limit of 36 MPa for detail N° 217 (equivalent to case c).
Furthermore, the results are in agreement with the findings of Bai and Jin (2016). The notch stress concentration factor at the weld root for the partial joint penetration case is much higher than the notch stress concentration factor at the weld toe for the complete joint penetration case. Bai and Jin (2016) investigated the stress concentration in terms of notch radius $\rho$, welt toe angle $\theta$, thickness of the plate $t$, root face length $g$ and the reinforcement width between the weld toes $w$, as presented in Figure 4.10.

![Figure 4.10. Notch stress concentration for different types of butt welds (Bai and Jin, 2016).](image)

(a) Full penetration case.  
(b) Partial penetration case.

It is recalled that AASHTO LRFD and EN-1993-1-9 do not have fatigue category details corresponding to single-sided V groove butt weld (Detail 6B). Therefore, based on
this investigation, the Detail 6B is hypothetically assumed to be AASHTO category E’ for the purpose of the fatigue assessment via MBE approach.

4.4 Fatigue Loading on the Identified Fatigue Details

4.4.1 Fatigue loading based on WIM data

For each detail identified in section 4.2 the fatigue load spectrum was obtained by coupling the WIM measurements with the influence surfaces extracted from the FE model in the specific detail location. The rainflow counting method was applied to the variable amplitude fatigue load spectrum to count the number of stress range cycles. Hence, for each detail the stress spectrum was reduced to a stress range histogram. Since the bridge FE model developed in Abaqus considered only the static behavior of the bridge, the dynamic load allowance of $IM=1.15$ for the fatigue limit state according to AASHTO LRFD Table 3.6.2.1-1 was employed.

4.4.1.1 Rainflow

Fatigue load spectrum may be reduced to simple histograms by the application of suitable cycle-counting technique. The rainflow counting method, firstly proposed by Matsuishi and Endo (1968), is the most popular technique to count stress (or strain) range cycles for fatigue assessment. According to O’Connor et al. 2010, rainflow cycle counting has shown to be among the superior methods for cycle counting of irregular loads, e.g., variable amplitude loading. The method is based on an ingenious analogy between the generation of hysteresis loop and the flow of raindrops down the roof of a Japanese pagoda.
(Manson and Halford, 2006). Hence, the authors named the method as “rainflow” referring to the raindrops on a Japanese pagoda roof.

In this method, the load-history is rotated in 90° such that the time axis is vertically downwards, and the history of peaks and valleys resemble a pagoda roof, as depicted in Figure 4.11a and Figure 4.11b.

![Standard load-history](image1.png)  ![Load-history plot for rainflow counting](image2.png)  ![Rainflow cycle analysis via closed loop identification](image3.png)

**Figure 4.11.** Load-history plot for rainflow counting method (Lee, Y.L. and Tjhung, T. 2012; O'Connor et al. 2010).

The cycles are identified in a manner in which closed hysteresis loops are identified from the stress–strain response of a material subject to cyclic loading. Figure 4.11c shows the closed hysteresis loops identified from the corresponding strain-time history. In this example, ranges A-D, B-C, E-F, and GH, would represent cycles counted under rainflow counting technique (O’Connor et al. 2010).

The cycle counting is performed in terms of reversals. The counting starts with a raindrop from the highest peak or the lowest valley and ends in the same highest peak (or lowest valley). The rainflow runs down and continues unless either the magnitude of the following peak (or valley) is equal to or larger than the peak (or valley) from which it
initiated, or a previous rainflow is encountered (Stephens et al. 2001). Each reversal is counted as half-cycle.

The original rainflow algorithm requires the rearrangement of the load-history in such a way that it begins and ends with the maximum peak (or minimum valley). Therefore, the entire load-history must be known before start counting the cycles. Downing and Socie (1982) addressed this limitation by proposing a rainflow algorithm that counts a history of peaks and valleys as they occur. As a result, it is suitable for “on-board” data processing since the entire load history is not known until the end of the test (Downing and Socie, 1982). This updated rainflow algorithm was incorporated by the ASTM E1049 (2017) – “Standard Practices for Cycle Counting in Fatigue Analysis”.

The MATLAB rainflow function was applied in the stress-history obtained from the WIM data coupled with the FE model to count the stress range cycles. The MATLAB rainflow function follows the ASTM E1049 algorithm and it is presented in the Appendix E. From the rainflow outputs the stress-range histogram was built for each identified fatigue-prone detail.

4.4.1.2 Stress-range histogram and effective Stress Range

Stress-range histograms for each fatigue sensitive detail was developed based on the rainflow counting of stress-range cycles. Suitable truncation on stress-ranges should be considered to avoid low magnitude stress cycles that do not contribute to the cumulative fatigue damage. The truncation criteria used in this research is explained in section 4.4.1.3. The truncated histogram is used to calculate a constant amplitude effective stress range, \( S_{re} \), that causes the same cumulative fatigue damage produced by the entire variable
amplitude stress-range spectrum. Therefore, by means of $S_{re}$ the fatigue assessment may be developed based on a constant amplitude stress rather than the variable amplitude stress ranges. In this research, the variable amplitude fatigue loading is represented by the effective stress range calculated based on the RMC technique, e.g., by applying Equation 2.1 with $B = 3$ (see Chapter 2).

4.4.1.3 Stress-range truncation criteria

According to Kwon (2011), for all welded steel details, the applicable stress range cut-off threshold can range between 0.5 ksi as a lower bound and 33% of the Constant Amplitude Fatigue Limit (CAFL) as a maximum. Kwon (2011) noted that smaller values of $S_{re}$ are obtained when stress ranges below the predefined cut-off are included in the stress-range histogram. As a result, the assessment of the fatigue number of cycles associated with the respective S-N curve may be overestimated.

Contradicting Kwon (2011), the MBE 3rd Edition in Article 7.2.2.2 proposes a stress-range cut-off at 45% of the CAFL (CAFL divided by a factor of 2.2) for the effective stress range calculation. This procedure relies on the calibration of AASHTO LRFD Fatigue I and Fatigue II Limit States. The load factor of 1.75 for Fatigue I limit state (infinite fatigue life) intends to represent the stress value that has 1/10,000 probability of being exceeded, aiming to represent maximum stress range. On the other hand, the load factor of 0.80 for Fatigue II limit state (finite fatigue life) aimed to reflect a stress level that represents the effective stress range of the truck population (Strategic Highway Research Program 2, 2015). The ratio of AASHTO Fatigue I load factor to Fatigue II load factor results in 2.2. Therefore, as stated in MBE Commentary C.7.2.2.2, AASHTO LRFD
assumes that the maximum stress range is 2.2 times the effective stress range. It is recalled that the calibration of AASHTO Fatigue limit states were based on the equivalent moments obtained from one-year WIM data divided by the moment produced by the AASHTO Fatigue Design Truck, at midspan for simply supported bridges and at 0.4 of the span lengths for continuous bridges. The span lengths varied from 30 ft to 200 ft and 15 WIM sites were considered to represent the national truck population. Moreover, trucks below 20 kips were filtered out from the WIM data since they are considered to not produce fatigue damage (Fisher, 1977). Since the MBE proposes to divide the CAFL by 2.2 to calculate the effective stress range, it is concluded that this procedure assumes that the maximum stress range observed in the stress history is equal to the CAFL. It is observed that the factor of 2.2 comes from an already truncated stress-history since trucks below 20 kips were not considered in AASHTO fatigue limit states calibration. Furthermore, this procedure may not be realistic when the nominal stresses are analyzed locally at fatigue prone detail, specially for transverse members. Therefore, the value of 45% of CAFL may not be suitable to express the stress-range cut-off threshold.

According to Connor et al. (2004, 2005), the definition of the truncation level should be selected based on two criteria: 1) previous research reported in NCHRP Report 354 has demonstrated that stress range less than 25% of the CAFL has little effect on the cumulative fatigue damage; 2) as the number of random variable cycles of lower stress range levels are considered, the predicted cumulative damage provided by the calculated effective stress range becomes asymptotic to the applicable S-N curve. Therefore, as long as the cut off level selected is consistent with the slope of the fatigue resistance curve, considering additional stress cycles at lower truncation levels does not improve the damage
assessment and can therefore be ignored (Connor et al., 2004). In order to exemplify the latter criterium, the truncation level investigation developed in Connor et al. (2004) is extended herein. The stress history from a category C detail on Birmingham Bridge is considered (same example used in Connor et al., 2004).

(a) Effective Stress Range according to different stress range cut-off levels.  
(b) Cumulative Fatigue Damage according to different stress range cut-off levels

Figure 4.12. Example for the study of different stress range cut-off levels.

The effective stress range curve represented by the blue dots in Figure 4.12a is defined by the calculation of the effective stress range and the corresponding number of cycles for different truncation levels applied in the stress-history (the calculated values are presented in Table 4.1). The truncation at 6.75 ksi yields an effective stress range curve that is nearly asymptotic to the S-N curve. Likewise, when the effective stress ranges are plotted against the corresponding cumulative fatigue damages, as presented in Figure 4.12b, the same asymptotic behavior it is observed at the stress cut-off level of 6.75 ksi. It is concluded from Figure 4.12b that cut-off levels below 6.75 ksi will not contribute to the cumulative fatigue damage. Therefore, to obtain an accurate estimation of the remaining fatigue life in this example, the stress-range cut-off level should occur at 6.75 ksi.
Table 4.1 – Calculated Effective Stress Range using different stress range cut-off levels (Connor et al. 2004).

<table>
<thead>
<tr>
<th>Cut-off level (ksi)</th>
<th>Number of Cycles</th>
<th>Effective Stress Range (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75</td>
<td>575,867</td>
<td>3.3</td>
</tr>
<tr>
<td>2.75</td>
<td>117,869</td>
<td>5.5</td>
</tr>
<tr>
<td>4.75</td>
<td>37,842</td>
<td>7.6</td>
</tr>
<tr>
<td>6.75</td>
<td>15,112</td>
<td>9.6</td>
</tr>
<tr>
<td>8.75</td>
<td>6,547</td>
<td>11.5</td>
</tr>
<tr>
<td>10.75</td>
<td>2,938</td>
<td>13.3</td>
</tr>
<tr>
<td>12.75</td>
<td>1,284</td>
<td>15.1</td>
</tr>
<tr>
<td>14.75</td>
<td>509</td>
<td>17.0</td>
</tr>
<tr>
<td>16.75</td>
<td>191</td>
<td>19.3</td>
</tr>
<tr>
<td>18.75</td>
<td>85</td>
<td>21.3</td>
</tr>
<tr>
<td>20.75</td>
<td>45</td>
<td>22.6</td>
</tr>
<tr>
<td>22.75</td>
<td>22</td>
<td>23.9</td>
</tr>
<tr>
<td>24.75</td>
<td>6</td>
<td>25.1</td>
</tr>
<tr>
<td>25.75</td>
<td>2</td>
<td>25.7</td>
</tr>
</tbody>
</table>

The investigation of the stress-range cut-off levels for the fatigue details 1, 2, 3, 5A, 5B, 5C, 6A, 6B and 7 are presented in Figure 4.13 to Figure 4.21, respectively. The truncation levels for each case are summarized in Table 4.2. It is noted that the effective stress range calculated based on the truncated $S_r$ histogram will be used only on the assessments of details with finite fatigue life.

(a) $S_{re}$ for different truncation levels  
(b) Cumulative Fatigue Damage
Figure 4.13. Study of the stress range truncation levels for fatigue Detail 1.

Figure 4.14. Study of the stress range truncation levels for fatigue Detail 2.

Figure 4.15. Study of the stress range truncation levels for fatigue Detail 3.
Since there is no AASHTO classification for detail 5C, only the cumulative fatigue damage is analyzed in Figure 4.18. In order to quantify the cumulative fatigue damage, the fatigue detail constant “A” corresponding to AASTHO category E’ was used. It is noted that the asymptotic behavior for the cumulative fatigue damage vs. the effective stress range curve does not depend on the fatigue detail constant A. In other words, other values of A would provide the same values for the stress range cut-off.
Figure 4.18. Study of the stress range truncation levels for fatigue Detail 5C.

(a) $S_{re}$ for different truncation levels  
(b) Cumulative Fatigue Damage

Figure 4.19. Study of the stress range truncation levels for fatigue Detail 6A.
The maximum stress range on Detail 7 corresponding to the Fix-Pin model is 0.55 ksi. Therefore, no study for the stress cut-off level was performed for this case and the truncation was considered at 0.5 ksi. It is observed that the stress analysis performed for Detail 7 is located 17 inches above the lower longitudinal stiffener. This location was selected based on the NDT results that will be discussed in Chapter 5.
Table 4.2 – Truncation Levels Results

<table>
<thead>
<tr>
<th>Detail / Model</th>
<th>Pin-Roller Model Stress Range Cut-off (ksi)</th>
<th>Fix-Pin Model Stress Range Cut-off (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detail 1</td>
<td>1.75</td>
<td>1.25</td>
</tr>
<tr>
<td>Detail 2</td>
<td>1.75</td>
<td>1.25</td>
</tr>
<tr>
<td>Detail 3</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>Detail 5A</td>
<td>1.50</td>
<td>1.25</td>
</tr>
<tr>
<td>Detail 5B</td>
<td>1.50</td>
<td>1.25</td>
</tr>
<tr>
<td>Detail 5C</td>
<td>1.75</td>
<td>1.00</td>
</tr>
<tr>
<td>Detail 6A</td>
<td>2.00</td>
<td>1.25</td>
</tr>
<tr>
<td>Detail 6B</td>
<td>2.00</td>
<td>1.25</td>
</tr>
<tr>
<td>Detail 7</td>
<td>0.50</td>
<td>0.50</td>
</tr>
</tbody>
</table>

4.4.1.4 Stress-range histograms

As discussed in Chapter 3, although the bridge was designed to behave as a simply-supported structure the diagnostic load test and the FE model simulations show that the bridge behaves as a fixed-pinned structure due to frozen bearing issues. Since this condition is unreliable and it can be reverted at the occurrence of an extreme event, the stress range histograms are presented for both scenarios.

The truncation levels presented in Table 4.2 and the number of bins according to Equation 4.1 were considered.

\[ n_{bins} = 1 + 3.3 \times \log_{10} n \]  
*(Equation 4.1)*

The stress-range histograms for each case are presented in Figure 4.22 to Figure 4.28. The corresponding effective stress range and maximum stress range are also indicated.
Figure 4.22. Stress-range histogram for Detail 1.

Figure 4.23. Stress-range histogram for Detail 2.
Figure 4.24. Stress-range histogram for Detail 3.

Figure 4.25. Stress-range histogram for Detail 5A and 5B.
**Figure 4.26.** Stress-range histogram for Detail 5C.

**Figure 4.27.** Stress-range histogram for Detail 6A and 6B.
The stress ranges for Detail 7 considering the Fix-Pin model are too low. After the truncation of 0.5 ksi, only one cycle was observed.

4.4.2 Fatigue loading based on AASHTO Fatigue Design Truck

The AASHTO Fatigue Design Truck was also used to determine the fatigue loading at the investigated details. According to AASHTO Commentary, C3.6.1.4.1, the fatigue design truck should be positioned both longitudinally and transversely on the bridge deck, ignoring the striped traffic lane, to create the worst stress or deflection, as applicable. Therefore, the Fatigue Design Truck was simulated on the exit lane for the analysis performed on the fascia girder B8. For each fatigue-prone detail, the peak stress from the stress-history caused by the fatigue truck passage was multiplied by the Fatigue I load factor and Fatigue II load factor to obtain the maximum stress range and the effective stress range, respectively. The dynamic amplification of 1.15 was assumed. The stress-histories are presented in Figure 4.29.
Stress (ksi)

(a) Detail 1

(b) Detail 2

Stress (ksi)

(c) Detail 3

(d) Details 5A and 5B

Stress (ksi)

(e) Detail 5C

(f) Details 6A and 6B
Table 4.3. shows the comparison of fatigue loading obtained by AASHTO Fatigue Truck and site specific WIM data, for the Pin-Roller model. It is observed that the maximum stress ranges obtained from the site-specific WIM data versus the AASHTO Fatigue Design Truck is in average 17.6% larger, when Detail 3 is not considered. This difference is attributed to the high frequency of overweight trucks in the site, as mentioned in section 3.2.3 of this dissertation. Furthermore, the maximum error found for the effective stress range $S_{re}$ calculation was 9.7% for Detail 5C, which confirms the proximity among both methods. Detail 3 is the only transversal fatigue detail investigated in the bridge, which might explain the high errors found in Table 4.3.

**Table 4.3 – Fatigue loading based on AASHTO Fatigue Truck vs. WIM data**

<table>
<thead>
<tr>
<th>Details</th>
<th>$S_{max}$ (ksi)</th>
<th>$S_{re}$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AASHTO Fatigue Truck</td>
<td>WIM data</td>
</tr>
<tr>
<td>Detail 1</td>
<td>7.43</td>
<td>9.11</td>
</tr>
<tr>
<td>Detail 2</td>
<td>7.71</td>
<td>9.36</td>
</tr>
<tr>
<td>Detail 3</td>
<td>1.10</td>
<td>2.81</td>
</tr>
<tr>
<td>Detail 5A/5B</td>
<td>4.86</td>
<td>5.93</td>
</tr>
<tr>
<td>Detail 5C</td>
<td>4.95</td>
<td>5.75</td>
</tr>
</tbody>
</table>
In order to further validate the results from Table 4.3, the bias ratio for the maximum stress range and the effective stress range were compared among the national WIM data used for the calibration of AASHTO fatigue limit states I and II (SHRP-2 R19, 2015), and the site-specific WIM data used in the case study. For the case of maximum stress range, the bias ratio is the actual moments with 0.01% probability of exceedance divided by the AASHTO Fatigue Truck moment. Likewise, the bias ratio for the effective stress range is the effective moment range obtained from the moment history produced by the truck population, divided by the AASHTO Fatigue Truck moment. Figure 4.30 and Figure 4.31 shows the results for a simply-supported bridge where the moments were calculated at midspan. The mean bias ratio for the maximum stress range is 1.67 when considering the national WIM data and 2.23 for the site-specific WIM data. This result shows that the truck population at the studied bridge is comprised of heavier trucks than the national WIM data. Yet, the bias ratio for the effective stress range was found to be very similar among both data, e.g., 0.768 for the national data and 0.759 for the site-specific data. Therefore, the consideration of the AASHTO Fatigue Truck for the Fatigue II limit state represents well the real truck population at the studied bridge, as shown also in Table 4.3.
Figure 4.30. Comparison of $S_{\max}$ bias ratio for simply supported bridges at midspan.

(a) National WIM data (SHRP2–R19B, 2015).
(b) Site-specific WIM data of I287 bridge.

Figure 4.31. Comparison of $S_{re}$ bias ratio for simply supported bridges at midspan.

(a) National WIM data (SHRP2–R19B, 2015).
(b) Site-specific WIM data of I287 bridge.
### 4.5 Remaining fatigue life assessment based on MBE Approach

In order to investigate whether the detail has finite or infinite fatigue life, the maximum stress range at each detail was compared to the corresponding constant amplitude fatigue threshold $(\Delta F)_{TH}$. It was concluded that Details 1, 2, 3, 5A and 6B have infinite fatigue life, as presented in Table 4.4. Since there is no AASHTO category to classify detail 5C, no conclusions regarding its fatigue life can be made using the stress-life method.

<table>
<thead>
<tr>
<th>Detail</th>
<th>Model</th>
<th>Maximum Sr (ksi)</th>
<th>WIM</th>
<th>AASHTO Fatigue Truck</th>
<th>$(\Delta F)_{th}$ (Ksi)</th>
<th>Fatigue Life</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detail 1</td>
<td>Pin-Roller</td>
<td>9.11</td>
<td>7.43</td>
<td>16</td>
<td>infinite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fix-Pin</td>
<td>5.45</td>
<td>4.34</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Detail 2</td>
<td>Pin-Roller</td>
<td>9.36</td>
<td>7.71</td>
<td>12</td>
<td>infinite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fix-Pin</td>
<td>5.56</td>
<td>4.48</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Detail 3</td>
<td>Pin-Roller</td>
<td>2.81</td>
<td>1.10</td>
<td>16</td>
<td>infinite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fix-Pin</td>
<td>0.85</td>
<td>0.85</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Detail 5A</td>
<td>Pin-Roller</td>
<td>5.93</td>
<td>4.86</td>
<td>16</td>
<td>infinite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fix-Pin</td>
<td>3.33</td>
<td>2.68</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Detail 5B</td>
<td>Pin-Roller</td>
<td>5.93</td>
<td>4.86</td>
<td>4.5</td>
<td>finite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fix-Pin</td>
<td>3.33</td>
<td>2.68</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Detail 6A</td>
<td>Pin-Roller</td>
<td>8.15</td>
<td>7.06</td>
<td>16</td>
<td>infinite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fix-Pin</td>
<td>3.87</td>
<td>3.20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Detail 6B</td>
<td>Pin-Roller</td>
<td>8.15</td>
<td>5.81</td>
<td>2.6</td>
<td>finite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fix-Pin</td>
<td>3.87</td>
<td>3.20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Detail 7</td>
<td>Pin-Roller</td>
<td>1.97</td>
<td>1.82</td>
<td>10</td>
<td>infinite</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fix-Pin</td>
<td>0.70</td>
<td>0.56</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For the cases of finite fatigue life, the remaining life was calculated based on MBE section 7.2.5, as presented in Chapter 2.
4.5.1 Calculations for \((ADTT_{SL})_{PRESENT}\)

The \((ADTT_{SL})_{PRESENT}\) was considered based on AASHTO LRFD provisions which assumes that it is equal to 80% of the average daily truck traffic \((ADTT)\) for highways with more than 3 traffic lanes. The \(ADTT\) for the bridge studied in this research was obtained from the data published in Lou et al. (2016) that shows the measurements from 1993 to 2011 in close by location (see Figure 4.32 and Figure 4.33). In this case, it was found that \((ADTT_{SL})_{PRESENT} = 4000\). Alternatively, the \((ADTT_{SL})_{PRESENT}\) according to the site-specific WIM data was found to be \((ADTT_{SL})_{PRESENT} = 1000\), as shown in Table 4.5. Since there is an exit ramp after the bridge, the exit lane with the right lane were considered as a single lane for the \((ADTT_{SL})_{PRESENT}\) counting. In order to be conservative, it is assumed that \((ADTT_{SL})_0 = (ADTT_{SL})_{PRESENT}\). The truck traffic growth rate was estimated as 2% based on Lou et al. data. All these assumptions were considered to estimate the remaining fatigue life in years for the stress-life method and LEFM approach.

![Figure 4.32. ADTT counting for WIM site 287 (Lou, Nassif, Su and Truban, 2016).](image-url)
Figure 4.33. WIM site 287 and studied bridge location.

Table 4.5 – ADTT based on site-specific WIM data (Exit Lane and Right Lane)

<table>
<thead>
<tr>
<th>Month - Year</th>
<th># days of recorded data</th>
<th># of vehicles</th>
<th># of trucks</th>
<th>ADT</th>
<th>(ADTTSL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>October 2018</td>
<td>12</td>
<td>212,111</td>
<td>14,167</td>
<td>17,676</td>
<td>1,181</td>
</tr>
<tr>
<td>November 2018</td>
<td>30</td>
<td>427,244</td>
<td>27,834</td>
<td>14,241</td>
<td>928</td>
</tr>
<tr>
<td>December 2018</td>
<td>18</td>
<td>253,094</td>
<td>15,222</td>
<td>14,061</td>
<td>846</td>
</tr>
<tr>
<td>January 2019</td>
<td>26</td>
<td>368,697</td>
<td>22,226</td>
<td>14,181</td>
<td>855</td>
</tr>
<tr>
<td>February 2019</td>
<td>25</td>
<td>325,576</td>
<td>18,940</td>
<td>13,023</td>
<td>758</td>
</tr>
<tr>
<td>March 2019</td>
<td>21</td>
<td>302,925</td>
<td>18,578</td>
<td>14,425</td>
<td>885</td>
</tr>
<tr>
<td>April 2019</td>
<td>22</td>
<td>305,684</td>
<td>16,545</td>
<td>13,895</td>
<td>752</td>
</tr>
<tr>
<td>May 2019</td>
<td>30</td>
<td>462,639</td>
<td>24,325</td>
<td>15,421</td>
<td>811</td>
</tr>
<tr>
<td>June 2019</td>
<td>30</td>
<td>429,880</td>
<td>26,157</td>
<td>14,329</td>
<td>872</td>
</tr>
<tr>
<td>July 2019</td>
<td>31</td>
<td>425,684</td>
<td>28,285</td>
<td>13,732</td>
<td>912</td>
</tr>
<tr>
<td>August 2019</td>
<td>31</td>
<td>437,233</td>
<td>28,461</td>
<td>14,104</td>
<td>918</td>
</tr>
<tr>
<td>September 2019</td>
<td>30</td>
<td>436,759</td>
<td>33,265</td>
<td>14,559</td>
<td>1,109</td>
</tr>
<tr>
<td>Average:</td>
<td></td>
<td></td>
<td></td>
<td>14,471</td>
<td>902</td>
</tr>
<tr>
<td>Approximately:</td>
<td></td>
<td></td>
<td></td>
<td>15,000</td>
<td>1,000</td>
</tr>
</tbody>
</table>

According to AASHTO LRFD Commentary C3.6.1.4.2, research has shown that the physical limit for the average daily traffic (ADT) in a single lane is about to 20,000 vehicles under normal conditions. For the purpose of the calculation of the remaining fatigue life it is assumed that the (ADTTSL)limit is 4000.
4.5.2 Calculations of the remaining fatigue life

For the stress-life method, different levels of safety were analyzed according to the Minimum, Evaluation 1, Evaluation 2, and Mean life S-N curves. Moreover, two tiers of analysis were performed: 1) MBE / AASHTO model that is based on AASHTO LRFD provisions for the computation of \((ADT_{SL})_{PRESENT}\) and fatigue loading; 2) MBE / WIM model that is based on site-specific WIM data. Since the Fix-Pin condition presents an unreliable condition, the assessment was performed only considering the Pin-Roller model. The results are presented in Table 4.6.

<table>
<thead>
<tr>
<th>Detail</th>
<th>(Y_{REM}) (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MBE / AASHTO</td>
</tr>
<tr>
<td></td>
<td>Min</td>
</tr>
<tr>
<td>5B</td>
<td>24</td>
</tr>
<tr>
<td>5C</td>
<td>---</td>
</tr>
<tr>
<td>6B</td>
<td>-17</td>
</tr>
</tbody>
</table>

It was found that MBE / WIM always provided higher fatigue lives than the MBE / AASHTO. For instance, when Minimum Life Curve is used, Detail 5B has 24 years and 88 years of remaining fatigue life when MBE / AASHTO and MBE / WIM models are considered, respectively. Since the effective stress range calculated based on AASHTO Fatigue Truck and WIM data are very similar, the difference on the results is attributed to a more accurate definition of \((ADT_{SL})_{PRESENT}\) when site-specific WIM data is used.

4.6 Remaining fatigue life assessment based on proposed S-N curve

As discussed in Chapter 2, the extrapolation of the S-N curve below the CAFL with the same slope of the finite life region has been criticized to be conservative. The S-N
curves provided in the EN-1993-1-9, as well as in the International Institute of Welding (IIW), address this conservatism by adopting Haibach’s model where the slope of \((2m - 1)\) is taken below the fatigue limit (CAFL). Therefore, the Eurocode S-N curve slopes are set to \(m = -3\) in the finite life region and, \(m = -5\) below the fatigue limit. Further research has confirmed that this model is better suited to estimate fatigue life (Pereira Batista, 2016).

Based on the literature review provided in Chapter 2, the following study has the objective of re-assessing AASHTO fatigue experimental data that was used to build the current S-N curves. Firstly, the compiled database of various fatigue tests programs provided in SHRP 2 R19B Report was revised. Secondly, the S-N curves obtained from the database was compared to the current AASHTO S-N curves in order to validate the data. Lastly, following the methodology proposed by Lou and Nassif (2017), a piecewise regression was performed in fatigue test datapoints. Based on the results, new S-N curves for the detail category E and E’ are proposed.

4.6.1 S-N curves from the fatigue database

The fatigue test results from various programs were compiled in a database for the formulation of AASHTO S-N curves. This database was published in SHRP 2 R19B Report. Although 2515 datapoints were provided, the interpretation of the data is not straightforward since the type of the tested details and the references from where the data was obtained, were not informed. Therefore, the identification of each datapoint to correlate its results to each appropriate fatigue category detail was needed. The validity of the data identification was performed by comparing the S-N plots in the log-log scale for
each type of detail with test the S-N plots provided in NCHRP Report 286 and Keating and Fisher (1986b). As a result, out of 2515 datapoints, 2398 were identified, as shown in Figure 4.34, Figure 4.35 and Figure 4.36, for the Transverse Web Stiffeners, Flange Surface Attachments and Coverplated beams, respectively. All the other cases are presented in Appendix F.

![Figure 4.34. Fatigue datapoints for Transverse Web Stiffeners detail.](image)

![Figure 4.35. Fatigue datapoints for Flange Surface Attachments detail](image)
The least-square method was used to build the mean S-N curves from the information provided in the database. The design curve, or lower bound curve, was determined based on 95% lower one-sided confidence limit. Although NCHRP Report 299 mentions that the AASHTO design S-N curves were obtained based on two standard deviations from the mean values, Bowman et al. (2012) in NCHRP Report 721 verified that actually a 95% lower one-sided confidence limit was used. For fatigue resistance one-sided limit should be used in lieu of two-sided limit since fatigue survival is given on one side and failure on the opposite side. Therefore, to be consistent with the findings of NCHRP Report 721, a 95% lower confidence interval was employed. Moreover, the slope and the intercept of the S-N curves followed the procedure reported in NCHRP Reports 286 and 299, e.g., a slope of $m = -3$ and the intercept $A$ taken at $2 \times 10^6$ cycles.

For validation purposes, the S-N curves obtained from the database were compared to the actual AASHTO S-N curves, as shown in Figure 4.37. The parameters that define the AASHTO design S-N curves are the detail category constant “A” (provided in AASHTO LRFD Table 6.6.1.2.5-1) and the curve slope $m = -3$ (NCHRP Report 286).

Complementarily, the parameter that defines the mean AASHTO S-N curves is the...
resistance factor $R_R$ (provided in MBE 3rd Edition Table 7.2.5.1-1), that corresponds to the ratio of the mean life intercept to the design life intercept ratio.

Figure 4.37. Comparison of the Database S-N curves versus AASHTO S-N curves.
The proximity between the database S-N curves and the current AASHTO S-N curves is observed in all cases, especially for the mean curves. It is stressed that during the calibration of AASHTO Fatigue I and II limit states, the statistical parameters of the fatigue test data was needed. Yet, the authors of SHRP2 R19B Report claimed that “any number of regression lines could have been used to describe this relationship between the stress range and fatigue life”. Therefore, in order to better analyze the fatigue test data, the number of cycles and stress range were coupled in the form of an effective stress range for each test specimen, as indicated in Equation 4.2. The parameter $S_{fl}$ was named as fatigue damage parameter corresponding to the specimen $i$ and $N$ was the number of cycles applied to the specimen at the stress range $S_{rl}$.

$$S_{fl} = (N \times S_{rl}^3)^{1/3}$$  \hspace{1cm} (Equation 4.2)

Aiming to obtain the statistical parameters from each set of data (for each fatigue category detail), $S_{fl}$ was plotted in the normal probability paper. Hence, in order to provide additional validation for the S-N curves developed in this research, the fatigue damage parameters were plotted in the normal probability paper and compared to the plots reported in SHRP2, as presented in Figure 4.38. It is highlighted that the authors of SHRP2 filtered the data for a Standard Normal Variable (z-score) below than 1.0 in order to reflect the fatigue behavior of each category more accurately. Comparing the normal probability plots, it is observed that the curves matched well below the cutoff value. Furthermore, the following observations are made:

- As the category detail is decreasing, e.g., from A to E, the plot tends to shift to the left, resulting in smaller $S_f$ mean values.
- The curves for category D (Database S-N vs. SHRP2) are not in agreement for the z-scores higher than zero.

- The curves for category E' (Database S-N vs. SHRP2) do not match when the z-scores are in between -1 and 1.

- For the abovementioned two points, it is observed that the SHRP2 curves for details D and E' tend to merge to category E. As pointed out, it is expected that the curves shift to the left as the category decreases. Therefore, for the current S-N curves formulation some of the datapoints from category E could have been misclassified as category D or E'.

![Graph showing fatigue damage parameter comparison](image)

Figure 4.38. Fatigue damage parameter comparison (Database S-N curves vs. SHRP2).

Based on the different validation procedures and in the observations made above, it is concluded that the S-N curves developed based on the fatigue database are reliable and properly represent the stress-life resistance for each AASHTO category detail.
4.6.2 Proposed S-N curves based on Piecewise regression

A common assumption when developing S-N curves based on the least square method is that the standard deviation error is constant. According to ASTM E739, this hypothesis is reasonable for notched and joint specimens up to about $10^6$ cycles. However, the code alerts that other statistical investigation may be needed if the standard deviation appears not be constant.

In order to account for the different standard deviation in the regression model a piecewise regression was performed in the database, following the approach proposed by Lou and Nassif (2017) for riveted components. The piecewise regression is suitable for cases when the slope of the linear relationship may differ for different intervals of the independent variable (stress range variable for the case of fatigue data). Thus, the candidate intervals for the slope change must be defined by breaking points, also known as knot values. According to Mendenhall and Sincich (2020), in the absence of theoretical knot values, the breaking points can be defined by visual inspecting the scatterplot and locating the points at which the slope appears to change. Therefore, for the current analysis, the knot value is defined at the CAFL, since much discussion about the S-N curve slope below the fatigue limit has been reported.

The piecewise regression model considering the transformed variables to the logarithm base is presented in Equation 4.2, where $k_1$ is the knot value defined at 4.6 ksi.

$$E(y) = \beta_0 + \beta_1 x_1 + \beta_2 (x_1 - k_1) x_2 \quad \text{where} \quad x_2 = \begin{cases} 1 & \text{if } x_1 > k_1 \\ 0 & \text{if } x_1 \leq k_1 \end{cases}$$

Equation (4.2)

The transformed dependent variable is $y = \log (N)$ and the transformed dependent variable is $x = \log (S)$. The regression intercept and slopes are calculated according to the expressions presented in Table 4.7.
According to BS ISO 12107 (2003), at least six datapoints are necessary for estimating the S-N curve in the infinite fatigue life regime. In order to comply with this requirement, the new S-N curves bases on the piecewise regression were doable only for AASHTO detail categories E and E’. The proposed S-N curves are shown in Figure 4.39. It is noted that indeed the standard deviation for the dependent variable (number of cycles) is not constant for different stress range levels. The parameters for the S-N curves obtained for categories E and E’ are summarized in Table 4.8.

![Figure 4.39. Bi-linear S-N curves for detail categories E and E’](image)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$x_1 \leq k_1$</th>
<th>$x_1 &gt; k_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>intercept</td>
<td>$\beta_0$</td>
<td>$\beta_0 - k_1 \beta_2$</td>
</tr>
<tr>
<td>slope</td>
<td>$\beta_1$</td>
<td>$\beta_1 + \beta_2$</td>
</tr>
</tbody>
</table>

Table 4.8 – Parameters for the S-N curves based on Piecewise Regression.

<table>
<thead>
<tr>
<th>Category E</th>
<th>Parameter</th>
<th>Above CAFL</th>
<th>Below CAFL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Deviation</td>
<td>0.20</td>
<td>0.13</td>
<td></td>
</tr>
<tr>
<td>Intercept (mean)</td>
<td>1.12e+09 (ksi³)</td>
<td>5.46e+09 (ksi³)</td>
<td></td>
</tr>
<tr>
<td>Slope</td>
<td>-2.80</td>
<td>-3.84</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Category E’</th>
<th>Parameter</th>
<th>Above CAFL</th>
<th>Below CAFL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Deviation</td>
<td>0.25</td>
<td>0.34</td>
<td></td>
</tr>
<tr>
<td>Intercept (mean)</td>
<td>1.43e+09 (ksi³)</td>
<td>1.86e+09 (ksi³)</td>
<td></td>
</tr>
<tr>
<td>Slope</td>
<td>-3.18</td>
<td>-3.47</td>
<td></td>
</tr>
</tbody>
</table>
The slopes below the CAFL of -3.84 and -3.47 for detail categories E and E’, respectively, matched well with the analytical examination of S-N curves reported by Crudele and Yen (2006), which suggested a slope of -4.

4.6.3 Remaining fatigue life based on proposed S-N curve

Since the fatigue damage accumulation computed based on bilinear S-N curves differs from the conventional AASHTO S-N curve, the calculation of the effective stress range and then the remaining fatigue life should be updated accordingly. According to Kosteas (1999), the effective stress range for bilinear S-N curves, $S_{re}^*$, can be calculated as expressed in Equation 4.3.

$$
S_{re}^* = \left[ \frac{\sum(n_{a,i}S_{a,i}^{m1}) + (CAFL^{m1-m2}) \sum(n_{b,j}S_{b,j}^{m2})}{\sum n_{a,i} + \sum n_{b,j}} \right]^{1/m1}
$$

(Equation 4.3)

where:

$S_{a,i}$ is the stress range cycles above the CAFL

$S_{b,j}$ is the stress range cycles below the CAFL

$n_{a,i}$ is the number of cycles corresponding to stress range $S_{a,i}$

$n_{b,j}$ is the frequency of occurrence of stress range $S_{b,j}$

$m_1$ is the S-N curve slope above the CAFL (finite life region)

$m_2$ is the S-N curve slope below the CAFL

The corresponding estimation of the number of cycles is given in Equation 4.4 (Kwon et al. 2012 and Yen et al. 2013).

$$
N_{m1m2} = \begin{cases} 
A_{m1} \times S_{re,m1m2}^{-m1} & \text{if } S_{re,m1m2} \geq CAFL \\
A_{m1} \times CAFL^{m2-m1} \times S_{re,m1m2}^{-m2} & \text{if } S_{re,m1m2} \leq CAFL
\end{cases}
$$

Equation (4.4)
where:

\( A_{m1} \) is the S-N curve intercept for the region above the CAFL.

\( A_{m2} \) is the S-N curve intercept for the region below the CAFL.

The proposed methodology is applied for the remaining fatigue life assessment of Detail 5B and Detail 6B, which are classified as AASHTO detail category E and E’, respectively.

To be consistent with the current AASHTO S-N curves, the 95% lower bound curve was considered as presented in Figure 4.40 and Table 4.9. The lower bound curves were defined based on the standard deviation from the regression above the CAFL due to the larger sample of datapoints. Additionally, the curves have a common point at the CAFL.

![Figure 4.40. Lower bound bi-linear S-N curves for detail categories E and E’](image)

<table>
<thead>
<tr>
<th>Category</th>
<th>Parameter</th>
<th>Above CAFL</th>
<th>Below CAFL</th>
<th>AASHTO S-N</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>Intercept (lower)</td>
<td>4.64e+08 (ksi³)</td>
<td>2.21e+09 (ksi³)</td>
<td>1.07e+09 (ksi³)</td>
</tr>
<tr>
<td></td>
<td>Slope</td>
<td>-2.80</td>
<td>-3.84</td>
<td>-3.00</td>
</tr>
<tr>
<td>E'</td>
<td>Intercept (lower)</td>
<td>5.51e+08 (ksi³)</td>
<td>7.20e+08 (ksi³)</td>
<td>3.91e+08 (ksi³)</td>
</tr>
<tr>
<td></td>
<td>Slope</td>
<td>-3.18</td>
<td>-3.47</td>
<td>-3.00</td>
</tr>
</tbody>
</table>
Both, the effective stress range, $S_{re}$, and the number of available cycles, $N_{av}$, were recalculated according to proposed bilinear S-N curve. In order to compute the appropriate $S_{re}$, the stress range cutoff level was investigated for Detail 5B and Detail 6B, as shown in Figure 4.41 and Figure 4.42, respectively. The same methodology presented in section 4.4.1.3 was applied herein. It was found that there is no significant fatigue damage contribution for stress ranges below 2 ksi, for both cases.

Figure 4.41. Cutoff level for Detail 5B: AASHTO and bilinear S-N curves.

(a) $S_{re}$ for different truncation levels  (b) Cumulative Fatigue Damage

Figure 4.42. Cutoff level for Detail 6B: AASHTO and bilinear S-N curves.

(a) $S_{re}$ for different truncation levels  (b) Cumulative Fatigue Damage
The remaining life is compared to the previous assessment using the MBE Minimum Life Curve with fatigue loading based on WIM data, as shown in Table 4.10.

Table 4.10 – Fatigue Life of Detail 5B: Conventional vs. Proposed Bilinear S-N curve.

<table>
<thead>
<tr>
<th>Detail</th>
<th>Parameter</th>
<th>MBE / WIM</th>
<th>Proposed Bilinear S-N curve / WIM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detail 5B</td>
<td>$N_{av}$</td>
<td>103,058,033</td>
<td>124,312,777</td>
</tr>
<tr>
<td></td>
<td>$(\Delta f)_{eff}$ (ksi)</td>
<td>2.21 ksi</td>
<td>2.12 ksi</td>
</tr>
<tr>
<td></td>
<td>$Y_{REM}$ (years)</td>
<td>88 years</td>
<td>103 years</td>
</tr>
<tr>
<td>Detail 6B</td>
<td>$N_{av}$</td>
<td>13,595,041</td>
<td>15,523,572</td>
</tr>
<tr>
<td></td>
<td>$(\Delta f)_{eff}$ (ksi)</td>
<td>3.06 ksi</td>
<td>3.07 ksi</td>
</tr>
<tr>
<td></td>
<td>$Y_{REM}$ (years)</td>
<td>34 years</td>
<td>36 years</td>
</tr>
</tbody>
</table>

For Detail 5B, the remaining fatigue life provided by the proposed bilinear curve is 17% higher than the MBE / WIM approach. Yen et al. (2013) found 50% higher fatigue lives based on bilinear S-N curve for detail category E, with slopes of $m_1 = -3$ and $m_2 = -4$. Since the $S_{re}^*$ for Detail 5B is less than the CAFL, the fatigue life based on the bilinear S-N curve is higher than the conventional curve due to the less steep slope below the CAFL and its higher intercept (bilinear curve intercept = 2.21e+09, whereas AASHTO S-N curve intercept = 1.07e+09, as shown in Table 4.9).

For Detail 6B, the remaining fatigue life provided by the proposed bilinear curve is 5.88% higher than the MBE / WIM approach. Since the $S_{re}^*$ for Detail 6B is larger than the CAFL, the slope and intercept of the proposed bilinear S-N curve used to calculate the number of cycles to failure are very close to the slope and intercept from the current AASHTO S-N, as shown in Table 4.9. Therefore, the remaining fatigue lives among both curves were very close.
CHAPTER 5

FATIGUE ASSESSMENT BASED ON EXPLICIT CRACK INFORMATION

This chapter focuses on the remaining fatigue life assessment based on fatigue resistance models that include crack information. It is highlighted that fatigue life is evaluated for the component, e.g., the critical fatigue details, rather than the whole system (the whole bridge). Firstly, the LEFM and UniGrow models are applied considering a range of initial crack sizes. Then, as a second tier of analysis, the crack-based models are coupled with the Phased Array Ultrasonic Testing (PAUT), which has enhanced capabilities of detecting flaws in comparison with other Non-Destructive Testing methods.

The LEFM models were developed according to the methodology presented in Chapter 2. The corresponding stress intensity factor for each detail was determined and the crack propagation was investigated based on Paris law. The crack growth, $C$, and the empirical material constant, $m$, were assumed based on the lower bound limit of ferritic-pearlritic steel from Barsom’s experimental data, e.g., $C = 3.6 \times 10^{-10}$ and $m = 3$. The applicability of LEFM was verified according to the criteria presented in section 2.3.4.
In this dissertation, Equation 2.40 proposed by Barsom et al. (1999) is used to obtain the stress intensity factor threshold, $\Delta K_{th}$. The calculation of the stress ratio $R$ followed the recommendation of BS7910 (2005), i.e., the effective value of stress ratio was calculated by superimposing the applied stress with the residual stress. Therefore, the minimum stress $\sigma_{min}$ is taken as the dead load stress combined with the residual stress, whereas the maximum stress $\sigma_{max}$ is the $\sigma_{min}$ plus the live load stresses.

A stress ratio of approximately $R = 0.9$ was found for the fatigue Details 5B and 5C. This result reflects a high minimum nominal stress due to the presence of the long overhang above girder B8 as well as the assumption of the residual stress equal to the yield strength of the material, as suggested elsewhere (BS7910). As a result, the threshold stress intensity factor for the LEFM models was found as $\Delta K_{th} = 1.77 \, MPa\sqrt{m} \, (1.61 \, ksi\sqrt{in})$. Thus, when common values of initial crack sizes are employed in LEFM model, the minimum $a_i$ value complies with the condition $\Delta K_i \geq \Delta K_{th}$. Further, it is noted that the BS7910 standard recommends a $\Delta K_{th} = 2.00 \, MPa\sqrt{m} \, (1.82 \, ksi\sqrt{in})$ for the assessment of welded joints in the air environment. On the other hand, Schroeder (2018) found that the same code allows a $\Delta K_{th}$ between $1.65 \, MPa\sqrt{m} \, (1.50 \, ksi\sqrt{in})$ to $2.19 \, MPa\sqrt{m} \, (2.00 \, ksi\sqrt{in})$ to determine the critical sizes of slag inclusions in welded joints. Therefore, the near threshold region can be characterized as $1.65 \, MPa\sqrt{m} \, (1.50 \, ksi\sqrt{in}) \leq \Delta K_i \leq 2.00 \, MPa\sqrt{m} \, (1.82 \, ksi\sqrt{in})$ where the UniGrow model would be more appropriate to predict crack growth based on the predominant elastic behavior at the crack tip.
5.1 Remaining fatigue life assessment based on LEFM approach

For all fatigue details, the correction factor due to stress concentration effects, $F_g$, was determined according to Equation 2.27. The inputs for the stress concentration factor at each element are extracted from specific two-dimensional FE model according to the detail geometry, similarly as Zettlemoyer et al. (1977). According to Pang (1991), when 2D models are used rather than 3D models, comparative calculations show that there could be an error up to 9% in terms of SIF, but on the conservative side. Therefore, in order to extract the stress distribution at the notch, 2D models are used in this research.

All fatigue analysis performed in this chapter considered the fatigue loading determined based on site-specific WIM data, as detailed in Chapter 4. Moreover, for the LEFM models, the critical crack size that determined the failure criterion was assumed as the plate thickness or the crack size corresponding to the material fracture toughness, $K_c$, of 32.9 MPa$\sqrt{\text{m}}$ (30 ksi$\sqrt{\text{in}}$).

5.1.1 LEFM model for Detail 5B

The analytical stress intensity factor for the weld termination of the longitudinal stiffener to the web connection is considered based on the semi-elliptical surface crack case. According to the fatigue tests performed in the University of Alberta and in the NCHRP Project 12-15 (3), typical fatigue cracks initiated at the fillet weld toe for the web attachment detail (Fisher et al. 1980, Comeau et al. 1979). The cracks are characterized as semi-elliptical surface cracks at the weld toe, as shown in Figure 5.1.
Fisher et al. (1980) assessed the crack growth for this type of detail by considering the SIF Equation 2.9, where the factors $F_s$ and $F_e$ were expressed as Equations 2.10 and 2.12. The factor $F_w$ was taken as 1.0 since the initial crack of 0.02 inches was too small in comparison to the plate thickness. The factor $F_g$ was obtained according to the detail geometry and the stress concentration factors found by FE analysis.

In this thesis, the crack propagation of Detail 5B is investigated based on SIF Equation 2.9 (Fisher method) as well as Equations 2.14 and 2.15 (Newman and Raju SIF expressions). For both cases, a two-dimensional FE model was developed to obtain the stress concentration factors along the expected crack path, so the factor $F_g$ could be calculated by Equation 2.27. In order to investigate the effect of the weld toe radius, $F_g$ was determined for a weld toe radius of $\rho = 0$ and $\rho = 0.008$ inches. It is noted that these values of $\rho$ are considered as sharp corners (Hobbacher, 2016).
5.1.1.1 Stress concentration factor for Detail 5B via FE model

The stress concentration factor for Detail 5B was investigated through FE modeling. The geometry of the weld toe radius followed the scheme showed in Schork (2018), presented herein in Figure 5.2. Since the stress concentration factor obtained in FE model is mesh dependent (Mauk 1999), a sensibility analysis was developed for the mesh convergence.

![Figure 5.2. Position of the weld toe radius \( \rho \) (Schork, 2018).](image)

The FE model was developed in similar manner as Norris et al (1981), i.e., the two-dimensional model consists of the web thickness, the longitudinal stiffener plate, and the weld termination. Figure 5.3 shows the model geometry, boundary conditions and a pressure load of 1 ksi applied at the end of the web. The triangle shell elements CPS6M were used from Abaqus library, and the analysis was performed under plane stress condition.
The mesh convergence for the stress concentration factor is presented in Figure 5.4 for the case of \( \rho = 0 \) and Figure 5.5 for the case of \( \rho = 0.008 \) inches. The \( K_t \) values were plotted against the dimensionless parameter \( L/t_w \) where \( L \) represents the crack path along the web thickness (as discussed in section 2.3.1.5) and \( t_w \) is the web thickness.

Figure 5.4. Detail 5B mesh convergence for the stress concentration factor considering weld toe radius \( \rho = 0 \).

(a) \( K_t \) plot along the web thickness  
(b) Meshes used in the FE model
The $K_t$ values are indeed mesh dependent and results are summarized in Table 5.1.

The error for Mesh 01 was calculated based on the ratio of the maximum stress concentration factor ($K_{tm}$) obtained from Mesh 01 over Mesh 02. The error for the other meshes followed this pattern.

Table 5.1 – Stress Concentration Factor investigation for Detail 5B

<table>
<thead>
<tr>
<th>Weld toe radius $\rho = 0$</th>
<th>Weld toe radius $\rho = 0.008$ in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mesh</td>
<td>Element Size on the expected crack path</td>
</tr>
<tr>
<td>Mesh 01</td>
<td>0.00250 in</td>
</tr>
<tr>
<td>Mesh 02</td>
<td>0.00100 in</td>
</tr>
<tr>
<td>Mesh 03</td>
<td>0.00042 in</td>
</tr>
<tr>
<td>Mesh 04</td>
<td>0.00031 in</td>
</tr>
</tbody>
</table>

Given that the mesh is sufficiently refined, for the case of $\rho = 0$, the maximum stress concentration factor converged at $k_{tm} = 11.60$. For the case of $\rho = 0.008$ in, $K_{tm}$
converged at 5.88 considering Mesh 03 where the elements along the expected crack path were 0.00063 inches long. It is noted that $K_{tm}$ for $\rho = 0.008$ inches is 49.3% smaller than the case with $\rho = 0$. These results are in reasonable agreement with the findings of Molski et al. (2020).

For Detail 5B, the weld leg size and the weld throat thickness are 0.25 inches and 0.1768 inches, respectively. Thus, the parametric weld toe radius $X$ corresponding to $\rho = 0.008$ inches is $X = 0.043$ and the parametric plate thickness is $Y = 0.2$. The stress concentration factor extracted from the plot of Figure 2.8b, for $X = 0.043$ and be $Y = 0.2$, is approximately 4.40. Therefore, it is concluded that there is a significant difference on $k_{tm}$ values for the Detail 5B when the minimum weld toe radius of $\rho = 0.008$ inches is considered. Moreover, the $k_{tm}$ values found in this study are slightly different from the ones reported in NCHRP Report 227 and Norris et al. (1981). Therefore, it is concluded that both $k_{tm}$ found from $\rho = 0$ and $\rho = 0.008$ inches models are needed to compute the stress gradient correction factor, $F_g$.

5.1.1.2 Stress concentration correction factor $F_g$ for Detail 5B

The stress concentration correction factor, $F_g$, was computed according to Green’s function and the stress distribution profile presented in Figure 5.4 and Figure 5.5. The decay of $F_g$ values when compared to $K_t$ followed similar trend observed in Zettlemoyer (1976) and Zettlemoyer et al. (1977), as shown in Figure 5.6a and Figure 5.6b for the cases of $\rho = 0$ and $\rho = 0.008$ inches, respectively.
(a) Model with $\rho = 0$

(b) Model with $\rho = 0.008$ in

Figure 5.6. Detail 5B: decay of $F_g$ versus $K_t$.

According to Zettlemoyer et al. (1977), the decay of $F_g$ is less rapid than $K_t$ decay. Both curves start at $K_{tm}$ and their separation is dependent on the detail geometry. An equation for $F_g$ as a function of the maximum stress concentration factor, $K_{tm}$, and the dimensionless parameter crack size over the web plate thickness ($a/t_w$) was developed based on the plot of $F_g/K_{tm}$ versus $a/t_w$, as shown in Figure 5.7.

The Curve Fitting Toolbox in MATLAB was used to obtain the expressions of $F_g$ based on the nonlinear least square method was employed. The resultant $F_g$ equations for
the Detail 5B when \( \rho = 0 \) and \( \rho = 0.008 \) inches are provided in Equations 5.1 and 5.2, respectively.

\[
F_{g,\rho=0} = K_{tm}[0.0461(a/t_w)^{-0.3999} + 0.0449] \quad \text{(Equation 5.1)}
\]

\[
F_{g,\rho=0.008} = K_{tm}[0.0989(a/t_w)^{-0.3635} + 0.0752] \quad \text{(Equation 5.2)}
\]

These equations are used for the calculation of the remaining fatigue life of Detail 5B using the LEFM approach.

5.1.1.3 Crack aspect ratio for Detail 5B

The crack growth analysis for web attachment detail presented in NCHRP Report 227 considered the evolution of the crack aspect ratio based on the empirical curves given in Figure 5.8 (units are in mm). The work published by Fisher (1984), also employed similar relationship for the crack aspect ratio in regard to this detail.

![Figure 5.8. Crack aspect ratio variation (NCHRP Report 227).](image-url)
The same criteria used in NCHRP Repot 227 is used in this study for the assessment of Detail 5B. The expressions given in Figure 5.8 were transformed to inches unit in Equations 5.3 and 5.4 (Fisher 1984).

\[ c = 0.132 + 1.29a \quad \text{for} \ a \leq 0.15 \text{ in} \]  
\[ c = 3.247a^{1.241} \quad \text{for} \ a > 0.15 \text{ in} \]  

5.1.1.4 Detail 5B results

The SIF range curve against the crack depth \( a \) is presented in Figure 5.9. The minimum crack size corresponds to the smallest initial crack size that complies with the requirements for LEFM application. It is observed that Fisher et al. (1980) expression tends to be more conservative after the crack depth is 0.3 inches (60% of the plate thickness).

![Figure 5.9. Detail 5B – stress intensity factor range.](image)

As mentioned before, the crack aspect ratio followed the curve proposed in NCHRP Report 227 that was formulated based on experimental data for this type of detail. The crack aspect ratio and the crack evolution for Detail 5B are shown in Figure 5.10.
The total number of cycles for the crack to propagate to its critical size was calculated based on common initial crack sizes adopted in the literature as well as the condition $\Delta K_I \geq \Delta K_{th}$ see (Table 2.1). The corresponding remaining fatigue life of Detail B was also computed in years according to Equation 2.4. The term $(N_{aw} - N_1)$, which corresponds to the remaining number of fatigue cycles, was substituted to the total number of cycles obtained from the LFEM analyses. Figure 5.11 shows the results.

(a) Total number of cycles $N$  
(b) Remaining fatigue life (years)

Figure 5.11. Detail 5B – remaining fatigue life based on LEFM approach.
It is observed that the range of initial crack sizes highly affects the results. For Detail 5B, the initial crack size assumption leads to a variation on the number of cycles up to 76% as seen in Figure 5.11a. As a result, the remaining fatigue life varies between 46 years to 93 years when the weld toe radius is taken as $\rho = 0$ (Fisher model). The weld toe radius caused a variation up to 2% in the fatigue life of Detail 5B and $\rho = 0.008$ inches always resulted in more fatigue life than $\rho = 0$, as expected. It is noted, though, that $\rho = 0$ and $\rho = 0.008$ inches are considered sharp weld toe radius, meaning that larger radius could produce longer fatigue lives. Furthermore, it was observed that Fisher and Newman & Raju (N & R) models presented similar results, which are summarized in Table 5.2.

<table>
<thead>
<tr>
<th>Fisher model</th>
<th>$a_i$ (inches)</th>
<th>Number of cycles</th>
<th>$Y_{REM}$ (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho = 0$</td>
<td>0.030</td>
<td>9.0502e+07</td>
<td>93</td>
</tr>
<tr>
<td>$\rho = 0$</td>
<td>0.200</td>
<td>2.1588e+07</td>
<td>46 (Min)</td>
</tr>
<tr>
<td>$\rho = 0.008$ (in.)</td>
<td>0.033</td>
<td>9.1270e+07</td>
<td>94</td>
</tr>
<tr>
<td>$\rho = 0.008$ (in.)</td>
<td>0.200</td>
<td>2.3643e+07</td>
<td>48</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Newman &amp; Raju model</th>
<th>$a_i$ (inches)</th>
<th>Number of cycles</th>
<th>$Y_{REM}$ (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho = 0$</td>
<td>0.043</td>
<td>9.1907e+07</td>
<td>94</td>
</tr>
<tr>
<td>$\rho = 0$</td>
<td>0.200</td>
<td>2.5794e+07</td>
<td>49</td>
</tr>
<tr>
<td>$\rho = 0.008$ (in.)</td>
<td>0.044</td>
<td>9.4115e+07</td>
<td>96 (Max)</td>
</tr>
<tr>
<td>$\rho = 0.008$ (in.)</td>
<td>0.200</td>
<td>2.8250e+07</td>
<td>51</td>
</tr>
</tbody>
</table>

5.1.2 LEFM model for Detail 5C

As reported in the literature the severity of the longitudinal stiffener splice (Detail 5C) depends on the quality of the butt weld used to connect the plates (NCHRP Report 227). Past events such as the one in Quinnipiac River Bridge on 1973 and in Delaware’s I-95 Bridge in 2003, indicate that fatigue cracking and even brittle fracture can be originated due to the lack of fusion of the transverse weld connecting the horizontal stiffeners (Fisher et al. 1980b, Fisher et al. 1980c, Hausammann 1980, Fisher 1984 and Chajes et al. 2005).
A survey reported by Lindberg and Schultz (2007) showed that states like Indiana and Montana have had issues with cracks that initiated at the welds connecting longitudinal stiffeners. In this survey, INDOT mentioned that three bridges had multiple cracks in horizontal stiffeners and in all cases the cracks developed in poor quality splice welds of the stiffener plates, and not in the welds to the girder webs. According to Hausammann (1980), the unfused weld is treated as a through-thickness crack that grows until the fracture, e.g., until the horizontal stiffener segments are completely separated. Fisher (1984) assessed the crack propagation of the longitudinal stiffener weld splice considering that the lack-of-fusion configured as symmetric and eccentric continuous through-thickness crack, as shown in Figure 5.12. In order to envelope all possible cases, in this study the edge crack configuration is investigated in addition to the symmetric and eccentric through thickness crack.

The analytical stress intensity factor used in the assessment followed Equation 5.5a for the through-thickness crack case and 5.5b for the edge crack case. It is noted that different back-surface correction factors were employed depending on the crack position.

![Figure 5.12. Continuous crack configuration in the longitudinal stiffener weld splice.](image)

\[ K_I = F_w \sigma \sqrt{\pi a} \]  
\[ (Equation \ 5.5a) \]

\[ K_I = F_w F_s F_g \sigma \sqrt{\pi a} \]  
\[ (Equation \ 5.5b) \]
For the through-thickness crack configuration, the back surface correction factor $F_w$ is calculated for the symmetric and eccentric cases as Equations 5.6 and 5.7, respectively (Fisher, 1984), where $a$ is half of the crack width and $t_s$ is the longitudinal stiffener thickness. Equation 5.6 used by Fisher (1984) is exactly the same equation reported by Tada et al. (1973) for the case of center cracked specimens. For the case of the eccentric crack, the correction factor $F_w$ assumes a ratio $2e/t_s = 0.7$, where $e$ is the eccentricity (as depicted in Figure 5.12).

$$F_{w,\text{sym}} = \left[ 1 - 0.025 \left( \frac{2a}{t_s} \right)^2 + 0.06 \left( \frac{2a}{t_s} \right)^4 \right] \sqrt{\sec \left( \frac{\pi a}{t_s} \right)}$$  \hspace{1cm} \text{(Equation 5.6)}

$$F_{w,\text{ecc}} = \left[ 1 + 0.86 \left( \frac{a}{t_s} \right) - 1.478 \left( \frac{2a}{t_s} \right)^2 + 2.307 \left( \frac{2a}{t_s} \right)^3 \right]$$  \hspace{1cm} \text{(Equation 5.7)}

The numerical SIF for the symmetric condition is evaluated along the width of the stiffener, based on the J-integral technique presented in Chapter 2. A 3D FE model was developed with hexahedrons element type C3D8R from Abaqus library. The longitudinal stiffener and the web plates were modeled and the symmetry in horizontal plane was considered. Figure 5.13 shows the model boundary conditions, the pressure load of 1 ksi and the mesh used to simulate the cracked component. The numerical stress intensity factors at the mid-width and at the free end of the plate matched with the analytical solution, as shown in Figure 5.14. At the plate end connected to the girder web, the SIF decreases considerably. Thus, in order to be conservative the assessment for the crack propagation is evaluated based on the SIF corresponding to the mid-width and free end values, which are represented by the analytical solution given in Equation 5.6.
Figure 5.13. FE model developed to extract the numerical SIF for Detail 5C.

(a) SIF along the plate width.  
(b) SIF at the end plate connected to web

(c) SIF at mid-width of the plate  
(d) SIF at the free end of the plate

Figure 5.14. Numerical SIF for detail 5C.
For the edge crack configuration, the back surface correction factor is considered as Equation 5.5a and the surface correction factor is assumed as $F_s = 1.12$ since bending is restrained by the web. The stress gradient correction factor $F_g$ needs to be considered due to the stress raiser caused by the weld reinforcement left in place, as depicted in Figure 5.15.

![Figure 5.15. Weld reinforcement in the longitudinal stiffener splice butt weld.](image)

As for Detail 5B, the non-linear stress concentration peak was studied based on a 2D FE model with triangular shell element type CPS6M from Abaqus library under plane stress condition. Various meshes were studied to extract the maximum stress concentration factor as well as the normal stress distribution from the weld toe throughout the plate thickness, as shown in Figure 5.16. The $F_g$ expression was obtained based on the Green’s function method. The decay of the stress concentration and $F_g$ are presented in Figure 5.17.
The Curve Fitting Toolbox in MATLAB was used to obtain the expressions of $F_g$ based on nonlinear least square method. The $F_g$ expression for Detail 5C is presented in Equations 5.8, where the maximum stress concentration factor is $K_{tm} = 7.59$.

$$F_{g,Detail5C} = K_{tm}[0.01458(a/t_s)^{-0.5153} + 0.1091]$$  \hspace{1cm} (Equation 5.8)

The SIF range curve against the crack depth $a$ is presented in Figure 5.18. For each case, the minimum crack size corresponds to the smallest initial crack size that complies with the requirements for LEFM application.
Figure 5.18. Detail 5C – stress intensity factor range.

The total number of cycles and the corresponding remaining fatigue life in years are presented in Figure 5.19 and Table 5.3.

Table 5.3 –Detail 5C: Remaining Fatigue Life results based on LEFM approach.

<table>
<thead>
<tr>
<th>Model</th>
<th>Initial crack size</th>
<th>Total Number of Cycles N</th>
<th>$Y_{REM}$ (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Symmetric</td>
<td>$a_i = 0.106$ in</td>
<td>$2.7523e+07$</td>
<td>45</td>
</tr>
<tr>
<td>Crack</td>
<td>$a_i = 0.200$ in</td>
<td>$1.1334e+06$</td>
<td><strong>32</strong> (minimum)</td>
</tr>
<tr>
<td>Eccentric</td>
<td>$a_i = 0.110$ in</td>
<td>$3.0816e+07$</td>
<td>49</td>
</tr>
<tr>
<td>Crack</td>
<td>$a_i = 0.200$ in</td>
<td>$2.5549e+06$</td>
<td>33</td>
</tr>
<tr>
<td>Edge</td>
<td>$a_i = 0.071$ in</td>
<td>$7.4543e+07$</td>
<td><strong>82</strong> (maximum)</td>
</tr>
<tr>
<td>Crack</td>
<td>$a_i = 0.200$ in</td>
<td>$2.0552e+07$</td>
<td>46</td>
</tr>
</tbody>
</table>
For Detail 5C, the assumption of through-thickness crack and edge crack resulted in large difference of remaining fatigue life. For the case of through-thickness crack, the minimum $a_t$ that will be subjected to propagation is $a_t = 0.106$ inches ($2a_t = 0.212$ inches). It is stressed that this value corresponds to 42.4% of the plate thickness. On the other hand, the minimum $a_t$ for the case of edge crack is $a_t = 0.071$ inches, which corresponds to 14.2% of the stiffener thickness. As a result, the remaining fatigue lives varied from 32 to 49 years when through-thickness crack is considered and from 46 and 82 years when the edge crack is considered.

5.1.3 LEFM model for Detail 6B

According to the bridge design plans, Detail 6B has a partial joint penetration (PJP) butt weld (see Figure 4.4). The unwelded area (0.5 inches x flange width) is treated like an initial crack as shown in Figure 5.20. The stress intensity factor is obtained numerically based on the $J$-integral method as discussed in Chapter 2. A 2D FE model was developed with shell element type CPS4R from Abaqus library. The boundary conditions, and the mesh used to calculate the SIF based on the $J$-integral values is presented in Figure 5.20.
Figure 5.20. FE model developed to extract the numerical SIF for Detail 6B.

The SIF range curve versus the crack depth $a$ and the corresponding total number of cycles are presented in Figure 5.21. As a result, the remaining fatigue life calculated through Equation 2.4 was found to be – 12 years.

![Figure 5.21. Detail 6B – stress intensity factor range and total number of cycles N.](image)

It is concluded that the hypothetical classification of category E’ for detail 6B does not hold true since LEFM resulted in less remaining fatigue life than the stress-life method. This agrees to the fact that V-groove welds with partial joint penetration were prohibited in AASHTO Standard Specification 14$^{th}$ Edition (see Figure 4.5) meaning that this type of detail would be worse than a category E’. Yet, PAUT inspection is recommended to confirm whether this welded joint is indeed a partial joint penetration (PJP).
5.1.3 LEFM model for Detail 7

Detail 7 is defined by the double-sided X-groove weld that connects web plates. As discussed in section 4.2, weld reinforcement was left in place which makes the weld toe as a potential crack initiation point (Hobbacher, 2016). Thus, for the LEFM assessment a semi-elliptical surface crack is considered. The crack model is presented in Figure 5.22.

(a) 3D view of the surface crack                              (b) 2D view of the surface crack

Figure 5.22. Semi-elliptical surface crack in the web splice detail.

The stress intensity factor for Detail 7 is evaluated according to Equation 2.9 (Fisher method) and Equations 2.14 and 2.15 (Raju and Newman method). Since Equation 2.9 accounts for the propagation throughout the plate thickness, a constant aspect ratio of 0.5 is assumed. On the other hand, when Newman and Raju equations are employed, the crack growth on both direction $a$ and $c$ are calculated and the crack aspect ratio is computed accordingly to Wu (1985) and Lu (1995), as discussed in section 2.3.2.1. The $F_g$ expression obtained for Detail 5C is also applicable for Detail 7 since both cases deal with the stress concentration of transversally loaded butt welds. Therefore, Equation 5.8 was applied to account for $F_g$. 

The semi-elliptical crack at the web splice is considered at 17 inches above the lower longitudinal stiffener. This location was selected based on the NDT results that are discussed in Section 5.2. For the selected range of initial crack sizes, i.e., $a_i = 0.006\,\text{in}$ to $a_i = 0.2\,\text{in}$, all of the SIF range were below the stress intensity factor threshold $\Delta K_{th} = 1.61\,\text{ksi}\sqrt{\text{in}}$, as shown in Figure 5.23. Therefore, it is concluded that there is no crack propagation for this case and Detail 7 has infinite fatigue life. This result matches with the assessment performed based on the stress-life method (Chapter 4).

![Figure 5.23. Detail 7 – stress intensity factor range.](image)

### 5.2 Remaining fatigue life assessment based on LEFM with PAUT

In this research, the PAUT technology was not employed to update the fatigue assessment based on the measurements of new discontinuities within the welded joints. Instead, it was employed to provide a more assertive values for the initial crack size, as recommended by Hobbacher (2011). Wang et al. (2009) adopted similar procedure for the fatigue assessment of existing steel bridges but using Ultrasonic Testing (UT). Moreover,
previous research on the NDT reliability have presented numerical examples where the initial crack size had to be assumed (Zhang and Mahadevan, 2001) before the updated fatigue assessment using NDT measurements. Thus, the results of the PAUT inspection were inputted in the LEFM and UniGrow models as initial crack sizes.

The recommended practices for executing the PAUT on the studied bridge were followed. AASHTO/AWS D1.5 Bridge Welding Code requires the calibration of PAUT based on the comparison of the shear wave velocity among the test object (girder weldment) and a IIW-type reference block, as shown in Figure 5.24. The ultrasound shear wave velocity is defined as the sound velocity through the thickness of the test object that is used to predict the refraction angles of the sound entering the steel. Differences within \( \pm 1\% \) of the shear wave velocity would result in negligible error in the sound amplitude and incidence angle. In order to guarantee the quality of results, the calibration block should be made of a material that is acoustic equivalent to the test object material (Connor et al., 2019).

![IIW-type reference block made of A1018 steel](curtis-test.com)

In this research the IIW-type reference block made of 1018 steel was used for the PAUT calibration. A small difference in shear wave speed between the calibration block and test object was obtained.
The American Society of Nondestructive Testing (ASNT) has a certification program to enhance the capabilities of NDT technicians. Three levels of certification are offered: Level I for technicians who are in training; Level II for technicians that are authorized to perform NDT without supervision; and Level III for the most experienced technicians. As mentioned before, PAUT requires high-skilled operators, so it is desirable to have the most experienced team to perform the test. The ASW D1.5 Bridge Welding Code requires Level II technicians at minimum to execute PAUT. In this research, a technician ASTN Level III, Ph.D, PE, performed the PAUT on selected weldments of the studied bridge.

5.2.1 NDT inspection performed in the studied bridge

The NDT inspection on the bridge occurred on 2/23/2021 through 2/26/21. Besides a close visual inspection, the weld types listed on Table 5.4 were investigated using PAUT and wet visual MT when needed. The locations of the NDT inspections are presented in Figure 5.25 to Figure 5.29.

Table 5.4. Weld types inspected in I-287 bridge.

<table>
<thead>
<tr>
<th>Weld type # / Fatigue detail #</th>
<th>Weld type</th>
<th>Inspected girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1 (D1)</td>
<td>Bottom flange-to-web fillet welds</td>
<td>B8 and B14</td>
</tr>
<tr>
<td>W2 (D5)</td>
<td>Longitudinal stiffener to web fillet welds</td>
<td>B8 and B14</td>
</tr>
<tr>
<td>W3 (D5C)</td>
<td>Longitudinal stiffener splice groove weld</td>
<td>B8 and B14</td>
</tr>
<tr>
<td>W4 (D7)</td>
<td>Girder web splice groove weld</td>
<td>B8 and B14</td>
</tr>
<tr>
<td>W5 (D6b)</td>
<td>Bottom flange transition groove weld</td>
<td>S3 and S4</td>
</tr>
</tbody>
</table>
Figure 5.25. NDT locations performed in I-287 bridge.

Figure 5.26. NDT locations on Girder B8.

Figure 5.27. NDT locations on Girder B14.

Figure 5.28. NDT locations on Girder S3.
The selected welds for the NDT were chosen based on the finite fatigue life outcomes obtained from the assessment performed according to the stress-life and LEFM approaches. Weld type W1 was selected to confirm whether the bottom flange-to-web fillet weld were fully continuous, since a discontinuity could result in much less fatigue resistance. Weld types W2 and W3 are related to Fatigue Details 5A, 5B and 5C. Weld type W4 represented the Fatigue Detail 7. Lastly, weld type W5 was evaluated to confirm the assumption of Detail 6B. It is recalled that girder B8 and B14 are the original fascia girders built in 1966, whereas girders S3 and S4 were added to the bridge in 1996. Therefore, in order to compare possible flaws identified in girder B8, girder B14 was also tested. Girders S3 and S4 were scanned with the purpose of confirming the assumption made for Fatigue Detail 6B containing a partial joint penetration (PJP).

The inspection complied with the requirements given in MBE 3rd Edition (2019 with interims), section 4.3.5.6.10, which states that: “the inspector should be familiar with the types of connections present on the bridge; welded connections should be checked for the development of fatigue cracks; verification of fine cracks on the weld toe should be done; if cracks are visually detected, microscopic or NDT tests should be performed to confirm and define the cracks”. As mentioned before, a technician ASTN Level III, Ph.D., PE, inspected the bridge.
The access to execute NDT on the bridge welds was provided by a truck man-lift. Traffic control was coordinated with NJDOT and it was in accordance with NJDOT’s lane and shoulder closing guidelines. The NDT inspection results are summarized in Table 5.5, Table 5.6 and Table 5.7 for girders B8, B14 and S3-S4, respectively. Figure 5.30 to Figure 5.34 depicts the PAUT outputs for the most important cases. Other results are presented in Appendix F.

### Table 5.5 – NDT results for girder B8

<table>
<thead>
<tr>
<th>Inspection # / Weld Type</th>
<th>Fatigue Detail</th>
<th>Component Tested</th>
<th>Detected discontinuities and Location</th>
<th>Acceptance (AWS D1.5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I01 W4</td>
<td>Detail 7</td>
<td>Web Splice / North</td>
<td>No indications.</td>
<td>Acceptable</td>
</tr>
<tr>
<td>I02 W4</td>
<td>Detail 7</td>
<td>Web Splice / South</td>
<td>Lack of fusion (LOF) at 17.75” – 19.25” from upper surface of lower longitudinal stiffener. Maximum discontinuity height = 0.05”.</td>
<td>Rejected</td>
</tr>
<tr>
<td>I03 W3</td>
<td>Detail 5C</td>
<td>Lower Long. Stiff. Splice / North</td>
<td>Multiple volumetric weld defects along length of weld. Maximum discontinuity height = 0.10” at 0.05” from the surface.</td>
<td>Rejected</td>
</tr>
<tr>
<td>I05 W3</td>
<td>Detail 5C</td>
<td>Lower Long. Stiff. Splice / South</td>
<td>No indications.</td>
<td>Acceptable</td>
</tr>
<tr>
<td>I06 W3</td>
<td>Detail 5C</td>
<td>Upper Long. Stiff. Splice / South</td>
<td>No indications.</td>
<td>Acceptable</td>
</tr>
<tr>
<td>I07 W2</td>
<td>Detail 5</td>
<td>Lower Long. Stiff. / North</td>
<td>Base metal discontinuity characterized as an impurity inclusion. Located 54”- 56” from North web splice. Maximum discontinuity height = 0.10”.</td>
<td>Acceptable</td>
</tr>
<tr>
<td>I08 W2</td>
<td>Detail 5</td>
<td>Lower Long. Stiff. / North</td>
<td>Base metal discontinuity characterized as an impurity inclusion. Located 85”- 86” from North web splice. Maximum discontinuity height = 0.10”.</td>
<td>Acceptable</td>
</tr>
<tr>
<td>I09 W2</td>
<td>Detail 5</td>
<td>Lower Long. Stiff. / North</td>
<td>Crack-like discontinuity close to weld root located at 131”- 133” from North web splice. Crack length &lt; 1”; crack</td>
<td>Rejected</td>
</tr>
<tr>
<td>Inspection # / Weld Type</td>
<td>Fatigue Detail</td>
<td>Component Tested</td>
<td>Detected discontinuities and Location</td>
<td>Acceptance (AWS D1.5)</td>
</tr>
<tr>
<td>--------------------------</td>
<td>----------------</td>
<td>------------------</td>
<td>---------------------------------------</td>
<td>------------------------</td>
</tr>
<tr>
<td>I10 W2 Detail 5</td>
<td>Lower Long. Stiff. / North</td>
<td>Crack-like discontinuity close to weld root located at 228” - 230” from North web splice. Crack length &lt; 1”; crack depth = 0.125”; maximum height = 0.075”.</td>
<td>Rejected</td>
<td></td>
</tr>
<tr>
<td>I11 W2 Detail 5</td>
<td>Upper Long. Stiff.</td>
<td>No indications.</td>
<td>Acceptable</td>
<td></td>
</tr>
<tr>
<td>I12 W1 Detail 1</td>
<td>Bottom Flange</td>
<td>No indications.</td>
<td>Acceptable</td>
<td></td>
</tr>
<tr>
<td>I13 W1 Detail 1</td>
<td>Bottom Flange</td>
<td>No indications.</td>
<td>Acceptable</td>
<td></td>
</tr>
<tr>
<td>I14 ---- -----</td>
<td>Bottom Flange</td>
<td>Maximum scratch depth = 0.05”.</td>
<td>Acceptable</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.6 – NDT results for girder B14.
Intermittent paint cracking and rust discoloration along bottom weld toe line along accessible weld areas. Paint was removed and wet MT was performed. Crack indication observed at likely previous weld repair location (54” - 59” from North web splice).

Lack of fusion (LOF) mid wall located 3” - 4” from web, one inch long. Maximum discontinuity height = 0.01”.

Table 5.7 – NDT results for girders S3 and S4.

<table>
<thead>
<tr>
<th>Inspection # / Weld Type</th>
<th>Fatigue Detail</th>
<th>Component Tested</th>
<th>Detected discontinuities and Location</th>
<th>Acceptance (AWS D1.5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I25 W5</td>
<td>Detail 6B</td>
<td>Bot. Flange Transition / North S3</td>
<td>No indications.</td>
<td>Acceptable</td>
</tr>
<tr>
<td>I26 W5</td>
<td>Detail 6B</td>
<td>Bot. Flange Transition / South S3</td>
<td>No indications.</td>
<td>Acceptable</td>
</tr>
<tr>
<td>I27 W5</td>
<td>Detail 6B</td>
<td>Bot. Flange Transition / North S4</td>
<td>No indications.</td>
<td>Acceptable</td>
</tr>
<tr>
<td>I28 W5</td>
<td>Detail 6B</td>
<td>Bot. Flange Transition / South S4</td>
<td>No indications.</td>
<td>Acceptable</td>
</tr>
</tbody>
</table>
Figure 5.30. PAUT results for inspection 103 on girder B8.

Figure 5.31. PAUT results for inspection 108 on girder B8.
Figure 5.32. PAUT results for inspection I18 on girder B14.

Figure 5.33. Intermittent paint cracking / rust discoloration and MT indication along bottom weld toe line on girder B14 – inspection I22.
From the PAUT results it is observed that the longitudinal stiffener butt weld splice (type W3 weld) is the most critical welded joint. Defects were found in 37.5% of the welds type W3 executed on the bridge (three out of eight longitudinal stiffener splice butt welds have defects). Besides the greater frequency of occurrence, W3 defects were larger than the defects found in welds type W2. Another conclusion from the NDT inspections is that
the bottom flange thickness transition of girders S3 and S4 configure a complete joint penetration (CJP). Therefore, the assumption of Detail 6B is not valid. This result was expected since hands-on inspection on the joint have not indicated the presence of cracks. The discontinuities found on inspection I07, I08 and I22 are parallel to the longitudinal direction of the girder. According to Fisher (1977), fatigue strength is determined by discontinuities that are perpendicular to the applied stress, therefore discontinuities that are parallel to the stress field have no influence on the member fatigue performance. As a result, details I07, I08 and I22 were not included in the current analysis.

5.2.2 Crack idealization based on BS7910

The real shape of a weld imperfection detected by NDT is irregular. In order to make the computation of stress intensity factor feasible, the irregular flaws are transformed into approximated regular shapes. Usually, rectangle and/or ellipses are assigned to the flaws detected in NDT results.

In this research the provision of BS7910 (2005) code were applied to the PAUT results to characterize the flaws. The basic premise is that planar flaws should be characterized by the height and length of their containment rectangles (see Figure 5.36). Therefore, the cracks would be characterized as a rectangle, full ellipse and semi-ellipse in the cases presented in Figure 5.36a, Figure 5.36b and Figure 5.36c, respectively. The possible interactions of multiple imperfections should also be considered.
(a) Through thickness flaw - required dimensions $2a$ and $B$.

(b) Embedded flaw - required dimensions $2a$, $2c$, $p$ and $B$.

(c) Surface flaw - required dimensions $a$, $2c$ and $B$.

Figure 5.36. Characterization of planar flaws (BS7910 2005)

According to BS7910 (2005) section 9.5.5., where there is doubt about the accuracy of the size of flaw established by the inspection procedure, it may be necessary to assume a larger flaw in order to ensure a safe assessment.

5.2.3 LEFM model with PAUT

According to the inspection results, all discontinuities that failed the AWS D1.5 acceptance criteria were further investigated through LEFM approach. Table 5.8 summarizes the details where rejected discontinuities were found and the corresponding crack idealization according to BS7910 provisions.

Table 5.8 – Crack configuration based on PAUT results and BS7910 provisions.

<table>
<thead>
<tr>
<th>Inspection / Detail / Location</th>
<th>Crack drawing</th>
<th>Crack Type / Crack Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Inspection I01</td>
<td></td>
<td>- Semi elliptical surface crack</td>
</tr>
<tr>
<td>- Detail 7</td>
<td></td>
<td>- crack depth $a = 0.05$ in.</td>
</tr>
<tr>
<td>- Girder B8, 17 inches above lower long. stiffener, at 25 ft from South Pier</td>
<td></td>
<td>- crack width $2c = 1.00$ in.</td>
</tr>
<tr>
<td>Inspection</td>
<td>Detail</td>
<td>Girder</td>
</tr>
<tr>
<td>------------</td>
<td>--------</td>
<td>--------</td>
</tr>
<tr>
<td>I03</td>
<td>5C</td>
<td>B8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>I09</td>
<td>5</td>
<td>B8</td>
</tr>
<tr>
<td>I10</td>
<td>5</td>
<td>B8</td>
</tr>
<tr>
<td>17</td>
<td>5C</td>
<td>B14</td>
</tr>
<tr>
<td>18</td>
<td>5C</td>
<td>B14</td>
</tr>
</tbody>
</table>
- Inspection I22  
  - Girder B14, lower long. stiffener 63 ft from South Pier  
  - Longitudinal surface crack  
  - Crack length = 5 in.  
  - No other dimensions were provided.

- Inspection I23  
  - Detail 5C  
  - Girder B14, upper long. stiffener splice, at 30 ft from South Pier  
  - Semi elliptical surface crack  
  - Crack depth \( a = 0.01 \) in.  
  - Crack width \( 2c = 1.00 \) in.

The stress range histogram and the effective stress range for the NDT inspection locations are presented in Figure 5.37. Site-specific WIM data was used to simulate the traffic loads for girder B8, and AASHTO Fatigue design truck was considered for girder B14. It is noted that Inspection 01 is equivalent to Detail 7 and Inspection 02 is equivalent to Detail 5C, thus the \( S_r \) histogram and \( S_{re} \) are presented in Figure 4.21 and Figure 4.18, respectively.

(a) Inspection 09 - \( S_r \) histogram and \( S_{re} \)  
(b) Inspection 10 - \( S_r \) histogram and \( S_{re} \)
Based on the information summarized on Table 5.8, the stress intensity factor expressions discussed in Chapter 2 were applied accordingly. It was found that most of the crack-like defects encountered on the structure will not propagate due to either the low effective stress range at the location or due to the small crack size, which result in stress intensity factor ranges below the stress intensity factor threshold. Table 5.9 shows the results.
Table 5.9 – Status of crack-like defects based on LEFM assessment.

<table>
<thead>
<tr>
<th>Inspection</th>
<th>Stress Intensity Factor Threshold $\Delta K_{th} \left( ksi\sqrt{in} \right)$</th>
<th>Stress Intensity Factor Range $\Delta K_I \left( ksi\sqrt{in} \right)$</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>I01</td>
<td>1.61</td>
<td>0.38</td>
<td>No propagation</td>
</tr>
<tr>
<td>I03</td>
<td>1.61</td>
<td>1.71</td>
<td>Propagation</td>
</tr>
<tr>
<td>I09</td>
<td>1.61</td>
<td>0.86</td>
<td>No propagation</td>
</tr>
<tr>
<td>I10</td>
<td>1.61</td>
<td>0.85</td>
<td>No propagation</td>
</tr>
<tr>
<td>I17</td>
<td>1.61</td>
<td>1.13</td>
<td>No propagation</td>
</tr>
<tr>
<td>I18</td>
<td>1.61</td>
<td>0.25</td>
<td>No propagation</td>
</tr>
<tr>
<td>I23</td>
<td>1.61</td>
<td>0.25</td>
<td>No propagation</td>
</tr>
</tbody>
</table>

Inspection detail I03, which corresponds to Detail 5C, is the only crack-like defect that will propagate. The corresponding remaining fatigue life for Inspection 03 detail was found to be 69 years. It is recalled that the initial LEFM assessment for Detail 5C yielded a remaining fatigue life in between 32 to 82 years. After PAUT was integrated to LEFM model, the remaining fatigue life of Detail 5C was found to be 69 years. It is concluded that PAUT results was paramount for reducing the uncertainty in the remaining fatigue life prediction. It is noted that no discontinuities were found for Detail 5B.

5.3 Remaining fatigue life assessment based on UniGrow model

The UniGrow model is applied for Details 5B and 5C which were the actual critical details of the studied bridge. Firstly, a range of initial crack sizes were assumed to predict the remaining fatigue life. As a second tier of analysis, the PAUT results were coupled with the UniGrow model for the fatigue life prediction (only for Detail 5C).

In order to apply the UniGrow model, the following information are needed: 1) monotonic and cyclic strain properties of the material; 2) the residual stress distribution at the fatigue detail and the residual stress intensity factor; 3) the stress intensity factor of the detail; 4) the elementary material block size $\rho^*$; and 5) the fatigue crack growth constant,
It is noted that after the residual stress intensity factor is calculated, the corresponding total maximum stress intensity factor, $K_{\text{max,tot}}$, and the total stress intensity factor range, $\Delta K_{\text{tot}}$ are calculated accordingly to section 2.4.2 of this dissertation.

5.3.1 Monotonic and cyclic strain properties

As mentioned before, A36 steel was used in all steel members of the bridge. The monotonic and cyclic strain properties were borrowed from Stephens et al. 2001 and presented in Table 5.10. The notation for each variable was defined in Section 2.4.1.

<table>
<thead>
<tr>
<th>Parameter name</th>
<th>Parameter notation</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain hardening exponent</td>
<td>$n_h$</td>
<td>----</td>
</tr>
<tr>
<td>Cyclic strain hardening exponent</td>
<td>$n'$</td>
<td>0.215</td>
</tr>
<tr>
<td>Strength coefficient</td>
<td>$K$</td>
<td>113 ksi</td>
</tr>
<tr>
<td>Cyclic strength coefficient</td>
<td>$K'$</td>
<td>160 ksi</td>
</tr>
<tr>
<td>Fatigue strength coefficient</td>
<td>$\sigma_f$</td>
<td>138 ksi</td>
</tr>
<tr>
<td>Cyclic fatigue Strength coefficient</td>
<td>$\sigma'_f$</td>
<td>143 ksi</td>
</tr>
<tr>
<td>Fatigue ductility coefficient</td>
<td>$\epsilon_f$</td>
<td>1.19</td>
</tr>
<tr>
<td>Cyclic fatigue ductility coefficient</td>
<td>$\epsilon'_f$</td>
<td>0.27</td>
</tr>
<tr>
<td>Fatigue strength exponent</td>
<td>$b$</td>
<td>-0.132</td>
</tr>
<tr>
<td>Fatigue ductility exponent</td>
<td>$c_d$</td>
<td>-0.45</td>
</tr>
</tbody>
</table>

5.3.2 Residual Stresses

For the case of welded joints, residual stresses are introduced due to the welding process before the specimen is subjected to service loads. Therefore, in this research, the residual stress distribution attributed to the welding process is considered in the UniGrow model in lieu of the residual stresses induced by the loading and unloading stress reversals. As a result, the first and only monotonic stress cycle is attributed to the residual stresses
occurred during the welding process, similarly to Pereira Batista (2016) approach for the local-global fatigue analysis of welded joints.

Previous authors have recommended empirical residual stress distribution in a variety of welded joints (Masubuchi and Martin, 1966 and Bate and Green, 1997), considering the longitudinal and transversal directions, as shown in Figure 5.38 for the case of a simple plate butt weld. The longitudinal residual stress, $\sigma_L$, is parallel to the weld seam whereas the transversal residual stress, $\sigma_T$, is perpendicular to the weld seam.

![Figure 5.38. Longitudinal and transversal residual stresses notation (Bate and Green, 1997).](image)

Before considering the effects of residual stresses in the fatigue behavior of any welded joint the appropriate stress directions must be selected. For instance, the applied longitudinal stresses of Detail 5C should be evaluated with the transversal residual stress along the butt weld.

According to BS7910 provisions, the distribution of transversal residual stress along the plate thickness, $\sigma_{r,T}$, for the fillet and butt-welded joints can be estimated by Equations 5.8 and 5.9, respectively.
\[
\sigma_{r,T,fillet} = \sigma_y \tag{Equation 5.8}
\]

\[
\sigma_{r,T,butt} = \sigma_y \left[ 0.9415 - 0.0319 \left( \frac{x}{t} \right) - 8.3394 \left( \frac{x}{t} \right)^2 + 8.660 \left( \frac{x}{t} \right)^3 \right] \tag{Equation 5.9}
\]

where, \( \sigma_y \) is maximum value for the tensile residual stress (often assumed as the yield strength of the material), \( x \) is the measure from the outer surface and \( t \) is the plate thickness.

It is noted that this Equation 5.9 accounts for the butt weld with no restraint, which is the case of Detail 5C, whereas Equation 5.8 is used for Detail 5B.

The corresponding residual stress intensity factor is calculated based on the weight function method and are presented in Equations 5.10 and 5.11. The nondimensional distribution of the residual stresses and the residual stress intensity factor along the plate thickness are depicted in Figure 5.39.

\[
K_{r,fillet} = \sigma_y \sqrt{\pi a} \tag{Equation 5.10}
\]

\[
K_{r,butt} = \left[ 0.9414 + 0.03315 \left( \frac{L}{t} \right) - 4.234 \left( \frac{L}{t} \right)^2 + 3.675 \left( \frac{L}{t} \right)^3 \right] \times \sigma_y \sqrt{\pi a} \tag{E 5.11}
\]

![Figure 5.39. Transversal residual stress distribution and residual stress intensity factor.](image)
5.3.3 Elementary material block size and fatigue crack growth constant

According to Noroozi et al. (2005, 2007), in order to determine the elementary material block size $\rho^*$ some fatigue crack growth data are necessary. The authors have suggested that the material elastic stress-strain relationship may be used for the computation of $\rho^*$ since the fatigue limit (CAFL) is less than the material yield strength.

A simplified way to estimate $\rho^*$ is to use fatigue crack growth data near the threshold region, considering stress ratio $R > 0.5$. At high stress ratios and close to the threshold region, the total stress intensity factors, $K_{max,tot}$ and $\Delta K_{tot}$, have the same magnitudes as the applied stress intensity factors, $K_{max,appl}$ and $\Delta K_{appl}$ (Noroozi et al. 2007). As a result, $\rho^*$ can be estimated based on the applied stress intensity factors.

If the near threshold fatigue crack growth data generated at $K_{max,appl}$ and $\Delta K_{appl}$ is known, the elastic constant $C$ can be determined from Equation 5.12 (Noroozi et al. 2005).

$$ C = \frac{da}{dN} \left[ (K_{max,tot})^p (\Delta K_{tot})^{1-p} \right]^{-\gamma} \quad (Equation \ 5.12) $$

Based on the calculated $C$ value from Equation 5.12, the elementary material block size can be estimated according to Equation 5.13.

$$ \rho^* = \left\{ \frac{C}{2} \left[ \frac{(\psi_{y,1})^2}{4\pi(\sigma_f')^2} \right]^{1/2b} \right\}^{2b/(2b+1)} \quad (Equation \ 5.13) $$

Then, the fatigue crack growth constant $C$ for the plastic and elastic-plastic behavior can be determined by substituting $\rho^*$ into Equations 2.65 and 2.67, respectively. Figure 5.40 summarizes the procedure to estimate $\rho^*$ and $C$ based on fatigue crack growth (FCG) data near the threshold subjected to high stress ratio, e.g., $R > 0.5$. Due to the scatter nature
of fatigue data Noroozi et al. (2007) recommend to compute $\rho^*$ based on several points near the threshold and assume the mean value as the final $\rho^*$ parameter.

Noroozi et al. (2005) concluded that the plastic solution for the fatigue crack growth constant, $C$, consolidated well the entire set of fatigue data above the $\Delta K_{th}$ region, whereas the elastic driving force should be used near the threshold region; “It can be noted that the elastic driving force may be used only to consolidate FCG data at very low fatigue crack growth rates; therefore, it should not be used for the fatigue crack growth predictions away from the threshold ”. As discussed in the beginning of this chapter, the near threshold region can be characterized as $1.50 \text{ ksi} \sqrt{\text{in}} \leq \Delta K_f \leq 1.82 \text{ ksi} \sqrt{\text{in}}$ where the UniGrow model would be more appropriate to predict crack growth based on the predominant elastic behavior at the crack tip. When $\Delta K_f > 1.82 \text{ ksi} \sqrt{\text{in}}$, the plastic driving force is used.
The fatigue crack growth data of the A36 steel published in the NCHRP Report 267 under a stress ratio of $R = 0.8$ was considered for the calculation of the elementary material block size, $\rho^*$, and the elastic and plastic crack growth constants, $C_{el}$ and $C_{pl}$, as seen in Figure 5.41 and Table 5.11.

![Figure 5.41. Fatigue crack growth data for ASTM A36 steel (Fisher et al. 1983).](image)

**Table 5.11 – UniGrow model parameters computed based on fatigue crack growth data.**

<table>
<thead>
<tr>
<th>Datapoint</th>
<th>$da/dN$ (in./cycle)</th>
<th>$\Delta K_{appl}$ (ksi$\cdot$in$^{-1/2}$)</th>
<th>$\rho^*$ (in.)</th>
<th>$C_{el}$</th>
<th>$C_{pl}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.532e-10</td>
<td>2.32</td>
<td>3.465e-02</td>
<td>3.927e-15</td>
<td>1.240e-10</td>
</tr>
<tr>
<td>2</td>
<td>4.642e-10</td>
<td>2.34</td>
<td>4.584e-02</td>
<td>1.814e-15</td>
<td>1.015e-10</td>
</tr>
<tr>
<td>3</td>
<td>3.652e-10</td>
<td>2.28</td>
<td>4.658e-02</td>
<td>1.736e-15</td>
<td>1.003e-10</td>
</tr>
<tr>
<td>4</td>
<td>3.265e-10</td>
<td>2.46</td>
<td>5.945e-02</td>
<td>8.855e-16</td>
<td>8.427e-11</td>
</tr>
<tr>
<td>5</td>
<td>2.154e-10</td>
<td>2.31</td>
<td>5.824e-02</td>
<td>9.369e-16</td>
<td>8.551e-11</td>
</tr>
<tr>
<td>Mean</td>
<td>4.649e-10</td>
<td>2.57</td>
<td>3.997e-02</td>
<td>1.860e-15</td>
<td>9.912e-11</td>
</tr>
</tbody>
</table>
5.3.4 Results

The number of cycles to failure and the remaining fatigue life in years for Detail 5B is presented in Figure 5.42

For the lower bound initial crack size of $a_i = 0.03$ inches, the UniGrow resulted in a remaining fatigue life of 162 years, whereas the LEFM model yielded 93 years when the Fisher model with $\rho = 0$ is used. The larger fatigue life in UniGrow model is attributed to its capability of properly modeling the predominant elastic stress state at the crack tip near the threshold region. For the upper initial crack size of $a_i = 0.02$ inches, UniGrow resulted in 40 years of remaining fatigue life whereas LEFM resulted in 46 years. It is stressed that the plastic driving force employed in the UniGrow model above the threshold stress intensity factor region, assumed a constant residual stress equal to the yield strength of the material, which is a conservative assumption.

For fatigue Detail 5C, the LEFM and UniGrow model were compared firstly assuming a range of initial crack sizes and secondly with the PAUT inputs.
The UniGrow provides higher fatigue life than the LEFM model for the case of small cracks due to its capability of modeling the crack growth assuming the elastic and plastic behavior. Thus, at the lower bound for the initial crack size values, the UniGrow resulted in 128 years whereas LEFM resulted in 82 years of remaining fatigue life. For the upper bound of initial crack size values, both LEFM and UniGrow models yielded 46 years of the remaining fatigue life. It is noted that the residual stress assumption for the fatigue Detail 5C was not as conservative as for the fatigue Detail 5C. As a result, when the distribution of residual stresses is properly employed in UniGrow model, the model tends to agree with the LEFM model in the region above the threshold stress intensity factor. When the PAUT result are included in the crack-based models, the remaining fatigue life was found to be 75 years and 69 years based on the UniGrow and LEFM models, respectively. The results were in good agreement due to the fact that the initial crack size obtained from PAUT yielded a stress intensity factor of 1.70 ksi√in which is close to the $\Delta K_{th} = 1.82 \text{ ksi} \sqrt{\text{in}}$. As a result, most of the fatigue crack growth occurs in the high-rate
regime, where the plastic driving force from UniGrow matched well with the LEFM model, as presented in Figure 5.43b.

5.3.4 Sensitivity analysis

Sensitivity analysis of the UniGrow model was performed considering the variability of the elementary material block size, \( \rho^* \). The number of cycles to failure and the remaining fatigue life in years of Detail 5C was computed for each value of \( \rho^* \), as depicted in Figure 5.44. For the UniGrow sensitivity analysis the initial crack size was fixed according to the PAUT results, e.g., \( a_i = 0.1 \text{ inches} \). The plot presented in Figure 5.44a shows the LEFM and LEFM PAUT results for reference purposes. It is observed that \( \rho^* \) may cause a variability of 35 years on the remaining fatigue life of Detail 5C. On the other hand, it should be noted that the minimum fatigue life obtained by UniGrow was 60 years, which is 20% less than the UniGrow model considering the mean \( \rho^* \) and 8% less than the LEFM with PAUT.

![Figure 5.44. Sensitivity analysis of \( \rho^* \).](image)
Therefore, the lower bound life of UniGrow model is in good agreement with previous results. Additionally, it is stressed that care must be taken when selecting fatigue crack growth data to compute $\rho^*$. 

### 5.4 Comparison of remaining fatigue life based on different approaches

A overall comparison of the remaining fatigue lives investigated in this research according to different approaches is presented in Table 5.12. Based on the analysis results the following conclusions can be drawn:

1. MBE / WIM always provided higher fatigue lives than the MBE / AASHTO. For instance, when Minimum Life Curve is used, Detail 5B has 24 years and 88 years of remaining fatigue life when MBE / AASHTO and MBE / WIM models are considered, respectively. This is attributed to a more accurate definition of $(\Delta T_{\text{SL}})^{\text{present}}$ when site-specific WIM data is used.

2. It is observed that the LEFM model led to higher fatigue lives than the stress-life method based on Minimum and Evaluation 1 life curves, as seen for Detail 5B.

3. The hypothetical classification of category E’ for detail 6B does not hold true since LEFM resulted in less remaining fatigue life than the stress-life method. This agrees to the fact that V-groove welds with partial joint penetration were prohibited in AASHTO Standard Specification 14th Edition (40), meaning that this type of detail would be worse than a category E’.

4. LEFM model with a range of initial crack sizes led to a variability of 50 years in the remaining fatigue life for both details 5B and 5C. For Detail 5C, the conventional
LEFM model resulted in fatigue lives between 32 and 82 years. After PAUT was integrated to LEFM model, the remaining fatigue life of Detail 5C was estimated as 69 years.

5. PAUT results were paramount for the confirmation of assumptions made on the types of welded joints, such as Detail 5B and 6B. Therefore, this methodology could benefit bridge owners in assertive decision regarding the repair, replacement, or inspection intervals of welded joints prone to fatigue.

6. The LEFM tend to be conservative in the fatigue crack growth prediction near the threshold stress intensity factor region, e.g., \(1.50 \text{ ksi} \sqrt{\text{in}} \leq \Delta K_i \leq 1.82 \text{ ksi} \sqrt{\text{in}}\).

7. UniGrow model is capable of simulating the predominant elastic stress state at the crack tip near the threshold stress intensity factor regime in order predict fatigue crack growth. As a result, higher fatigue lives were observed in this region when comparing with LEFM model results.

8. A proper representation of the residual stresses in the UniGrow model improves the prediction of the remaining fatigue life. This conclusion was observed from the comparison of the fatigue crack growth in the high-rate regime between the LEFM and UniGrow models. For Detail 1, where the residual stresses were assumed as the yield strength of the material, the UniGrow model was more conservative than the LEFM. On the other hand, for Detail 2, the distribution of residual stresses was properly modeled. As a results, the remaining fatigue lives between LEFM and UniGrow were very close.

9. It is concluded that care must be taken to select the fatigue crack growth data near the threshold region in order to define the \(\rho^*\) in the simplified manner as proposed by Noroozi et al. (2005).
Table 5.12 – Remaining fatigue life according to different approaches.

<table>
<thead>
<tr>
<th>Detail</th>
<th>MBE / AASHTO Minimum Life</th>
<th>MBE / WIM Minimum Life</th>
<th>Proposed S-N / WIM Minimum Life</th>
<th>LEFM Min</th>
<th>LEFM Max</th>
<th>LEFM with PAUT Min</th>
<th>LEFM with PAUT Max</th>
<th>UniGrow Min</th>
<th>UniGrow Max</th>
<th>UniGrow with PAUT Min</th>
<th>UniGrow with PAUT Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>5B</td>
<td>24</td>
<td>88</td>
<td>103</td>
<td>46</td>
<td>96</td>
<td>N.A.</td>
<td>40</td>
<td>162</td>
<td>N.A.</td>
<td>N.A</td>
<td>N.A</td>
</tr>
<tr>
<td>5C</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>32</td>
<td>82</td>
<td>69</td>
<td>45</td>
<td>128</td>
<td>75</td>
<td>N.A</td>
<td>N.A</td>
</tr>
<tr>
<td>6B</td>
<td>-17</td>
<td>35</td>
<td>36</td>
<td>-12</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A.</td>
<td>N.A</td>
<td>N.A.</td>
<td>N.A</td>
<td>N.A</td>
</tr>
</tbody>
</table>
CHAPTER 6

RELIABILITY-BASED REMAINING FATIGUE LIFE ASSESSMENT

Since many of the variables involved in the fatigue evaluation have intrinsic uncertainties, this chapter aims to estimate the remaining fatigue life of the studied bridge based on the reliability theory introduced in section 2.6 of this thesis. The following subsections present the limit state functions for each methodology used as well as the variability of the random variables considered for the probabilistic fatigue assessment. Lastly, reliability methods are applied, and the remaining fatigue life is estimated based on a target reliability index, $\beta_T$. Since the calibration of AASHTO Fatigue II Limit State was based on a reliability index of $\beta = 1$, the target reliability index for the remaining fatigue life calculation considered in this research is $\beta_T = 1$.

6.1 Variability in the Effective Stress Range, $S_{re}$

As discussed in section 4.4.1.3, before computing the effective stress range, $S_{re}$, appropriate truncation on the stress range histogram is required. The literature has shown
that the truncation level, e.g., stress range cut-off level, can vary from 0.5 ksi to 45% of the CAFL of the corresponding detail category (see section 4.4.1.3). It is recalled that the maximum stress-range cut-off level is defined based on the stress ranges that do not contribute to the cumulative fatigue damage. As lower stress ranges are added in the $S_r$ histogram, the corresponding cumulative fatigue damage tends to become asymptotic, as presented in Figure 4.12. For convenience, the fatigue damage evaluation for different truncation levels for Details 5B, 5C and 6B is presented in Figure 6.1. The effective stress range according to the upper cutoff level is also presented. For all cases the lower cutoff level was considered as 0.50 ksi. It is noted that the dynamic amplification is not considered herein, since this factor is considered as random variable in the limit state function.

![Upper truncation for Detail 5B](image1)

![Upper truncation for Detail 5C](image2)

![Upper truncation for Detail 6B](image3)

**Figure 6.1.** Upper truncation and its corresponding effective stress range for Details 5B, 5C and 6B.
The assessment of the $S_{re}$ variability followed Kwon and Frangopol (2010) methodology. For each stress range cut-off level, different PDFs were fitted to the truncated stress range histogram and the corresponding $S_{re}$ was calculated accordingly. For each assumed PDF, the mean and the standard deviation of the effective stress range was calculated. The Lognormal, Weibull and Gamma distribution were fitted to the truncated stress range histograms. Equations 6.1 to 6.3 show the PDFs of each assumed distribution, respectively.

**Lognormal PDF**

$$f_{S_{r}}(s_{r}) = \frac{1}{s_{r}\sigma \sqrt{2\pi}} \exp \left\{ \frac{-(\log s_{r} - \mu)^{2}}{2\sigma^{2}} \right\}$$  \hspace{1cm} \text{(Equation 6.1)}

where, $\mu$ is the mean (shape parameter) and $\sigma$ is the standard deviation (scale parameter).

**Weibull PDF**

$$f_{S_{r}}(s_{r}) = \frac{\beta \left(\frac{s_{r}}{\alpha}\right)^{\beta-1}}{\alpha} \exp \left\{ -\left(\frac{s_{r}}{\alpha}\right)^{\beta} \right\}$$  \hspace{1cm} \text{(Equation 6.2)}

where, $\alpha$ is the scale parameter and $\beta$ is the shape parameter.

**Gamma PDF**

$$f_{S_{r}}(s_{r}) = \frac{1}{b^{a}\Gamma(a)} s_{r}^{(a-1)} \exp \left\{ -\left(\frac{s_{r}}{b}\right)^{a} \right\}$$  \hspace{1cm} \text{(Equation 6.3)}

where, $a$ is the shape parameter, $b$ is the scale parameter and $\Gamma(\cdot)$ is the gamma function.

For the single slope S-N curves (AASHTO S-N curves), the effective stress range, $S_{re}$, was calculated by using the $m$-th moment of the stress range as defined in Equation 6.4, where $m$ is the slope of the S-N curve and $f_{S_{r}}$ is the assumed stress range PDF. For the case of $m = 3$, Equation 6.4 is equivalent to the root-mean-cube technique.
\[ S_{re} = \left[ \int_{0}^{\infty} s_r^m f_{s_r}(s_r) ds \right]^{1/m} \]  
(Equation 6.4)

Likewise, for bi-linear S-N curves, the \( S_{re}^* \) is calculated according to Equation 6.5.

\[ S_{re}^* = \left[ \int_{0}^{CAFL} (CAFL^{m1-m2}) s_r^{m2} f_{s_r}(s_r) ds + \int_{CAFL}^{\infty} s_r^{m1} f_{s_r}(s_r) ds \right]^{1/m1} \]  
(Equation 6.5)

6.1.1 Statistical parameters of \( S_{re} \) and \( S_{re}^* \) for Detail 5B

For each stress range cutoff level the Lognormal, Weibull and Gamma distribution were fitted to the stress range histograms. Figure 6.2 shows all the histograms considered for Detail 5B and the fitted PDFs. The variability of the fitted PDFs is summarized in Figure 6.3.
Based on the cumulative damage assessment provided by different stress range cut-off levels, the lower and upper stress range cut-off levels are 0.50 ksi and 1.50 ksi for the conventional AASHTO S-N curve, and 0.50 ksi and 2.00 ksi for the bilinear S-N curve. Table 6.1 and Table 6.2 show the mean and standard deviation of the effective stress range and the corresponding PDFs parameters, for the AASHTO S-N curve and the bilinear S-N curve, respectively. For comparison purposes, the effective stress range computed based on the analytical expressions, e.g., Root-Mean cube technique (Equation 2.1) for the case of the single-slope S-N curve, and Equation 4.3 for the case of the bilinear S-N curve, are also presented.
Table 6.1 – $S_{re}$ and PDF parameters based on AASHTO S-N curve – Detail 5B

<table>
<thead>
<tr>
<th>Cut-off level</th>
<th>0.50 (ksi)</th>
<th>0.75 (ksi)</th>
<th>1.00 (ksi)</th>
<th>1.25 (ksi)</th>
<th>1.50 (ksi)</th>
<th>Mean (ksi)</th>
<th>Std. Dev. (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Log.</td>
<td>1.41</td>
<td>1.53</td>
<td>1.76</td>
<td>1.94</td>
<td>2.10</td>
<td>1.75</td>
<td>0.25</td>
</tr>
<tr>
<td>Weibull</td>
<td>1.41</td>
<td>1.55</td>
<td>1.78</td>
<td>1.96</td>
<td>2.12</td>
<td>1.77</td>
<td>0.26</td>
</tr>
<tr>
<td>Gamma</td>
<td>1.38</td>
<td>1.52</td>
<td>1.76</td>
<td>1.94</td>
<td>2.10</td>
<td>1.74</td>
<td>0.26</td>
</tr>
<tr>
<td>Eq. 2.1</td>
<td>1.43</td>
<td>1.56</td>
<td>1.78</td>
<td>1.96</td>
<td>2.11</td>
<td>1.77</td>
<td>0.25</td>
</tr>
</tbody>
</table>

**PDF parameters (shape and scale)**

<table>
<thead>
<tr>
<th>PDF</th>
<th>Shape (mean)</th>
<th>Scale (std. deviation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Log.</td>
<td>0.546</td>
<td>0.165</td>
</tr>
<tr>
<td>Weibull</td>
<td>7.897</td>
<td>1.880</td>
</tr>
<tr>
<td>Gamma</td>
<td>43.316</td>
<td>0.040</td>
</tr>
</tbody>
</table>

Table 6.2 – $S_{re}$ and PDF parameters based on bilinear S-N curve – Detail 5B

<table>
<thead>
<tr>
<th>Cut-off level</th>
<th>0.50 (ksi)</th>
<th>0.75 (ksi)</th>
<th>1.00 (ksi)</th>
<th>1.25 (ksi)</th>
<th>1.50 (ksi)</th>
<th>1.75 (ksi)</th>
<th>2.00 (ksi)</th>
<th>Mean (ksi)</th>
<th>Std. Dev. (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Log.</td>
<td>1.01</td>
<td>1.10</td>
<td>1.30</td>
<td>1.46</td>
<td>1.61</td>
<td>1.79</td>
<td>1.98</td>
<td>1.47</td>
<td>0.33</td>
</tr>
<tr>
<td>Weibull</td>
<td>1.00</td>
<td>1.11</td>
<td>1.32</td>
<td>1.49</td>
<td>1.65</td>
<td>1.82</td>
<td>2.01</td>
<td>1.49</td>
<td>0.34</td>
</tr>
<tr>
<td>Gamma</td>
<td>0.97</td>
<td>1.08</td>
<td>1.29</td>
<td>1.46</td>
<td>1.62</td>
<td>1.80</td>
<td>1.98</td>
<td>1.46</td>
<td>0.34</td>
</tr>
<tr>
<td>Eq. 4.3</td>
<td>1.03</td>
<td>1.14</td>
<td>1.33</td>
<td>1.49</td>
<td>1.64</td>
<td>1.82</td>
<td>2.00</td>
<td>1.49</td>
<td>0.33</td>
</tr>
</tbody>
</table>

**PDF parameters (shape and scale)**

<table>
<thead>
<tr>
<th>PDF</th>
<th>Shape (mean)</th>
<th>Scale (std. deviation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Log.</td>
<td>0.357</td>
<td>0.248</td>
</tr>
<tr>
<td>Weibull</td>
<td>4.960</td>
<td>1.623</td>
</tr>
<tr>
<td>Gamma</td>
<td>17.622</td>
<td>0.083</td>
</tr>
</tbody>
</table>

6.1.2 Statistical parameters of $S_{re}$ for Detail 5C

Figure 6.4 shows all the truncated stress range histograms considered for Detail 5C and the fitted PDFs. The curves from each distribution obtained from each truncated histogram is presented in Figure 6.5.
The mean and the standard deviation of the effective stress range as well as the parameters for the assumed distribution are summarized in Table 6.3.

### Table 6.3 – $S_{re}$ and PDF parameters – Detail 5C

<table>
<thead>
<tr>
<th>Cut-off level</th>
<th>Log. mean (ksi)</th>
<th>Log. std. dev. (ksi)</th>
<th>Weibull mean (ksi)</th>
<th>Weibull std. dev. (ksi)</th>
<th>Gamma mean (ksi)</th>
<th>Gamma std. dev. (ksi)</th>
<th>Eq. 2.1 mean (ksi)</th>
<th>Eq. 2.1 std. dev. (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50 ksi</td>
<td>1.40</td>
<td>1.59</td>
<td>1.82</td>
<td>1.94</td>
<td>2.09</td>
<td>2.26</td>
<td>2.27</td>
<td>1.85</td>
</tr>
<tr>
<td>0.75 ksi</td>
<td>1.40</td>
<td>1.59</td>
<td>1.83</td>
<td>1.96</td>
<td>2.11</td>
<td>2.27</td>
<td>2.27</td>
<td>1.86</td>
</tr>
<tr>
<td>1.00 ksi</td>
<td>1.40</td>
<td>1.59</td>
<td>1.83</td>
<td>1.96</td>
<td>2.11</td>
<td>2.27</td>
<td>2.27</td>
<td>1.86</td>
</tr>
<tr>
<td>1.25 ksi</td>
<td>1.40</td>
<td>1.59</td>
<td>1.83</td>
<td>1.96</td>
<td>2.11</td>
<td>2.27</td>
<td>2.27</td>
<td>1.86</td>
</tr>
<tr>
<td>1.50 ksi</td>
<td>1.40</td>
<td>1.59</td>
<td>1.83</td>
<td>1.96</td>
<td>2.11</td>
<td>2.27</td>
<td>2.27</td>
<td>1.86</td>
</tr>
<tr>
<td>1.75 ksi</td>
<td>1.40</td>
<td>1.59</td>
<td>1.83</td>
<td>1.96</td>
<td>2.11</td>
<td>2.27</td>
<td>2.27</td>
<td>1.86</td>
</tr>
</tbody>
</table>

Figure 6.4. Stress range histograms with fitted PDFs – Detail 5C.

Figure 6.5. Variability of fitted PDFs – Detail 5C.
6.1.3 Statistical parameters of $S_{re}$ and $S_{re}^*$ for Detail 6B

Similar to the other fatigue prone details, the statistical evaluation of $S_{re}$ and $S_{re}^*$ for Detail 6B is presented in Figure 6.6, Figure 6.7, Table 6.4 and Table 6.5.

<table>
<thead>
<tr>
<th>PDF</th>
<th>Shape (mean)</th>
<th>Scale (std. deviation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Log.</td>
<td>0.602</td>
<td>0.178</td>
</tr>
<tr>
<td>Weibull</td>
<td>7.443</td>
<td>1.988</td>
</tr>
<tr>
<td>Gamma</td>
<td>35.862</td>
<td>0.051</td>
</tr>
</tbody>
</table>

Figure 6.6. Stress range histograms with fitted PDFs – Detail 6B.
Figure 6.7. Variability of fitted PDFs – Detail 6B.

Table 6.4 – $S_{re}$ and PDF parameters based on AASHTO S-N curve – Detail 6B

<table>
<thead>
<tr>
<th>Cut-off level</th>
<th>0.50 (ksi)</th>
<th>0.75 (ksi)</th>
<th>1.00 (ksi)</th>
<th>1.25 (ksi)</th>
<th>1.50 (ksi)</th>
<th>1.75 (ksi)</th>
<th>2.00 (ksi)</th>
<th>Mean (ksi)</th>
<th>Std. Dev. (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Log.</td>
<td>1.92</td>
<td>2.01</td>
<td>2.23</td>
<td>2.48</td>
<td>2.63</td>
<td>2.74</td>
<td>2.89</td>
<td>2.42</td>
<td>0.34</td>
</tr>
<tr>
<td>Weibull</td>
<td>1.88</td>
<td>2.01</td>
<td>2.24</td>
<td>2.49</td>
<td>2.65</td>
<td>2.77</td>
<td>2.91</td>
<td>2.42</td>
<td>0.36</td>
</tr>
<tr>
<td>Gamma</td>
<td>1.85</td>
<td>1.98</td>
<td>2.21</td>
<td>2.47</td>
<td>2.62</td>
<td>2.74</td>
<td>2.89</td>
<td>2.39</td>
<td>0.37</td>
</tr>
<tr>
<td>Eq. 2.1</td>
<td>1.92</td>
<td>2.04</td>
<td>2.25</td>
<td>2.49</td>
<td>2.68</td>
<td>2.76</td>
<td>2.90</td>
<td>2.43</td>
<td>0.35</td>
</tr>
</tbody>
</table>

PDF parameters (shape and scale)

<table>
<thead>
<tr>
<th>PDF</th>
<th>Shape (mean)</th>
<th>Scale (std. deviation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Log.</td>
<td>0.872</td>
<td>0.156</td>
</tr>
<tr>
<td>Weibull</td>
<td>7.985</td>
<td>2.577</td>
</tr>
<tr>
<td>Gamma</td>
<td>41.486</td>
<td>0.058</td>
</tr>
</tbody>
</table>

Table 6.5 – $S_{re}$ and PDF parameters based on bilinear S-N curve – Detail 6B

<table>
<thead>
<tr>
<th>Cut-off level</th>
<th>0.50 (ksi)</th>
<th>0.75 (ksi)</th>
<th>1.00 (ksi)</th>
<th>1.25 (ksi)</th>
<th>1.50 (ksi)</th>
<th>1.75 (ksi)</th>
<th>2.00 (ksi)</th>
<th>Mean (ksi)</th>
<th>Std. Dev. (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Log.</td>
<td>1.95</td>
<td>2.03</td>
<td>2.24</td>
<td>2.49</td>
<td>2.63</td>
<td>2.75</td>
<td>2.89</td>
<td>2.43</td>
<td>0.34</td>
</tr>
<tr>
<td>Weibull</td>
<td>1.89</td>
<td>2.02</td>
<td>2.24</td>
<td>2.49</td>
<td>2.66</td>
<td>2.78</td>
<td>2.92</td>
<td>2.43</td>
<td>0.36</td>
</tr>
<tr>
<td>Gamma</td>
<td>1.85</td>
<td>1.98</td>
<td>2.21</td>
<td>2.47</td>
<td>2.63</td>
<td>2.75</td>
<td>2.89</td>
<td>2.40</td>
<td>0.36</td>
</tr>
<tr>
<td>Eq. 4.3</td>
<td>1.94</td>
<td>2.06</td>
<td>2.26</td>
<td>2.49</td>
<td>2.65</td>
<td>2.76</td>
<td>2.91</td>
<td>2.44</td>
<td>0.34</td>
</tr>
</tbody>
</table>

PDF parameters (shape and scale)

<table>
<thead>
<tr>
<th>PDF</th>
<th>Shape (mean)</th>
<th>Scale (std. deviation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Log.</td>
<td>0.877</td>
<td>0.153</td>
</tr>
<tr>
<td>Weibull</td>
<td>7.997</td>
<td>2.583</td>
</tr>
<tr>
<td>Gamma</td>
<td>41.758</td>
<td>0.057</td>
</tr>
</tbody>
</table>
6.2 Error in WIM measurements

Although past research has investigated the probabilistic remaining fatigue life of steel bridges based on WIM data (Guo et al., 2012) no consideration on the WIM measurement uncertainties was provided.

In this research, the randomness of the error obtained in the WIM measurements is taken from the calibration results performed for the site-specific WIM system of the studied bridge, as described in section 3.2 of this thesis. According to the JCSS Probabilistic Model Code – Part 1, the measurement uncertainty is based on the difference between the physical measurement and the actual value. Following the JCSS Code provisions, the WIM error, $e_{WIM}$, is computed as the ratio of the measured GVW over the actual GVW (see GVW measurements on Table 3.1 and Table 3.2). For the developed case study, the GVW load effects governs over the axle weight load effects. Hence, in this research the WIM measurement uncertainty is attributed only to the GVW measurement.

According to JCSS Probabilistic Mode Code – Part 1 the normal distribution has extensively used to model the measurement or modeling uncertainty. The plotted $e_{WIM}$ values in the normal probability paper (see Figure 6.8) confirms that the normal distribution is suitable for $e_{WIM}$ (see the straight line in the normal probability plot). Moreover, Figure 6.8 informs the $e_{WIM}$ mean and standard deviation. It is recalled that the actual GVW of the calibration truck is 66,780 pounds (see Table 3.1 and Table 3.2).
### Table

<table>
<thead>
<tr>
<th>Run #</th>
<th>WIM measured GVW (lbs)</th>
<th>$e_{WIM}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>60,251</td>
<td>0.902</td>
</tr>
<tr>
<td>2</td>
<td>65,046</td>
<td>0.974</td>
</tr>
<tr>
<td>3</td>
<td>71,308</td>
<td>1.068</td>
</tr>
<tr>
<td>4</td>
<td>72,854</td>
<td>1.091</td>
</tr>
<tr>
<td>5</td>
<td>72,712</td>
<td>1.089</td>
</tr>
<tr>
<td>6</td>
<td>65,238</td>
<td>0.977</td>
</tr>
<tr>
<td>7</td>
<td>69,007</td>
<td>1.033</td>
</tr>
<tr>
<td>8</td>
<td>62,019</td>
<td>0.929</td>
</tr>
<tr>
<td>9</td>
<td>56,182</td>
<td>0.841</td>
</tr>
<tr>
<td>10</td>
<td>59,830</td>
<td>0.896</td>
</tr>
<tr>
<td>11</td>
<td>67,482</td>
<td>1.011</td>
</tr>
<tr>
<td>12</td>
<td>68,888</td>
<td>1.032</td>
</tr>
<tr>
<td>13</td>
<td>72,067</td>
<td>1.079</td>
</tr>
<tr>
<td>14</td>
<td>72,125</td>
<td>1.080</td>
</tr>
<tr>
<td>15</td>
<td>71,582</td>
<td>1.072</td>
</tr>
<tr>
<td>16</td>
<td>75,052</td>
<td>1.124</td>
</tr>
</tbody>
</table>

**Statistics**

<table>
<thead>
<tr>
<th>$e_{WIM}$</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.01</td>
<td>0.08</td>
<td>Normal</td>
</tr>
</tbody>
</table>

**Figure 6.8.** Investigation of WIM error statistics and PDF.

### 6.3 Probabilistic Stress-Life Approach

#### 6.3.1 Limit State Functions

As stated in section 2.6, the limit state function for the stress-life method is developed in terms of cumulative fatigue damage (see Equation 2.78). Using the AASHTO S-N curves to compute the number of cycles to failure and expanding the fatigue damage accumulation along the time, $D(t)$, the limit state function can be defined as in Equation 6.6.

$$g(X, t) = \Delta - \frac{N(t). (DLA. e_{WIM} \cdot S_{re})^m}{A} \quad \text{(Equation 6.6)}$$

where, $\Delta$ is the critical damage accumulation, $e_{WIM}$ is the error attributed to the WIM data measurements, $N(t)$ is the deterministic number of cycles per year, $DLA$ is the dynamic load amplification, $S_{re}$ is the effective stress range obtained from calibrated FE model.
coupled with site-specific WIM data, $A$ is the fatigue detail category constant and $m$ is the deterministic slope of the AASHTO S-N curves.

If the number of cycles to failure is computed based on bi-linear S-N curves, e.g., the slope below the CAFL is not equal to the slope in the finite life region, the limit state function is separated in the regions below and above the CAFL (Kwon et al. 2012 and Soliman et al., 2013), as shown in Equation 6.7.

$$g_1(X, t) = \Delta - \frac{N(t) \cdot (DLA \cdot e_{WIM} \cdot S_{r \sigma}^e)^{m_1}}{A_1} \quad \text{for } N(t) \leq N_2 = \frac{A_1}{(CAFL)^{m_1}} \quad (\text{Equation 6.7a})$$

$$g_2(X, t) = \Delta - \frac{N(t) \cdot (DLA \cdot e_{WIM} \cdot S_{r \sigma}^e)^{m_2}}{[CAFL^{(m_2-m_1)}]A_2} \quad \text{for } N(t) > N_s = \frac{A_1}{(CAFL)^{m_1}} \quad (\text{Equation 6.7b})$$

where $A_1$ and $m_1$ are the fatigue detail category constant and the slope above the CAFL, respectively, $A_2$ and $m_2$ are the fatigue detail category constant and the slope below the CAFL, respectively, and $S_{r \sigma}^e$ is the effective stress range computed based on bi-linear S-N curve.

The prediction of the accumulated number of cycles per year, $N(t)$ follows Equation 6.8 (Kwon and Frangopol, 2010), where $y$ is the number of years and $g$ is the traffic increase rate per year.

$$N(t) = 365. ADT_{SL} \int_0^y (1 + g)^y dy \quad \text{(Equation 6.8)}$$

To be consistent with the assumptions made for the deterministic assessment, the traffic increase rate per year is not considered whenever the $ADT_{SL}$ reaches $(ADT_{SL})_{LIMIT}$ of 4000, as discussed in section 4.5.1.
6.3.2 Random Variables

The statistical parameters of the random variables for the probabilistic stress-life approach are shown in Table 6.6.

Table 6.6 – Random variables for the probabilistic Stress-Life approach.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Distribution</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Critical Fatigue Damage, $\Delta$</td>
<td>1.0</td>
<td>0.30</td>
<td>Lognormal</td>
<td>Wirsching, 1984</td>
</tr>
<tr>
<td>Dynamic Load Amplification, $DLA$</td>
<td>1.10</td>
<td>0.08</td>
<td>Lognormal</td>
<td>Guo et al., 2012</td>
</tr>
<tr>
<td>Error in WIM measurements, $e_{WIM}$</td>
<td>1.01</td>
<td>0.08</td>
<td>Normal</td>
<td>This research</td>
</tr>
<tr>
<td>Cat $E$ fatigue constant, $A$ (ksi$^3$)</td>
<td>1.25E+09</td>
<td>1.22E+08</td>
<td>Lognormal</td>
<td>NCHRP Report 721 / SHRP2 R19</td>
</tr>
<tr>
<td>Cat $E'$ fatigue constant, $A$ (ksi$^3$)</td>
<td>6.10E+08</td>
<td>8.05E+07</td>
<td>Lognormal</td>
<td>NCHRP Report 721 / SHRP2 R19</td>
</tr>
<tr>
<td>AASHTO S-N curve slope, $m$</td>
<td>3.00</td>
<td>---</td>
<td>Deterministic</td>
<td>NCHRP Report 286</td>
</tr>
<tr>
<td>Cat $E$ fatigue constant, $A_1$ (ksi$^3$)</td>
<td>1.12E+09</td>
<td>3.61E+08</td>
<td>Lognormal</td>
<td>This research</td>
</tr>
<tr>
<td>Cat $E$ fatigue constant, $A_2$ (ksi$^3$)</td>
<td>5.46E+09</td>
<td>1.24E+09</td>
<td>Lognormal</td>
<td>This research</td>
</tr>
<tr>
<td>Cat $E$ slope, $m_1$</td>
<td>2.80</td>
<td>---</td>
<td>Deterministic</td>
<td>This research</td>
</tr>
<tr>
<td>Cat $E$ slope, $m_2$</td>
<td>3.84</td>
<td>---</td>
<td>Lognormal</td>
<td>This research</td>
</tr>
<tr>
<td>Cat $E'$ fatigue constant, $A_1$ (ksi$^3$)</td>
<td>1.43E+09</td>
<td>5.32E+08</td>
<td>Lognormal</td>
<td>This research</td>
</tr>
<tr>
<td>Cat $E'$ fatigue constant, $A_2$ (ksi$^3$)</td>
<td>1.86E+09</td>
<td>7.88E+08</td>
<td>Lognormal</td>
<td>This research</td>
</tr>
<tr>
<td>Cat $E'$ slope, $m_1$</td>
<td>3.19</td>
<td>---</td>
<td>Deterministic</td>
<td>This research</td>
</tr>
<tr>
<td>Cat $E'$ slope, $m_2$</td>
<td>3.47</td>
<td>---</td>
<td>Deterministic</td>
<td>This research</td>
</tr>
</tbody>
</table>

6.3.3 Reliability Index Calculation

The First Order Reliability Method (FORM) is used to calculate the reliability indexes. If all the random variables are lognormally distributed, the limit state function presented in section 6.3.1 can be stated in a linear format, which makes the FORM suitable to this type of problem. Therefore, for the reliability index calculation based on the FORM, the lognormal effective stress range is considered. In addition, the lognormal PDF is assumed for the error in the WIM measurements. As a result, the computation of $\beta$ for the limit state functions stated on Equations 6.6 and 6.7 is performed according to Equations 6.9 and 6.10, respectively.
\[
\beta(t) = \frac{\mu_\Delta + \mu_A - \{m_1 \cdot \mu_{D_{LA}} + m_1 \cdot \mu_{e_{WIM}} + m_1 \cdot \mu_{S_{re}} + \ln[N(t)]\}}{\sqrt{\sigma_\Delta^2 + \sigma_A^2 + (m_1 \cdot \sigma_{D_{LA}})^2 + (m_1 \cdot \sigma_{e_{WIM}})^2 + (m_1 \cdot \sigma_{S_{re}})^2}}
\]  
(Equation 6.9)

\[
\beta_1(t) = \frac{\mu_\Delta + \mu_{A_1} - \{m_1 \cdot \mu_{D_{LA}} + m_1 \cdot \mu_{e_{WIM}} + m_1 \cdot \mu_{S_{re}} + \ln[N(t)]\}}{\sqrt{\sigma_\Delta^2 + \sigma_{A_1}^2 + (m_1 \cdot \sigma_{D_{LA}})^2 + (m_1 \cdot \sigma_{e_{WIM}})^2 + (m_1 \cdot \sigma_{S_{re}})^2}}
\]  
(Equation 6.10a)

\[
\beta_2(t) = \frac{\mu_\Delta + \mu_{A_2} - \{m_2 \cdot \mu_{D_{LA}} + m_2 \cdot \mu_{e_{WIM}} + m_2 \cdot \mu_{S_{re}} + \ln[N(t)]\}}{\sqrt{\sigma_\Delta^2 + \sigma_{A_2}^2 + (m_2 \cdot \sigma_{D_{LA}})^2 + (m_2 \cdot \sigma_{e_{WIM}})^2 + (m_2 \cdot \sigma_{S_{re}})^2}}
\]  
(Equation 6.10b)

where \( \mu \) and \( \sigma \) are the shape (mean) and scale (standard deviation) of the lognormal random variable.

In order to contemplate the random variables with other PDFs rather than the lognormal distribution, the reliability index is also calculated based on simulation techniques such as the Monte Carlo simulation and the Latin Hypercube Sampling technique.

For all methodologies, it is assumed that there is no correlation among the variables.

6.3.4 Results

The reliability indexes considering the FORM technique is presented in Figure 6.9. Both the AASHTO S-N curve and the proposed bilinear S-N curve were considered.
In order to be consistent with the deterministic approach, the age of the details were subtracted from the origin, e.g., the past years are presented as negative values and the present age is set equal to year zero. The probabilistic remaining fatigue lives are obtained by reading the year value at the intersection between the FORM curves with the horizontal black dashed line representing $\beta_T = 1.0$. The comparison of the deterministic and probabilistic remaining fatigue lives for Detail 5B and 6B, based on the stress-life approach, are presented in Table 6.7. The difference between the deterministic and probabilistic approach is 10% for the AASHTO S-N curves and 3% for the bilinear S-N curve, considering Detail 5B. For Detail 6B, the differences are 42% and 25%, respectively.

<table>
<thead>
<tr>
<th>Detail</th>
<th>Deterministic MBE / WIM (AASHTO S-N)</th>
<th>Deterministic Bilinear S-N curve</th>
<th>Probabilistic FORM AASHTO S-N curve</th>
<th>Probabilistic FORM Bilinear S-N curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detail 5B</td>
<td>88</td>
<td>103</td>
<td>79</td>
<td>100</td>
</tr>
<tr>
<td>Detail 6B</td>
<td>35</td>
<td>36</td>
<td>20</td>
<td>45</td>
</tr>
</tbody>
</table>
6.3.5 Sensitivity Analysis for MC and LHS results.

A sensitivity analysis was performed to compare the reliability indexes obtained from FORM, MC and LHS, considering Detail 5B and the limit state function stated in Equation 6.6. The FORM betas were considered as the benchmark results.

The reliability indexes were calculated for the current year (year zero) up to the next 150 years. The Lognormal, Weibull and Gamma PDFs were assumed for the effective stress range distribution. The results are presented in Figure 6.10 and Figure 6.11, for the Monte Carlo and Latin Hypercube simulations, respectively. Due to the high computational effort involved in these methods, the efficiency of sample sizes of 1,000, 10,000 and 100,000 were compared.

![Figure 6.10. Sensitivity analysis: FORM vs. Monte Carlo simulation.](image)

(a) Sample of 100,000  (b) Sample of 10,000  (c) Sample of 1,000

![Figure 6.11. Sensitivity analysis: FORM vs. Latin Hypercube Sampling.](image)

(a) Sample of 100,000  (b) Sample of 10,000  (c) Sample of 1,000
In general, the reliability indexes among the three methods are very similar especially when $S_{re}$ is assumed to be lognormally distributed (maximum error of 9%). It is noted that the $\beta$s for the $S_{re}$ with Weibull and Gamma distribution are higher than the $S_{re}$ with Lognormal distribution in the range of $\beta \geq \beta_T$. Thus, one can conclude that it is conservative to estimate the probabilistic fatigue lives using the lognormal distribution for the effective stress range. Table 6.8 and Table 6.9 provide the quantitative comparison of betas.

<table>
<thead>
<tr>
<th>Year</th>
<th>FORM</th>
<th>Log</th>
<th>Wei</th>
<th>Gam</th>
<th>Log</th>
<th>Wei</th>
<th>Gam</th>
<th>Log</th>
<th>Wei</th>
<th>Gam</th>
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</thead>
<tbody>
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<td>2.24</td>
<td>2.48</td>
<td>2.39</td>
<td>2.24</td>
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<td>2.34</td>
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<td>2.65</td>
<td>2.51</td>
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<tr>
<td>45</td>
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<td>1.95</td>
<td>2.17</td>
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<td>70</td>
<td>1.23</td>
<td>1.22</td>
<td>1.24</td>
<td>1.30</td>
<td>1.24</td>
<td>1.24</td>
<td>1.29</td>
<td>1.22</td>
<td>1.17</td>
<td>1.37</td>
</tr>
<tr>
<td>95</td>
<td>0.64</td>
<td>0.63</td>
<td>0.57</td>
<td>0.67</td>
<td>0.62</td>
<td>0.54</td>
<td>0.68</td>
<td>0.70</td>
<td>0.59</td>
<td>0.71</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Year</th>
<th>FORM</th>
<th>Log</th>
<th>Wei</th>
<th>Gam</th>
<th>Log</th>
<th>Wei</th>
<th>Gam</th>
<th>Log</th>
<th>Wei</th>
<th>Gam</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>2.24</td>
<td>2.25</td>
<td>2.50</td>
<td>2.39</td>
<td>2.25</td>
<td>2.46</td>
<td>2.40</td>
<td>2.46</td>
<td>2.58</td>
<td>2.33</td>
</tr>
<tr>
<td>45</td>
<td>1.95</td>
<td>1.95</td>
<td>2.14</td>
<td>2.08</td>
<td>1.97</td>
<td>2.09</td>
<td>2.09</td>
<td>1.90</td>
<td>2.14</td>
<td>2.01</td>
</tr>
<tr>
<td>70</td>
<td>1.23</td>
<td>1.23</td>
<td>1.24</td>
<td>1.30</td>
<td>1.23</td>
<td>1.24</td>
<td>1.29</td>
<td>1.19</td>
<td>1.28</td>
<td>1.25</td>
</tr>
<tr>
<td>95</td>
<td>0.64</td>
<td>0.64</td>
<td>0.56</td>
<td>0.67</td>
<td>0.64</td>
<td>0.56</td>
<td>0.67</td>
<td>0.63</td>
<td>0.57</td>
<td>0.69</td>
</tr>
</tbody>
</table>

The efficiency of each method is compared based on the accuracy of beta results (as discussed above) and the computation time. The beta calculation for eleven different years, considering the sample size of 100,000, took 181s and 111s for the MC and LHS, respectively. When the sample size was reduced to 1000, the computation time is decreased to 1.46s and 1.43s for the MC and LHS, respectively.
Therefore, for the purpose of estimating the probabilistic remaining lives, the LHS with sample size of 1000, considering the lognormal distribution for the $S_{re}$ provides reasonable outcomes.

### 6.4 Probabilistic LEFM Approach

#### 6.4.1 Limit State Functions

The limit state function for the probabilistic LEFM analysis is developed in the crack domain rather than the time domain (number of cycles), as shown in Equation 2.79, repeated herein in Equation 6.11 for convenience. The Paris’s law is considered for the probabilistic calculation of the crack size throughout the time $t$.

$$g(X, t) = a_{cr} - a(t) \quad \text{(Equation 6.11)}$$

Paris’s law expression can be written as Equation 6.12, where $F$ represents the generalized stress intensity correction factor, $C$ is the fatigue crack growth constant and $S$ is the stress range.

$$\frac{da}{(F\sqrt{\pi a})^m} = CS^m dN \quad \text{(Equation 6.12)}$$

Following Zhao et al. (1994) and Guo and Chen (2011) approach, the limit state function can be written in terms of a damage accumulation functions $\Phi_R(a_{cr}, a_0)$ and $\Phi_Q(a_N, a_0)$ to facilitate the probabilistic analysis. The expanded form of $\Phi_R$ and $\Phi_Q$ are presented in Equations 6.13 and 6.14, respectively. Also, the updated limit state function is shown in Equation 6.15, where $a_{cr}$ is the critical crack size, $a_i$ is the initial crack size, $N_{cr}$ is the
critical number of cycles to failure, \( N_i \) is the initial number of cycles and \( N(t) \) is the number of cycles at the time \( t \).

\[
\Phi_R(a_{cr}, a_i) = \int_{a_i}^{a_{cr}} \frac{da}{(F\sqrt{\pi a})^m} = \int_{N_0}^{N_{cr}} CS^m dN = CS^m(N_{cr} - N_i) \quad \text{(Equation 6.13)}
\]

\[
\Phi_Q(a_N, a_i) = \int_{a_i}^{a_{N(t)}} \frac{da}{(F\sqrt{\pi a})^m} = \int_{N_0}^{N(t)} CS^m dN = CS^m(N(t) - N_i) \quad \text{(Equation 6.14)}
\]

\[
g(X, t) = \Phi_R(a_{cr}, a_i) - \Phi_Q(a_N, a_i) \quad \text{(Equation 6.15)}
\]

Substituting Equations 6.13 and 6.14 into Equation 6.15, the limit state function can be written as in Equation 6.16, assuming that \( N_i = 0 \).

\[
g(X, t) = \int_{a_i}^{a_{cr}} \frac{da}{(F\sqrt{\pi a})^m} - CS^m N(t) \quad \text{(Equation 6.16)}
\]

In order to further simplify the limit state function, some authors have integrated the left-hand side of Equation 6.16, assuming that the generalized stress intensity correction factor is constant (Guo et al. 2011 and Kwon et al. 2012). However, it is well known that \( F \) is highly dependent on the crack size \( a \), thus is not constant. This research considers Equation 6.16 to account for the variability of \( F \). Furthermore, the error associated with the computation of stress intensity factor is accounted by the random variable \( B_F \) with a mean value of 1 and standard deviation of 0.1 (Wang et al. 2009). Lastly, the dynamic load amplification and the error attributed to the WIM measurements, are added in the limit state function, as presented in Equation 6.17.

\[
g(X, t) = \int_{a_0}^{a_{cr}} \frac{da}{\left(B_F F\sqrt{\pi a}\right)^m} - C (D L A e_{WIM S_{re}})^m N(t) \quad \text{(Equation 6.17)}
\]
6.4.2 Random Variables

The statistical parameters of the random variables for the probabilistic LEFM approach are shown in Table 6.10. The critical crack sizes are treated as deterministic variables and its values follow the same approach used in the deterministic analysis, e.g., they are assumed as the plate thickness or the crack size corresponding to the material fracture toughness, $K_c$. The statistical parameters for the initial crack sizes follows Table 2.1.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Distribution</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic Load Amplificatoin, $DLA$</td>
<td>1.10</td>
<td>0.08</td>
<td>Lognormal</td>
<td>Guo et al., 2012</td>
</tr>
<tr>
<td>Error in WIM measurements, $e_{WIM}$</td>
<td>1.01</td>
<td>0.08</td>
<td>Normal</td>
<td>This research</td>
</tr>
<tr>
<td>Fatigue crack growth constant, $C$</td>
<td>2.05E-10</td>
<td>1.29E-10</td>
<td>Lognormal</td>
<td>Fisher (1984)</td>
</tr>
<tr>
<td>Error for stress intensity factor, $B_F$</td>
<td>1.0</td>
<td>0.1</td>
<td>Lognormal</td>
<td>Wang et al. (2009)</td>
</tr>
</tbody>
</table>

6.4.3 Reliability Index Calculation

The limit state function defined in Equation 6.17 is much more complex that the ones defined in Equations 6.6 and 6.7 for the probabilistic stress-life approach. As a result, the computation effort for the beta calculation becomes higher as well. Therefore, based on the conclusions of the sensitivity analysis performed in section 6.3.5, the LHS with sample size of 1,000 is selected for the computation of the reliability indexes corresponding to Equation 6.17.
6.4.4 Results for Detail 5B and 6B

The reliability indexes based on the LEFM approach are presented in Figure 6.12 for Detail 5B and 6B. The $\beta$ curves from the stress-life approach are also presented. The overall remaining fatigue life results for Details 5B and 6B are summarized in Table 6.11.

![Figure 6.12. Reliability indexes for Detail 5B and 6B.](image)

(a) Reliability indexes for Detail 5B  
(b) Reliability indexes for Detail 6B

For Detail 5B, the LEFM probabilistic remaining fatigue life is higher than the lives obtained in the probabilistic stress-life approach. This trend was also observed in the deterministic approach. Moreover, it is observed that the LEFM results are close to the proposed bilinear S-N curve results, for both the deterministic and probabilistic analyses. Both, the deterministic and probabilistic LEFM results show that indeed the fatigue category corresponding to Detail 6B is worse than AASHTO detail category E’. It is
recalled that Detail 6B was linked to the single-sided V-groove weld presented in the bridge design plan for the girder bottom flange thickness transition.

6.4.5 Results for Detail 5C

According to PAUT results, Detail 5C with the edge crack was the only detail prone to crack propagation. Therefore, the reliability indexes were investigated considering the initial crack sizes as per Table 2.1 as well as the PAUT values.

Previous research on the reliability-based fatigue assessment considered the ultrasonic testing results as the source for the initial crack size variability, as seen in Wang et al. (2009). Likewise, this research uses the crack sizes obtained by PAUT as indicated in Table 5.8 to determine the mean, standard deviation, and the distribution of the initial crack size for the probabilistic LEFM assessment of Detail 5C. Based on the crack values plotted in the normal probability paper, as shown in Figure 6.13, the initial crack size mean and standard deviation are 0.097 inches and 0.06 inches, respectively. In addition, the normal distribution is assumed.

The reliability indexes throughout the time are presented in Figure 6.14. The overall remaining fatigue life results for Details 5C is depicted in Table 6.12. It is noted that the deterministic and probabilistic remaining fatigue life are very similar, with an maximum error of 10% taken the deterministic value as a reference. Therefore, it is concluded that the probabilistic assessment corroborated to the deterministic values, which contributes to a more certain estimation of fatigue life.
Figure 6.13. Crack values from PAUT plotted in the normal probability paper.

Figure 6.14. Reliability indexes for Detail 5C – edge crack.

Table 6.12 – Deterministic and Probabilistic Remaining fatigue life for Detail 5C.

<table>
<thead>
<tr>
<th>Detail 5C</th>
<th>Approach</th>
<th>Deterministic</th>
<th>Probabilistic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LEFM</td>
<td>82</td>
<td>86</td>
</tr>
<tr>
<td></td>
<td>LEFM with PAUT</td>
<td>69</td>
<td>62</td>
</tr>
</tbody>
</table>
CHAPTER 7

LOAD-INDUCED FATIGUE DAMAGE CAUSED BY TRUCK PLATOONING

7.1 Introduction

Truck platooning technology is considered by various researchers to yield positive results in terms of fuel consumption, decrease in CO₂ emission, and increase in highway capacity. The studies completed in 2011 by California’s PATH Program conducted experiments on truck platoons made up of three single trailer 5-axle trucks, longitudinally automated with gaps of 33 feet (Lu and Shladover, 2011). The gap was defined as the headway distances between trucks. The results showed a 10% improvement in fuel consumption. Additionally, between 2008 and 2013, the Japanese Energy Intelligent Transportation System (ITS) program performed a study of platoons made up of three 25-ton trucks plus one light truck with gaps of 33 feet and 15 feet. The results indicated 8% and 15% of fuel savings for the cases of 33-feet and 15-feet gap, respectively. In addition, the study concluded that if 40% of the heavy trucks on expressways were within platoons, the total CO₂ reduction along expressways would be 2.1% at 33-feet gap and 4.8% at 15-feet gap (Tsugawa et al. 2016).
In order to implement this technology in the future and better understand the effects of truck platooning on bridges, further analysis regarding the safety, serviceability and remaining life is needed. This research investigated truck platooning effects on steel girder bridges regarding the fatigue finite life limit state in the cases of simple span and two-equal continuous span bridge. Only longitudinal members were considered. The analysis focused on three aspects:

1. Compare the fatigue damage of truck platoons with different legal trucks; based on the results the number of cycles per truck passage on the bridge when the truck is within a platoon was determined.

2. Investigate the effect of number of trucks and gap distance of platooning on the fatigue damage.

3. Identify the cases where it is more beneficial to travel as truck platoons rather than as individual trucks.

This study assumes that all stress cycles in this study cause cumulative fatigue damage. The fatigue damage analysis used in this paper is following the procedure used in calibrating Fatigue II limit state. Additionally, it was assumed that the trucks in the platoons have the same axle weights and axle configurations. It is important to mention that bridges will behave differently in terms of dynamic impact for trucks traveling in platoons and traveling individually. Nassif and Nowak (1995) concluded the structural response caused by dynamic load, in absolute terms, is practically constant and does not depend on truck weight. However, this study focuses on the relative fatigue damage of truck platoons, which is defined as the ratio between the average fatigue damage by each truck in the truck platoons and fatigue damage caused by the AASHTO LRFD fatigue truck. In order to
simplify the analysis, it is assumed the same dynamic impact for trucks travel in platoons and travel individually. Therefore, the effect of dynamic impact is not discussed in this study. An in-depth study is needed to investigate the dynamic behavior of bridges under the truck platoons and how it will impact the current AASHTO LRFD specifications and AASHTO MBE. More details can be found in Braguim et al. (2021).

7.2 Methodology

A set of state legal loads, including AASHTO’s legal loads, were used to form the platoons made of two, three, and four trucks. In this study, the gap distance is defined as the spacing between the rear axle of the front truck and the front axle of the rear truck. The considered gap distance ranges from 20 ft to 40 ft with intervals of 10 ft. The truck platoons are then simulated in line girder analysis with span lengths ranging from 20 ft to 300 ft, for both simple spans and two-equal continuous spans. The moments are then extracted from the line girder analysis at the midspan of simple spans and at the middle support of continuous spans. Since the beam cross-sections are not defined, the beam stiffnesses are unknown. Thus, the bending moments are used to analyze fatigue loading in lieu of stress ranges. The rainflow counting method is applied to extract the moment ranges, \( M_i \), and their corresponding number of cycles, \( n_i \), for each live load case. This approach counts the number of full reversal cycles, as well as partial cycles, and their range amplitude for a given load time history (SHRP2, 2015). From the outputs of the rainflow counting method, the average fatigue damage caused by each truck in a platoon is then calculated using Miner’s rule criteria, as shown in Equation 7.1.
\[
D_{AVG\ P,1} = \sum_{i} \frac{n_{P_i} \cdot M_{P_i}^3}{A \cdot S_x^3} \times \frac{1}{j} \quad \text{(Equation 7.1)}
\]

where:

- \(D_{AVG\ P,1}\) is the average fatigue damage caused by one truck in a platoon,
- \(M_{P_i}\) is the \(i\)th moment range caused by the truck platoon,
- \(n_{P_i}\) is the number of stress cycles for the stress range caused by \(M_{P_i}\),
- \(A\) is a material constant,
- \(S_x\) is the elastic section modulus, and
- \(j\) is the number of trucks of the platoon.

Based on the basis of Equation 6.1, the average fatigue damage by each truck in the truck platoons is then normalized by the fatigue damage caused by the AASHTO LRFD fatigue load, as shown in Equation 7.2. Equation 7.2 is defined as the relative damage in this study.

\[
\frac{D_{AVG\ P,1}}{D_F} = \frac{\sum_{i} \frac{n_{P_i} \cdot M_{P_i}^3}{A \cdot S_x^3} \times \frac{1}{j}}{\sum_{i} \frac{n_{F_i} \cdot M_{F_i}^3}{A \cdot S_x^3} \times \frac{1}{j}} \quad \text{(Equation 7.2)}
\]

where:

- \(D_F\) is the fatigue damage caused by the AASHTO LRFD fatigue load,
- \(M_{F_i}\) is the \(i\)th moment range caused by the AASHTO LRFD fatigue load, and
- \(n_{F_i}\) is the number of stress cycles for the stress range caused by the moment range \(M_{F_i}\).

Another investigated aspect in this research was to identify the situations when truck platoons are more beneficial than trucks travelling individually. The damage ratio is defined as the average damage caused by one truck in a platoon divided by the damage caused by the same truck but travelling individually. If the damage ratio is less than one, it
means that running the trucks as platoons results in less fatigue damage than running the trucks individually. The damage ratio is defined in Equation 7.3 below:

\[
\frac{D_{AVG\,PL}}{D_{TRUCK}} = \frac{\sum l \frac{n_{Pl} \cdot M_{Pl}^3}{A \cdot S_x^3} \times \frac{1}{j}}{\sum l \frac{n_{Ti} \cdot M_{Ti}^3}{A \cdot S_x^3} \times \frac{1}{j}}
\]

(Equation 7.3)

where:

- \(D_{TRUCK}\) is the fatigue damage caused by the individual truck,
- \(M_{Ti}\) is the \(i^{th}\) moment range caused by the individual truck, and
- \(n_{Ti}\) is the number of stress cycles for the stress range caused by the moment range \(M_{Ti}\).

Based on results from previous studies, it has been shown that single trailer 5-axle trucks are the most promising truck configuration for platooning. Table 7.1 shows the configuration of legal loads selected from various states for the analysis: 1) AASHTO Type 3S2, 2) Colorado Type 3S2, 3) Delaware T540, 4) Florida C5, 5) Illinois IL-PS5-36, and 6) Ohio 5C1. The Florida C5 has the shortest wheelbase while the Illinois IL-PC5-41 has the longest wheelbase. The Illinois IL-PC5-41 has the maximum tandem weight and the Colorado Type 3S2 has the maximum GVW.

<table>
<thead>
<tr>
<th>Legal Load</th>
<th>TW* (ft)</th>
<th>GVW (kips)</th>
<th>AS1** (ft)</th>
<th>AS2 (ft)</th>
<th>AS3 (ft)</th>
<th>AS4 (ft)</th>
<th>AW1*** (kips)</th>
<th>AW2 (kips)</th>
<th>AW3 (kips)</th>
<th>AW4 (kips)</th>
<th>AW5 (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO Type 3S2</td>
<td>41.0</td>
<td>72.0</td>
<td>11.0</td>
<td>4.0</td>
<td>22.0</td>
<td>4.0</td>
<td>10.0</td>
<td>15.5</td>
<td>15.5</td>
<td>15.5</td>
<td>15.5</td>
</tr>
<tr>
<td>Colorado Type 3S2</td>
<td>45.0</td>
<td>85.0</td>
<td>12.0</td>
<td>4.0</td>
<td>25.0</td>
<td>4.0</td>
<td>12.0</td>
<td>20.0</td>
<td>20.0</td>
<td>16.5</td>
<td>16.5</td>
</tr>
<tr>
<td>Delaware T540</td>
<td>41.0</td>
<td>80.0</td>
<td>11.0</td>
<td>4.0</td>
<td>22.0</td>
<td>4.0</td>
<td>8.0</td>
<td>20.0</td>
<td>20.0</td>
<td>16.0</td>
<td>16.0</td>
</tr>
<tr>
<td>Florida C5</td>
<td>36.0</td>
<td>80.0</td>
<td>10.0</td>
<td>4.2</td>
<td>17.7</td>
<td>4.2</td>
<td>10.0</td>
<td>20.0</td>
<td>20.0</td>
<td>15.0</td>
<td>15.0</td>
</tr>
<tr>
<td>Illinois IL-PC5-41</td>
<td>52.0</td>
<td>82.0</td>
<td>10.0</td>
<td>4.0</td>
<td>34.0</td>
<td>4.0</td>
<td>12.0</td>
<td>22.4</td>
<td>22.4</td>
<td>12.6</td>
<td>12.6</td>
</tr>
<tr>
<td>Ohio 5C1</td>
<td>51.0</td>
<td>80.0</td>
<td>12.0</td>
<td>4.0</td>
<td>31.0</td>
<td>4.0</td>
<td>12.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
</tr>
</tbody>
</table>
7.3 Results and Discussion

The effects of the number of trucks and gap distances of truck platoons on the relative damage were investigated as shown in the following sections. In this study, the span to wheelbase ratio, \( SR \), is defined as the ratio of span length and truck wheelbase.

7.3.1 Relative Fatigue Damage \( D_{\text{avg},1}/D_F \) for Simple Span

Figure 7.1 shows a series of plots for the relative damage produced from different platoons for simple span bridges. Each plot shows the relative damage caused by different platoon configurations in terms of the number of trucks and gap distances.

In the cases of spans less than 55 ft (\( SR < 1.5 \)), the Illinois IL-PC5-41 caused the highest damage among all cases due to its heavy tractor tandem based on the results from Figure 7.1a through Figure 7.1i. The Ohio 5C1 has similar GVW and configuration to the Illinois IL-PC5-41, however, the Ohio 5C1 has lower tandem weight. Therefore, for short spans, the fatigue damage is controlled by the tandem weight. For spans greater than 55 ft (\( SR > 1.5 \)), either the Florida C5 with the shortest wheelbase, or the Colorado Type 3S2 with the heaviest GVW, produced the highest fatigue damage. It is observed that for trucks with same gross vehicle weight, e.g. the Florida C5, the Delaware T540 and the Ohio 5C1, the longer is the wheelbase, the lower is the fatigue damage. In some cases, platoons comprised of four trucks cause less fatigue damage than platoons made of two or three trucks. This
happens at span lengths of 40ft to 100ft (Figure 7.1a, Figure 7.1d and Figure 7.1g), 70ft to 150ft (Figure 7.1b, Figure 7.1e and Figure 7.1h), and 90ft to 170ft (Figure 7.1c, Figure 7.1f and Figure 7.1e), for 20ft gap, 30ft gap, and 40ft gap, respectively. This indicates that more trucks in platoons could help decrease the average fatigue damage. This can be explained by the fact that although truck platoons might bring higher load effects, it decreases the number of cycles. The effect of decreasing the number of cycles outweighs the effect of increasing the load effects. Another finding is that, for spans greater than 100 ft, the relative damage decreases as the gap distance increases. The effects of gap distance are discussed in more details in the next section.
7.3.2 Relative Fatigue Damage $D_{AVG P,1}/D_F$ for Continuous Span

For the cases of two-equal continuous spans, the results of $D_{AVG P,1}/D_F$ for the selected trucks are shown in Figure 7.2. For the majority of span lengths, the heaviest truck, the Colorado Type 3S2, and the shortest truck configuration, the Florida C5, are the two trucks that produced more fatigue damage at the middle support of two-equal continuous span bridges. For platoons of two trucks (Figure 7.2a through Figure 7.2c), the maximum relative fatigue damage occurs at span to wheelbase ratios around 2. For platoons of three trucks (Figure 7.2d through Figure 7.2f), the maximum relative fatigue damage occurs at span to wheelbase ratios between 2 and 4. For platoons of four trucks (Figure 7.2g to Figure 7.2i), the maximum relative damage occurs at span to wheelbase ratios between 4 and 6. Additionally, similar important observations confirms that for span to wheelbase ratio less than 2.5, the lowest relative fatigue damage comes from the platoons of four trucks and the highest relative fatigue damage comes from the platoons of two trucks.
7.3.3 Effect of gap distance on fatigue damage

The effects of gap distance on the fatigue damage was analyzed for different span lengths and truck wheelbases. The relationship “$D_{AVG, P,1}/D_F$” versus “Gap (ft)” was evaluated for the cases of simple and two-equal continuous span bridges. For different truck wheelbases it was observed that there are specific span to wheelbase ratios, beyond
which the fatigue damage decreases as the gap distance increases. As an example, the relative damage $D_{AVG,P,1}/D_F$ produced by the Colorado Type 3S2 is plotted in Figure 7.3 for the case of simple span bridge. It was observed that in the case of span length of 80 ft, the relative damage decreases as the gap distance increases from 20 ft to 25 ft, and increases as the gap distance increases from 25 ft to 40 ft. In the cases of span lengths of 107 ft and 120 ft, the relative damage decreases as the gap distances increases, as shown in Figure 7.3b, and Figure 7.3c, respectively. For simple span bridges, the span length of 107 ft is the boundary that defines when larger gaps minimize fatigue damage, for this specific truck, Colorado Type 3S2.

![Figure 7.3](image_url)  
(a) Span length of 80 ft  
(b) Span length of 107 ft  
(c) Span length of 120 ft

Figure 7.3. Fatigue damage ratio $D_{AVG,P,1}/D_F$ versus gap distance for Colorado Type 3S2.

Figure 7.4 summarizes these findings by plotting the above-mentioned specific span to wheelbase ratios for different truck wheelbases. Note that the span ratio (SR) is defined in this study as the span length divided by truck wheelbase (L/TW). In order to exemplify the application of Figure 7.4, taking a truck with wheelbase equal to 35 ft, the fatigue damage for simple spans decreases as the gap distance increases when the span ratio is greater than 3, or the span length is greater than 115 ft. Linear regressions were performed to represent these “boundaries” as shown in Equation 6.4 to Equation 6.7 for

![Figure 7.4](image)

**Figure 7.4. Boundary beyond larger gaps minimize fatigue damage.**

Simple span case (2-, 3- and 4-truck platoon): \( SR = -0.063 \times TW + 5.0 \) \hspace{1cm} (Eq. 7.4)

Two-equal continuous span (2-truck platoon): \( SR = -0.063 \times TW + 7.0 \) \hspace{1cm} (Eq. 7.5)

Two-equal continuous span (3-truck platoon): \( SR = -0.063 \times TW + 9.0 \) \hspace{1cm} (Eq. 7.6)

Two-equal continuous span (4-truck platoon): \( SR = -0.063 \times TW + 11.0 \) \hspace{1cm} (Eq. 7.7)
7.3.4 Damage ratio $D_{AVG\,P,1}/D_{TRUCK}$

Various cases can be identified when transporting loads through truck platoons are more beneficial in terms of cumulative fatigue damage than by using individual trucks. When the damage ratio $D_{AVG\,P,1}/D_{TRUCK}$ (calculated through Equation 6.3) is less than one, means that the truck within the platoon causes less damage than the singular truck traveling individually. It was found in this study that depending on the platoon configuration, there is a range of span lengths where it is more beneficial to run as platoons to minimize fatigue damage. In order to exemplify this conclusion, damage ratios of the Delaware T540 are depicted in Figure 7.5. Figure 7.5a presents the damage ratios in 3-D plots and Figure 7.5b shows the damage ratios when the gap distance is fixed at 30 ft.

Based on Figure 7.5, for the cases of simple span, $D_{AVG\,P,1}/D_{TRUCK}$ is less than one when the span to wheelbase ratio is less than 2.9 for all scenarios. For the case when the gap distance is 30 ft, the damage ratio is less than one for span to wheelbase ratios up to 3.6. For the cases of continuous span bridges, it is also shown that when the span to wheelbase ratio is less than 1.1, the average fatigue damage caused by one truck within the platoon is lower than the damage caused by the truck traveling individually.
(a) $D_{AVG\ P,1}/D_{TRUCK}$ for simple span (SS) and two-equal continuous spans (CS)

(b) $D_{AVG\ P,1}/D_{TRUCK}$ when gap distance is fixed at 30 ft.

Figure 7.5. Fatigue damage ratio $D_{AVG\ P,1}/D_{TRUCK}$ for Delaware T540 legal load.

### 7.4 Conclusions

A set of state legal loads, including AASHTO LRFD legal loads, were used to simulate truck platooning configured with two, three, and four trucks, on a range of hypothetical bridge span lengths. This study focused on the single trailer five-axle trucks. The fatigue damage was evaluated at midspan of simple spans and middle support section of two-equal continuous spans bridges. The average fatigue damage produced by each truck within platoons was normalized by the fatigue damage produced by the AASHTO LRFD fatigue load. The average fatigue damage caused by each truck within platoons is also compared with the fatigue damage caused by truck travelling individually to identify the cases when transporting loads through truck platoons are more beneficial.

Based on the analysis results of this study, the following conclusions can be drawn:

1. For simple spans with span length less than 55 ft, or span to wheelbase ratio less than 1.5, the fatigue damage of truck platoon is controlled by the tandem weight. For simple
spans with span length greater than 55 ft, or span to wheelbase greater than 1.5, shorter wheelbase and larger GVW produce higher fatigue damage.

2. For simple spans with span lengths greater than 100 ft, the relative damage decreases as the gap distance increases. In some cases, platoons comprised of four trucks cause less fatigue damage than platoons made of two or three trucks. This indicate that platoons with more trucks could help decrease the average fatigue damage, since although truck platoons bring higher load effects, it also decreases the number of cycles. The effect of decreasing the number of cycles outweighs the effect of increasing load effects in terms of fatigue damage.

3. For continuous spans, the maximum relative fatigue damage happens at span to wheelbase ratio of 2, 2 to 4, and 4 to 6, for platoons of two trucks, platoons of three trucks, and platoons of four trucks, respectively. For span to wheelbase ratio less than 2.5, the lowest relative fatigue damage comes from the platoons of four trucks and the highest relative fatigue damage comes from the platoons of two trucks.

4. For different truck wheelbases, there are specific span to wheelbase ratios, beyond which, fatigue damage decreases as gap distance increases. These specific span to wheelbase ratios are defined as “boundaries” and are represented with linear regressions.

5. Depending on the platooning configuration, there are ranges of span lengths where it is more beneficial to travel as platoons than as an individual truck.

6. A full calibration study should be performed to develop new live load factors for Fatigue I and II limit states assuming platoons are present in the load spectra.
CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 Summary

This research investigated different methodologies to assess the remaining fatigue life of welded joints of steel bridges, using the deterministic and probabilistic approaches. A case study was performed in a steel bridge located in New Jersey where the proposed methodologies were applied. For the case study, the fatigue loading was based on calibrated Finite Element (FE) model subjected to AASHTO Fatigue truck as well as the site-specific WIM data. Four fatigue resistance models were employed to assess the remaining fatigue lives of the critical fatigue detail: 1) The current MBE methodology based on AASHTO S-N curves was used as the benchmark model; 2) proposed bilinear S-N curve with a less steel slope below the constant amplitude fatigue limit was developed according to the database containing the fatigue experimental tests used to formulate the current AASHTO S-N curves; 3) LEFM and, 4) UniGrow. Phased Array Ultrasonic Testing (PAUT) was used to improve the crack-based fatigue resistance models, e.g., LEFM and UniGrow.

On the load side perspective, the cumulative fatigue damage produced by truck platoons was compared with the fatigue damage caused by the AASHTO Fatigue Truck.
Moreover, an evaluation regarding the gap distances between trucks following each other (headway distance) to minimize fatigue damage was developed. This research also presented cases where it is more beneficial to travel as a platoon rather than individual truck, in terms of cumulative fatigue damage.

8.2 Proposed methodology for fatigue assessment

After a first tier of fatigue analysis based on the stress life method, this research proposes the application of crack-based fatigue resistance models coupled with non-destructive testing as a second tier of analysis in order to provide a more assertive assessment for bridge owners to make decisions regarding the repairing or replacement of a steel member due to fatigue issues. It is highlighted that UniGrow is an alternative crack-based model which is able to address some of the limitations of LEFM, such as the prediction of fatigue crack growth near the threshold stress intensity factor region. For the fatigue load model, it is recommended site-specific WIM data with calibrated FE rather than the conventional AASHTO Fatigue Truck. Figure 8.1 summarizes the proposed methodology.
8.3 Conclusions

A comprehensive fatigue assessment was performed using several methodologies. Based on the result analysis, the following conclusions can be drawn:

1. A piecewise regression applied on the experimental fatigue datapoints used to formulate the current AASHTO S-N curve suggests that the S-N curve slope below the constant amplitude fatigue limit is less steep than the slope from the finite life region. As a consequence, the proposed bi-linear S-N curve yields longer fatigue lives when comparing to the current ones.

2. For small initial crack sizes (magnitude as suggested per JCSS Probabilistic Model Code), the crack-based fatigue resistance models yielded longer fatigue lives than the stress-life method proposed in MBE using the Minimum life curve.
3. The PAUT was paramount to confirm the assumptions regarding the fatigue details 6B and 5B. After the test was implemented in the bridge it was confirmed that the assumption of the V-groove weld with partial joint penetration was not real.

4. A variability of more than 50 years was found for the remaining fatigue life in crack-based models when a range of initial crack sizes are used. Therefore, PAUT highly improved the LEFM and UniGrow models by providing the appropriate initial crack size and thus reducing the uncertainty in the remaining fatigue life.

5. The LEFM tend to be conservative for the fatigue crack growth prediction near the threshold stress intensity factor region, e.g., $1.50 \text{ksi} \sqrt{\text{in}} \leq \Delta K_i \leq 1.82 \text{ksi} \sqrt{\text{in}}$.

6. UniGrow model is capable of simulating the predominant elastic stress state at the crack tip near the threshold stress intensity factor regime in order predict fatigue crack growth. As a result, higher fatigue lives were observed in this region when comparing with LEFM model results.

7. A proper representation of the residual stresses in the UniGrow model improves the prediction of the remaining fatigue life. This conclusion was observed from the comparison of the fatigue crack growth in the high-rate regime between the LEFM and UniGrow models. For Detail 5B, where the residual stresses were assumed as the yield strength of the material, the UniGrow model was more conservative than the LEFM. On the other hand, for Detail 5C, the distribution of residual stresses was properly modeled. As a results, fatigue lives after the threshold region matched well between LEFM and UniGrow.

8. The sensitivity analysis for the elementary material block size showed that the lower bound fatigue life obtained from UniGrow model is 8% less than the LEFM with PAUT model and 20% less than the UniGrow with the mean value of $\rho^*$. It is concluded
that care must be taken to select the fatigue crack growth data near the threshold region in order to define the $\rho^*$ in the simplified manner as proposed by Noroozi et al. (2005).

9. The probabilistic remaining fatigue life assessment corroborated to the deterministic evaluation, which contributed to a more assertive and reliable prediction of fatigue life.

10. It is concluded that trucks within platoons causes less cumulative fatigue damage for certain scenarios. That means that for the fatigue damage analysis, the effect of decreasing the number of cycles outweighs the effect of increasing load effects.
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APPENDIX A

NUMERICAL STRESS INTENSITY FACTOR

The numerical stress intensity factor for a single-edged crack in a finite plate under tension stress is compared to the analytical expression given in Equation 2.11c. Following the notation of Figure 2.6c, the plate geometry is: width \( w = 100 \text{ mm} \), length \( L = 350 \text{ mm} \) and thickness \( t = 8 \text{ mm} \). An initial single-edged crack of 10mm was assumed in the mid-length of the plate. The results are presented in the next Figures.

The finite element model was developed in ABAQUS/CAE 2017. The sensitivity analysis was performed in 2D models under plane stress condition to check whether the stress distribution was satisfactory at the critical cross-section. The initial crack size was modeled as 10 mm, using the crack seam function in ABAQUS. A seam defines an edge or a face in the FE model that is originally closed but can open during an analysis (Abaqus/CAE User’s Guide). This function duplicates the nodes at the seam line making them unrestrained. In this manner the crack opening can be simulated. The plate was modeled under plane stress condition and quadratic shell elements CPS8R were used. For the boundary conditions (BC) were set to restraint the vertical and horizontal displacements in the bottom edge of the plate. Figure 9.1 presents the load and the boundary conditions applied in the plate as well as the three meshes used in the sensitivity analysis.
Figure 9.1. Loads, Boundary conditions and meshes applied to the plate in FEM analysis.

A summary of the mesh features among Mesh 01 to Mesh is presented in Table 9.1.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Mesh 01</th>
<th>Mesh 02</th>
<th>Mesh 03</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overall element size</td>
<td>12 mm</td>
<td>8 mm</td>
<td>4 mm</td>
</tr>
<tr>
<td># of elements 1st contour</td>
<td>8</td>
<td>16</td>
<td>32</td>
</tr>
<tr>
<td>Radius of 1st contour</td>
<td>5 mm</td>
<td>5 mm</td>
<td>5 mm</td>
</tr>
</tbody>
</table>

For all meshes, the stress distribution in the Y direction in the cracked cross section was investigated up to the crack tip. Additionally, the SIF obtained from the FEM via J-integral analysis was compared to the analytical result.

Figure 9.2, Figure 9.3, and Figure 9.4 show the stress distribution in the vertical direction (Y direction), for mesh 01, mesh 02 and mesh 03, respectively.
Figure 9.2. Stress distribution for Mesh 01.

Figure 9.3. Stress distribution for Mesh 02.

Figure 9.4. Stress distribution for Mesh 03.

The comparison of the analytical stress intensity factors to the numerical SIF obtained from meshes 01, 02 and 03 are presented in.

<table>
<thead>
<tr>
<th>Mesh</th>
<th>J-Integral</th>
<th>$K_I (N/mm^{3/2})$</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mesh 01</td>
<td>0.0002090</td>
<td>6.624</td>
<td>-1.16%</td>
</tr>
<tr>
<td>Mesh 02</td>
<td>0.0002103</td>
<td>6.645</td>
<td>-0.85%</td>
</tr>
<tr>
<td>Mesh 03</td>
<td>0.0002112</td>
<td>6.660</td>
<td>-0.63%</td>
</tr>
</tbody>
</table>
The plots in Figure 9.2 to Figure 9.4 show that the maximum vertical stress at the crack tip tends to stabilized around 7 ksi. Additionally, the error for numerical SIF drops from 1.16% in mesh 01 to 0.63% in mesh 03. Therefore, it is concluded that mesh 03 is good enough to proceed with the investigation of SIF for other crack sizes. Also, similar mesh will be used in the 3D FEM. Moreover, it is observed that coarser meshes may result in reasonable values for the J-integral.

Numerical stress intensity factors are obtained via FEM with the J-integral technique as explained in section 6.1.3.7. Under the linear behavior, the J-integral is equivalent to the energy release rate $G$. Thus, the correlation to the J-integral values to the stress intensity factor for mode I and plane stress condition is given in Equation A.1.

$$K = \sqrt{J \times E}$$  \hspace{1cm} (Equation A.1)

Five contours were considered for the calculation of the path-independent integral. The radius for the first contour of 5 mm was defined by applying a partition in the part module of the Abaqus model. Abaqus/CAE computes successive contour integrals for a two-dimensional model by adding layers of elements (Abaqus/CAE User’s Guide), as depicted in Figure 9.5.
If the geometry of the crack region defines a sharp crack, the strain field becomes singular at the crack tip. Including the singularity in the model for a small-strain analysis improves the accuracy of the contour integral and the stress and strain calculations (Abaqus/CAE User’s Guide). In this study a $1/\sqrt{r}$ singularity for LEFM application was considered at the crack tip.

The investigation of numerical SIF via FE with the J-integral technique was performed in 2D plane stress model and 3D models with solid elements. For the 2D FE model, the elements, loads and BC used were already mentioned in the sensitivity analysis section. For the 3D FE model, linear solid elements type C3D8R were employed. The bottom of the plate was fixed and a pressure of 1 MPa was applied on the top of the plate.
(analogous to the 2D case). The definition of the crack seam and crack tip in the 2D and 3D FE model are presented in Figure 9.6.

![Figure 9.6. Crack seam and crack tip defined for 2D and 3D models.](image)

Table 9.3 presents the results for the SIF for cracks from 10 mm to 50 mm. The overall maximum error among the analytical and numerical SIF was 3.2%, showing that the numerical procedure is reliable.

<table>
<thead>
<tr>
<th>a (mm)</th>
<th>2D FEM</th>
<th>3D FEM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>J integral</td>
<td>K (N/mm²)</td>
</tr>
<tr>
<td>10</td>
<td>0.0002112</td>
<td>6.7</td>
</tr>
<tr>
<td>15</td>
<td>0.0003382</td>
<td>8.4</td>
</tr>
<tr>
<td>20</td>
<td>0.0005490</td>
<td>10.7</td>
</tr>
<tr>
<td>25</td>
<td>0.0008206</td>
<td>13.1</td>
</tr>
<tr>
<td>30</td>
<td>0.0012104</td>
<td>15.9</td>
</tr>
<tr>
<td>35</td>
<td>0.0017738</td>
<td>19.3</td>
</tr>
<tr>
<td>40</td>
<td>0.0026011</td>
<td>23.4</td>
</tr>
<tr>
<td>45</td>
<td>0.0038524</td>
<td>28.4</td>
</tr>
<tr>
<td>50</td>
<td>0.0057906</td>
<td>34.9</td>
</tr>
</tbody>
</table>

The plot for the SIF versus the crack size curve is shown in Figure 9.7 for the analytical and numerical cases. It is noted that for the case of 3D FE model the mid-thickness SIF factor was considered.
Figure 9.7. SIF curves for analytical and numerical solutions.

The differences were very small and the curves overlapped. Therefore, it is concluded that the FEM with the J-integral technique is a reliable way to obtain SIF.
APPENDIX B

WIM DATA QUALITY ASSURANCE

Figure 10.1. WIM quality control for I287 bridge – October 2018.

Figure 10.2. WIM quality control for I287 bridge – September 2018.
Figure 10.3. WIM quality control for I287 bridge – December 2018.

Figure 10.4. WIM quality control for I287 bridge – January 2019.

Figure 10.5. WIM quality control for I287 bridge – February 2019.
Figure 10.6. WIM quality control for I287 bridge – March 2019.

Figure 10.7. WIM quality control for I287 bridge – April 2019.

Figure 10.8. WIM quality control for I287 bridge – May 2019.
Figure 10.9. WIM quality control for I287 bridge – June 2019.

Figure 10.10. WIM quality control for I287 bridge – July 2019.

Figure 10.11. WIM quality control for I287 bridge – August 2019.
Figure 10.12. WIM quality control for I287 bridge – September 2019.
APPENDIX C

STRUCTURAL HEALTH MONITORING

C.1 SHM Sensors Location

Note: “T.F.” stands for Top Flange; “B.F.” stands for Bottom Flange; “E” stands for East and “W” stands for West.

Figure 11.1. Sensor location on the east fascia girder B8.

Figure 11.2. Sensor location on the adjacent interior girder B9.
Figure 11.3. Sensor location on girders B11 and B14.

Figure 11.4. Tiltmeter location on girders B8 and B9.
Figure 11.5. Diagnostic load test results.
APPENDIX D

FINITE ELEMENT MODEL

Figure 12.1. Bridge framing plan with structural members notation.

Figure 12.2. Geometry of the steel plates used in girders B8, B14, S3 and S4.
Figure 12.3. Geometry of the steel plates used in girders B9 to B13.

Figure 12.4. Geometry and angle sections used in cross-frames CF1 to CF3.

Figure 12.5. Geometry and steel sections used in diaphragms DIA1 to DIA3.

(a) Fixed Bearing at South Pier  (b) Expansion Bearing at North Pier
Figure 12.6. Girder bearing types according to the bridge design plans.

Figure 12.7. Strain response in the mid-web of girder B8 – sensor B3683.

(a) Exit lane
(b) Right Lane

Figure 12.8. Strain response in the mid-web of girder B9 – sensor B2489.

(a) Exit lane
(b) Right Lane

Figure 12.9. Strain response in the mid-web of girder B11 – sensor B2054.
Figure 12.9. Strain response in the bottom flange of girder B11 – truck on the Right Lane.

(a) Five-axle truck (Class9)          (b) Truck weight and configuration based on WIM data

(c) Girder B8 strain response                          (d) Girder B9 strain response

Figure 12.10. Strain analysis based on truck recorded in WIM system – truck 02 on Right Lane.

(a) Five-axle truck (Class9)          (b) Truck weight and configuration based on WIM data
Figure 12.11. Strain analysis based on truck recorded in WIM system – truck 03 on Exit Lane.

Figure 12.12. Strain analysis based on truck recorded in WIM system – truck 04 on Exit Lane.
(a) Five-axle truck (Class9)  (b) Truck weight and configuration based on WIM data

(c) Girder B8 strain response  (d) Girder B9 strain response

Figure 12.13. Strain analysis based on truck recorded in WIM system – truck 05 on Exit Lane.
**APPENDIX E**

**FATIGUE DETAILS**

<table>
<thead>
<tr>
<th>Description</th>
<th>Category</th>
<th>Constant $A$ (ksi)$^2$</th>
<th>Threshold $S_{10}^M$ ksi</th>
<th>Potential Crack Initiation Point</th>
<th>Illustrative Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete joint penetration groove welds back-grooved and welded from the second side, or by continuous fillet welds parallel to the direction of applied stress.</td>
<td>B</td>
<td>$120 \times 10^6$</td>
<td>16</td>
<td>From surface or internal discontinuities in the weld away from the end of the weld</td>
<td><img src="image1.png" alt="Image" /></td>
</tr>
</tbody>
</table>

**Figure 13.1. Classification of fatigue-prone Detail 1 per AASHTO LRFD (AASHTO LRFD 8th Edition, 2017).**

<table>
<thead>
<tr>
<th>Description</th>
<th>Category</th>
<th>Constant $A$ (ksi)$^2$</th>
<th>Threshold $S_{10}^M$ ksi</th>
<th>Potential Crack Initiation Point</th>
<th>Illustrative Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1 Base metal at the toe of transverse stiffener-to-flange fillet welds and transverse stiffener-to-web fillet welds. (Note: includes similar welds on bearing stiffeners and connection plates). Base metal adjacent to bearing stiffener-to-flange fillet welds or groove welds.</td>
<td>C'</td>
<td>$44 \times 10^6$</td>
<td>12</td>
<td>Initiating from the geometrical discontinuity at the toe of the fillet weld extending into the base metal</td>
<td><img src="image2.png" alt="Image" /></td>
</tr>
</tbody>
</table>

**Figure 13.2. Classification of fatigue-prone Detail 2 per AASHTO LRFD (AASHTO LRFD 8th Edition, 2017).**

<table>
<thead>
<tr>
<th>Description</th>
<th>Category</th>
<th>Constant $A$ (ksi)$^2$</th>
<th>Threshold $S_{10}^M$ ksi</th>
<th>Potential Crack Initiation Point</th>
<th>Illustrative Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.3 Base metal at the gross section of high-strength bolted joints designed as slip-critical connections with pretensioned high-strength bolts installed in holes drilled full size or扩machined and rounded to size — e.g., bolted flange and web splice and bolted stiffeners. (Note: see Condition 2.3 for bolted holes punched full size; see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates.</td>
<td>B</td>
<td>$120 \times 10^6$</td>
<td>16</td>
<td>Through the gross section near the hole</td>
<td><img src="image3.png" alt="Image" /></td>
</tr>
</tbody>
</table>

**Figure 13.3. Classification of fatigue-prone Detail 3 per AASHTO LRFD (AASHTO LRFD 8th Edition, 2017).**
7.1 Base metal in a longitudinally loaded component at a detail with a length \(L\) in the direction of the primary stress and a thickness \(t\) attached by groove or fillet welds parallel or transverse to the direction of primary stress where the detail incorporates no transition radius:

- \(L < 2\) in. C \(44 \times 10^6\) 10
- \(2\) in. \(\leq L \leq 12\) or 4 in. D \(22 \times 10^6\) 7
- \(L > 12\) or 4 in.
- \(t < 1.0\) in. E \(11 \times 10^6\) 4.5
- \(t \geq 1.0\) in. E' \(3.9 \times 10^6\) 2.6

(Nota: see Condition 7.2 for welded angle or tee section member connections to gusset or connection plates.)

Figure 13.4. Classification of fatigue-prone Detail 4 per AASHTO LRFD (AASHTO LRFD 8th Edition, 2017).

5.3 Base metal and weld metal in longitudinal stiffener- web or longitudinal stiffener-box flange welds:

With the stiffener attached by welds and with transition radius provided at the termination:

- Stiffener thickness \(< 1.0\) in. E \(11 \times 10^6\) 4.5
- Stiffener thickness \(\geq 1.0\) in. E' \(3.9 \times 10^6\) 2.6

6.2 Base metal and weld metal in longitudinal web or longitudinal box-stiffeners connected by continuous fillet welds parallel to the direction of applied stress:

- \(A = 120 \times 10^6\) 16

From surface or internal discontinuities in the weld away from the end of the weld.

Figure 13.5. Classification of fatigue-prone Detail 5A and Detail 5B per AASHTO LRFD (AASHTO LRFD 8th Edition, 2017).
MATLAB rainflow algorithm (Mathwork.com)

Initially, rainflow turns the load history into a sequence of *reversals*. Reversals are the local minima and maxima where the load changes sign. The function counts cycles by considering a moving reference point of the sequence, $Z$, and a moving ordered three-point subset with these characteristics:

1. The first and second points are collectively called $Y$.
2. The second and third points are collectively called $X$.
3. In both $X$ and $Y$, the points are sorted from earlier to later in time, but are not necessarily consecutive in the reversal sequence.
4. The range of $X$, denoted by $r(X)$, is the absolute value of the difference between the amplitude of the first point and the amplitude of the second point. The definition of $r(Y)$ is analogous.

The rainflow algorithm is as follows:

<table>
<thead>
<tr>
<th>Description</th>
<th>Category</th>
<th>Constant $A$ (Gal)^2</th>
<th>Threshold $(\Delta F)^{th}$ kip</th>
<th>Potential Crack Initiation Point</th>
<th>Illustrative Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.3 Base metal and weld metal in or adjacent to the toe of complete joint penetration groove welded T or corner joints, or in complete joint penetration groove welded butt splices, with or without transverse thickness having slopes no greater than 1:2.5 when weld reinforcement is not removed. (Note: cracking in the flange of the &quot;T&quot; may occur due to out-of-phase bending stresses induced by the stem.)</td>
<td>C</td>
<td>$44 \times 10^6$</td>
<td>10</td>
<td>From the surface discontinuity at the toe of the weld extending into the base metal or along the fusion boundary</td>
<td><img src="image" alt="Illustration" /></td>
</tr>
</tbody>
</table>
Figure 13.8. MATLAB rainflow function algorithm (Mathworks.com).
APPENDIX F

FATIGUE DATABASE

The identification of the datapoints provided in the fatigue database in SHRP2 (2015) is presented below.

Figure 14.1. Fatigue datapoints for Plain Rolled Beams detail.

Figure 14.2. Fatigue datapoints for Longitudinal Welds detail.
Figure 14.3. Fatigue datapoints for Flange Splices detail.

(a) AASHTO database (NCHRP 286)  
(b) Identified datapoints of SHRP2 Database

Figure 14.4. Fatigue datapoints for Longitudinal Welds, Box Girder detail.

(a) AASHTO database (NCHRP 286)  
(b) Identified datapoints of SHRP2 Database

Figure 14.5. Fatigue datapoints for Web Attachments detail.

(a) AASHTO database (NCHRP 286)  
(b) Identified datapoints of SHRP2 Database
Figure 14.6. Fatigue datapoints for Web Gusset Plates detail.

Figure 14.7. Fatigue datapoints for Flange Tip Attachments, Rectangular Plates detail.

Figure 14.8. Fatigue datapoints for Flange Tip Attachments, Groove Welded with Radius detail.
Figure 14.9. Fatigue datapoints for Flange Tip Attachments, Fillet Welded with Radius detail.

Figure 14.10. Fatigue datapoints for Coverplated Beams Wide Plate detail.

Figure 14.11. Fatigue datapoints for Thick Coverplated Beams detail.
Figure 14.12. Fatigue datapoints for Cruciform Joint detail.

Figure 14.13. Fatigue datapoints for Longitudinal Attachment Specimens detail.

Figure 14.14. Fatigue datapoints for Attachment Specimens detail.
APPENDIX G

PAUT RESULTS

Figure 15.1. PAUT results for inspection I02 on girder B8.

Figure 15.2. PAUT results for inspection I07 on girder B8.
Figure 15.3. PAUT results for inspection I17 on girder B14.

(a) I22: intermittent paint cracking and rust discoloration along bottom weld toe line.

(b) I22: wet visual MT indication along weld toe line.

Figure 15.4 Visual and wet MT for inspection I22 on girder B14.