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Review on fatigue life assessment methods for welded joints in orthotropic steel decks of long-span bridges

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Abstract. Orthotropic Steel Decks have been used in long-span bridges for several decades because of their high capacity to weight ratio. However, many fatigue related issues have been reported. This paper provides an overview of the main existing fatigue prediction models and discusses their relevance for the fatigue life assessment of Orthotropic Steel Bridge Decks (OSBDs). Several case studies have proven the importance of considering the combined effect of wind and traffic loadings to estimate the fatigue life of long-span bridges. The importance of incorporating welding residual stresses is also well documented while it is often disregarded in design practices. Reliability-based fatigue assessment methods make it possible to quantify how the sources of uncertainty related to loading conditions, welding residual stresses or fabrication defects can affect the fatigue reliability of OSBDs. Monte Carlo simulations are often used to perform probabilistic analyses, but machine-learning algorithms are very promising and computationally efficient. The shortcomings of the Palmgren-Miner rule are discussed and the need for alternative damage accumulation indexes is clear. A number of conclusions are drawn from the analysis of fatigue tests conducted on OSBDs.

1. Introduction

Orthotropic Steel Bridge Decks (OSBDs) appeared in the 1920s. They are characterized by a low dead weight and a high loading bearing capacity. They have been used in various types of structures such as cable-supported bridges. Most orthotropic bridges consist of deck plates (typical thickness between 10 and 16 mm) stiffened in the longitudinal direction by ribs and in the transverse direction by cross beams or diaphragms regularly spaced longitudinally, typically every 3 to 5 meters.

The present paper aims to discuss the main aspects of the fatigue assessment of Orthotropic Steel Decks in long-span brides. In the first section, the main fatigue pathologies observed in existing OSBDs are described. In Section 2, an overview of the main fatigue prediction models is provided, and the importance of damage accumulation rules is also emphasized. Section 3 examines the possibilities offered by reliability-based assessment methods. The main loads expected to affect the fatigue life of OSBDs are discussed in Section 4. Finally, Section 5 gathers some important lessons from fatigue tests carried out on OSBDs.

Different types of stiffeners have been used since the introduction of OSBDs in the bridge industry. The present paper discusses only the fatigue behavior of OSBDs with closed trapezoidal stiffeners. As described in [1] and by De Jong[2], fatigue cracks have been observed in many existing bridges with OSBDs. As revealed by traffic measurement campaigns conducted in the Netherlands, these damages can allegedly be explained by the number of heavy vehicles passing the bridge exceeding the traffic volume the bridges with reported fatigue cracks were designed to accommodate.

Zhang [3] refers to an inspection program conducted in Japan on 7000 OSBDs where fatigue damages have been reported. From the inspection data, two main locations emerge as the most likely locations for fatigue cracks in traditional OSBDs: Rib-decks joints (19% of cases) and Rib-diaphragm

joints (38% of cases). De Jong mentions four weld locations where fatigue induced cracks are prone to appear: (1) Deck plate, (2) Rib-to-deck joint, (3) Rib-splice joint and (4) Rib-diaphragm joint.

Cracks in the deck plate can result in visible damages in the asphalt layer as shown in Figure 1. They may constitute an element of risk with regards to traffic safety.

Figure 1. Indication of the presence of deck plate cracks [1].

Cracks in the deck plate can occur either at the intersection between the trapezoidal stiffeners and the diaphragm or in the field, between two diaphragms. According to De Jong [1], the crack initiates at the root of the longitudinal fillet weld between the stiffener and the deck plate. A through-thickness crack growth phase ensues. A third phase of horizontal crack growth in the longitudinal starts. De Jong states that the length of the crack at the bottom plate is about four times the deck plate thickness.

The welded joints between the webs of the stiffeners and the diaphragms behave like rigid connections, attracting fixed moments and thus leading to high stress concentration factors. In addition, the deck plate acts as a top flange for the diaphragms. Therefore, deck plate cracks jeopardize the bearing capacity of the diaphragms.

Cracks in the rib-deck joints initiate often at the root of the weld between the fillet weld between the webs of the stiffeners and the deck plate and grows into the weld itself as shown in the left-hand figure in Figure 2.
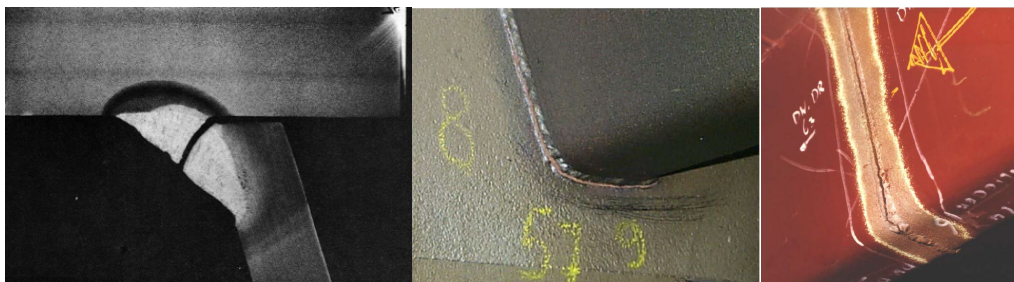

Figure 2. *(Left)* Fatigue failure of the rib-to-deck joint [4] *(Centre)* Crack in rib-diaphragm joint [2] *(Right)* Crack in a stiffener splice joint [2].

Once the crack has grown through the thickness of the weld, it continues to expand in the longitudinal direction. Other fracture mechanisms in the rib-deck joints are discussed in section 5.1.

Rib-splice joints constitute the most common technique to weld stiffeners extremities by means of backing strips. The splice joints are typically positioned in the areas with lowest bending moments. However, because of misalignment issues, insufficient penetration or shrinkage capacity of the weld, splice joints can exhibit insufficient fatigue capacity (see the right-hand figure in Figure 2).

As for the rib-to-deck joint, potential fatigue cracks occurring in the rib-diaphragm joint do not represent a direct threat to traffic safety as other load paths allow stress redistribution. A crack in the rib-diaphragm joint can be seen in the middle figure in Figure 2.

As mentioned by Leendertz [5], many small fatigue cracks in rib-diaphragm joints with and without cope holes were revealed by routine inspections.

In his doctoral thesis, Kolstein [6] studied two additional welded joints in a typical OSBD:

- Butt joint in the deck plate
- Diaphragm to deck plate joint

2. Standard evaluation methods

The previous chapter showed that OSBDs are not problem free. Most damages requiring repair programs are fatigue related. The main reasons behind these reported defects can be one of the following issues:

- Increase of the traffic volume and heavy vehicle weights exceeding the initial estimations
- Execution (misalignment, weld quality)
- Design errors (inadequate detail category, choice of fatigue prediction model)

The intent of the section 2. is to give an overview of the main fatigue prediction models. Additional models can be found in the literature.

2.1. Fatigue prediction models

2.1.1. Main assessment methods. The European standards [8] defines the main structural details in an orthotropic steel bridge deck whose fatigue life shall be thoroughly evaluated (Figure 3.)
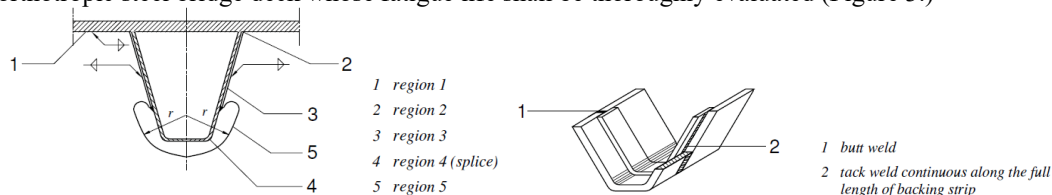

Figure 3. *(left)*. Critical regions for fatigue design and *(right)*. Stiffeners with splice plates [8].

Two main approaches to fatigue design are allowed in the Eurocodes:

(1) Safe life method: the considered structure should maintain the intended level of performance, throughout its design lifetime, without requiring maintenance or reparation and guarantee the desired level of reliability.

(2) Damage tolerant approaches: The desired reliability level should be guaranteed despite the presence of initial defects whose growth throughout the intended design life will be studied. Maintenance and repair programs shall be considered when evaluating the fatigue life of a structure, defined as the number of cycles necessary for the initial imperfection to expand to a critical size

Design standards cover only stress-based methods for both approaches. This constitutes a significant limitation as stress-based methods fail to capture accumulated local plastic damage.

As highlighted by Suresh [15], stress-based approaches are relevant for loading cycles of constant amplitude. In the current engineering practice, simplified rules based on the Miner damage summation rule are given. The fatigue assessment methodology relates the stress amplitude blocks from the measured or simulated loads to the results of constant amplitude fatigue tests listed in most design standards.

2.1.2. Stress-based prediction models. Hobbacher [17] provides a description of the main stress-based approaches. The European standards [7] provide only guidelines on the use of the nominal and hot-spot stress methods.

Other prediction models are discussed in the subsequent paragraphs.

- *Nominal stresses*: Nominal stresses account for the macro-geometric stress raising effects but they disregard the effect of the welded joints themselves. Once the stress ranges induced by the fluctuating loads are determined, the fatigue capacity of the considered detail will be determined by means of standard S-N curves.

  If the imperfections at a given welded joint are expected to exceed the potential angular or axial misalignment and fabrication defects already covered in the S-N curves provided, for instance, in [7], then the secondary stresses resulting from fabrication flaws must be incorporated in the fatigue analysis by means of an additional stress magnification factor. As discussed by Kang [10], the nominal stress approach tends to be overly conservative for steel of high quality as the fatigue strength curves are based on the assumption of multiple welding defects. The nominal stress is not suited to assess the fatigue performance of complex weld geometries. The local prediction models (hot-spot stress, notch stress, strain energy based) should be preferred when assessing the fatigue life of complex welded details.

- *Hot spot stresses*: Hot spot stresses are used to model the stress raising effects of a given detail without taking into account the geometry of the weld itself. The principle of the hot-spot method is to determine the stress level at the expected crack initiation point, typically the weld toe. Stress at the hot spot is obtained by extrapolation from reference points. The estimation of the hot spot requires the selection of adequate types of elements and mesh size. To circumvent these modelling challenges, Dong [11] proposed two mesh-insensitive numerical procedures to extrapolate the hot-spot stress. The first one is based on the evaluation of stress components and was proven by Doerk [12] to deliver inaccurate results when components of high stress concentration are studied. The second procedure expresses the equilibrium equations as a function of nodal forces and seems to yield more reliable results.

- *Effective notch stress*: The effective notch stress is defined as the stress at the root of a fictitious notch located at weld toes or weld roots. The fictitious enlarged notch is an idealization and is employed to model the stress reduction caused by the presence of notches. The concept of enlarged notches was first introduced by Neuber [18] but further improved by Fricke [19] who developed a method to account for the notch effect at weld root by the introduction of keyholes- or U-shaped notches. Even though the effective notch concept was originally developed for high-cycle fatigue, the International Institute of Welding (IIW) permits its application to low- and medium-cycle fatigue. The use of a standard and conservative notch radius of 1 mm, as initially suggested by Radaj [20], for sharp and mild weld notches, shall be combined with the design S-N curve corresponding to the fatigue design class FAT 225 scaled down with a minimum fatigue notch factor 1.6 at weld root or toe. Further details can be found in [19] and [21].

  The shape of the weld is taken into account in the form of an effective shape defined by a unique flank angle. Axial and angular misalignments can significantly affect the local notch stress and may have to be considered either by incorporating them in the numerical models or by the introduction of stress amplification factors. Guidelines are provided by Hobbacher [17].

  The theory of critical distance presented by Taylor [13] constitutes a method to estimate stress concentration at notches in high-cycle fatigue. Instead of determining the stress directly at the notch, the stress is evaluated at a critical distance from the notch. The critical distance is a function of material properties. Different variations of the methods are described by Taylor. The material is assumed to remain elastic in the vicinity of the stress evaluation point, which simplifies the stress estimation procedure without compromising its reliability.

2.1.3. Strain Based prediction models: Strain life approaches are relevant at relatively large stress levels that are likely to induce substantial plastic strain. Initially, strain-based approaches were used exclusively for low-cycle fatigue life assessments. Dong [22] developed a structural strain model for

low-cycle fatigue. Dong's structural strain model can be applied to materials exhibiting elastic perfectly plastic behavior or strain-hardening behavior. Pei [23] extended the applicability of Dong's structural strain to high-cycle fatigue by introducing an equivalent structural strain range parameter whose validity for both low- and high-cycle fatigue was verified experimentally.

Kang [10] emphasizes that strain-based methods can only be used to describe the crack nucleation phase. To perform a total life evaluation, fracture mechanics would have to be applied to the remaining crack propagation life.

2.1.4. Notch stress intensity: Atzori [25] shows that Notch Stress Intensity Factors (NSIFs) can be efficient to evaluate stress fields at weld joint singularities subjected to mode I and mode II loadings, corresponding to crack opening and sliding modes of fracture. The NSIF based approach constitutes a good alternative to fracture mechanics as it can be used to describe short crack propagation and the complex interaction between the different modes of fracture. In the NSIF approach, the weld toe region is modelled, conservatively, as a V-notch with a notch tip radius equal to zero. Luo [26] proposed parametric formulae to estimate NSIFs of weld roots for rib-to-deck joints of OSBs. The parameters included in the formulae are: the penetration rate, the weld flank angle, ratio rib/deck thickness and the relative weld height. Kang [10] discusses the possibility of estimating NSIF by FEM with highly refined meshing. Lazzarin [27] proved that NSIFs could be efficiently derived from the strain energy density (SED) averaged over a control volume.

2.1.5. Density of strain energy: The fatigue capacity of welded joints can be estimated with the averaged Strain Energy Density (SED) approach in which the density of strain energy is averaged over a control volume whose radius is a function of the fracture toughness and fatigue strength.

Berto [28] gives a description of the SED based methods and emphasizes the main advantages of these methods. The evaluation of Strain Energy Density (SED) does not require highly refined meshing, it can account for the interaction of different modes of fracture, and it can also efficiently treat the problem of multiple crack initiation. Luo [29] showed that the SED method could predict more accurately the fatigue life of rib-deck joints than the conventional hot-spot stress and effective notch stress methods.

2.1.6. Fracture mechanics: Fatigue life comprises of two main phases: (1) a crack initiation phase and (2) a crack propagation phase. Randaj [21] underlines that the distribution of the fatigue life between the two phases depends on the presence of notches in the considered specimen. For notched specimen, most of the fatigue life will be spent on propagating the existing defects while unnotched specimens will remain in the crack initiation phase for the most part of their fatigue life.

It remains challenging to simulate the crack initiation phase, but the theory of linear elastic fracture mechanics (LEFM) provides useful information on the development of an existing crack or imperfection. In fatigue assessments, fracture mechanics is used to study the growth of a crack from its initial size to a size that is deemed to induce failure of the specimen, typically between 50 % and 70 % of the wall thickness. The stress intensity factor (SIF) range evaluated at the tip of a crack is the main driving factor of fatigue crack growth, Hobbacher [17] provides a list of explicit relations to calculate the SIF of standard configurations. Gadallah [24] developed a FEM based methodology to obtain the SIF of complex structural details subjected to mixed modes of loading, including in presence of welding residual stress.

Kang [10] underlines the importance of choosing an adequate crack growth law and recommends the use of the Paris' law, in compliance with the recommendations of the IIW. There exists no consensus on the size of the initial fatigue crack, values between 0.05 mm and 0.2 mm can be found in the literature. Hobbacher [17] and Randaj [14] recommend an initial crack depth of at least 0.1 mm.

Atzori [25] provides a methodology to derive the SIFs for cracks located at weld toes or roots from the NSIFs of uncracked specimens. The NSIF based model is primarily to be used in the crack nucleation and short crack propagation phase. The methodology proposed by Atzori makes it possible to combine the NSIF and fracture mechanics approaches into a unified evaluation of the total fatigue life of a welded joint.

2.1.7. Continuum damage mechanics model: Continuum damage mechanics (CDM) models aim to characterize damage evolution through the entire fatigue life. However, the selection of the damage variables and the law describing their evolution requires a high level of expertise.

Cui [30] used an elastic-plastic fatigue damage model to study the effect of Welding residual stresses and their relaxation. The total strain tensor is then divided into an elastic component and a plastic component, derived from a plastic dissipation potential. The case study conducted by Cui illustrates the implementation of a CDM based method. The material hardening parameters used in the CDM model must be determined experimentally and are specific to the chemical composition of the construction material. The CDM based method is very promising but it remains limited to materials for which experimental data is available.

2.2. Effect of multiaxial loading

The prediction models described in section 2.1. correspond to uniaxial fatigue theory. Despite the absence of clear guidelines in design standard, it can be relevant to consider the effect of multiaxial loading on the fatigue resistance of welded structures, such as OSBDs, subjected to complex stress fields.

Kang [10] describes the main multiaxial fatigue prediction models: the effective equivalent stress model, the critical plane model and the modified Wöhler's curve method. Other models are available in the literature.

The European standards and the IIW provide guidelines to assess the fatigue strength of welded joints subjected to multiaxial loading. The Von Mises stress, the principal stress as well as interaction rules between shear and normal stresses are presented as valid models. Kurshid [31] showed that the models proposed in design standards predict successfully the fatigue capacity of welded joints when the multiaxial stress state is characterized by a constant principal stress direction (proportional loading) but they tend to yield unconservative results in presence of a varying principal stress direction (non-proportional loading). Alternative prediction models for non-proportional loading must then be considered.

2.2.1. Critical plane method: Fatemi [32] states that, in the case of ductile materials, fatigue cracks are likely to nucleate and propagate on the plane where shear stresses are high. The plane of preferred crack propagation is also referred to as the critical plane. Different variations of the critical plane method exist: strain-based, stress-based or energy-based. They have different realms of application. Stress-based models, such as the Findley [33] model, are well suited for high-cycle multiaxial fatigue where limited plastic deformation is expected.

The Findley model and its alternative forms do not capture the effect of non-proportional loading accurately. Shen [34] introduced a non-proportional correction coefficient in the Findley criterion to account for the effect of phase difference between the multiaxial loadings.

Fu [35] demonstrated the need to consider the multiaxial stress state of deck-rib joints under the passage of vehicles, making the use of multiaxial fatigue theory necessary. Fu recommends the use of the critical plane method to assess the fatigue damage from non-proportional loading at rib-deck joints.

2.2.2. Modified Wöhler Curve Method: The Modified Wöhler Curve Method (MCWM) is used to evaluate the fatigue capacity of notched specimens in the medium- and high-regime. It considers the interaction of shear stress amplitude and maximum normal stress in the crack initiation phase. Susmel [36] discusses the background of the MWCM and argues that the persistent slip bands forming under cyclic loading and leading to microcracks form along the plane of least microstructural resistance, also referred to as the easy glide plane. Susmel argues that fatigue damage is mainly influenced by the microplastic shear strain acting along the easy glide plane.

In the MWCM described by Kurshid [31], a multiaxial stress ratio parameter is defined as the ratio between the maximum normal stress and the shear stress amplitude on the reference plane. The defined ratio is then a function of the phase shift in loading. The corrected fatigue life is derived from the stress ratio and the reference shear stress which is obtained from the reference range in pure axial or bending

and the reference range in pure torsion at 2×10^6 cycles provided by the design standards. The inverse slope of the modified Wöhler curve is derived from the reference S-N curves given in standards.

2.2.3. Effective equivalent stress hypotheses: Kurshid [31] describes the procedure to calculate the Effective Equivalent Stress Hypothesis (EESH) for ductile materials subjected to non-proportional loading. Shear stress is calculated and integrated over all planes. The result of the angular integration defines a damage parameter.

The equivalent stress is defined as a function of the combined in-phase loading and the ratio between the in-phase damage parameter and out-of-phase damage parameter. The equivalent stress can then be compared with the appropriate S-N curve corresponding to pure axial loading.

2.3. Limitations of the Palmgren-Miner rule and alternative damage accumulation method
Suresh [15] and Schijve [16] underline the limitations of the Palmgren-Miner rule (PM rule), commonly used to assess damage accumulation. The validity of the PM rule relies on the following assumptions:

- The loading sequence has no impact on the fatigue life of the considered structure or specimen
- The loading can then be divided into blocks of the same amplitude. From the number of cycles corresponding to each stress block, the corresponding fraction of damage is obtained.

The accumulation of fatigue damage involves complex phenomena such as local strain hardening, the presence of residual stresses or the formation of local plasticity at the crack tip. The damage caused by a given component of the load spectra will depend on the accumulated damage caused by previous cycles. A reliable accumulation damage index must incorporate the effect of load sequence.

Agerskov [37] showed, through fatigue tests and fracture mechanics analyses, that the use of the PM rule could lead to unconservative results as it fails to capture the effect of crack closure mechanisms by overlooking the distribution between tensile and compressive stresses but also the presence of welding residual stresses. Agerskov conducted both constant and variable amplitude fatigue tests on steel structures and concluded that fatigue life is generally shorter with variable amplitude loading.

As stipulated in [7], under loadings of variable amplitude, if at least one stress range, in the considered sample, exceeds the cut-off limit then fatigue damage will be induced by all stress ranges. This way, the contributions of stress ranges below the cut-off limit are no longer unduly ignored as suggested originally in the PM rule.

Despite its obvious shortcomings, the PM rule is still widely used in the fatigue assessment of OSBDs of long-span bridges. This is partly due to its ease of implementation. Schijve [16] estimates that the PM rule can perform satisfactorily when the load spectrum can be characterized as flat and when a positive mean stress is expected.

As discussed in Section 4.2, welding residual stresses (WRS) play a significant role on the fatigue life of welded joints in OSBDs. The PM rule does take their presence into account. The need for alternative damage indexes becomes obvious when considering the fatigue life of long-span suspension bridges that are subjected to complex dynamic loads.

Aïd [38] describes an alternative damage accumulation index that accounts for the sequence effect of variable amplitude fatigue loading. The new nonlinear fatigue damage model proposed by Aearan [39] is very promising as it successfully models the loading sequence. No additional material parameter is required. The applicability of this new damage index to the fatigue life assessment of long-span bridges with an orthotropic steel deck should be further studied.

3. Fatigue reliability

3.1. Purpose of fatigue reliability analyses
Fatigue design is challenging because of the numerous sources of uncertainty affecting the mechanisms of fatigue and, consequently, the fatigue life of a structure. Probabilistic methods constitute efficient frameworks to quantify how the uncertain parameters affect the fatigue reliability. Særstad [38] compared the findings from a traditional deterministic approach with a probabilistic method to assess

the fatigue life of a road bridge. The probabilistic approach yields a much shorter fatigue life than the deterministic method.

Han [38] underlines the relevance of probabilistic approaches to assess the fatigue life of long-span steel bridges. The case study carried out by Han illustrates the possibilities offered by reliability-based approaches to evaluate how certain parameters such as traffic volume and vehicle weight can affect the fatigue life of a bridge. In a later publication where the combined effect of wind and random traffic loads on a long-span suspension bridge was studied, Han[42] proposed an influence surface approach as an alternative to the computationally demanding Monte Carlo simulations method without compromising the accuracy of the results.

Structural health monitoring programs are a valuable and continuous source of data to establish representative probabilistic models for the wind or traffic loading observed on an existing bridge. However, given the dimensions of strain gauges, it can be challenging to monitor precisely the evolution of strain at the location where high stress concentrations are expected to take place. Numerical approaches, despite their complexity, are often preferred to monitor the stress level at the fatigue prone details of OSBDs. Chen [43], Ni [45] and Deng [46] proposed efficient frameworks to evaluate the fatigue reliability of long-span steel bridges. Most fatigue reliability assessments rely on the establishment of realistic probability distribution functions for each individual loading. When the effect of multiple loadings is studied, the randomness of combination must be accounted for. Monte Carlo simulations can be used to generate random load combinations from probabilistic load models and then the structural response can be estimated by using a Finite Element Model. The case studies conducted by Chen, Ni and Deng showed that the probability distribution of the stress range at the sensitive locations of an OSBDs are affected by several engineering decisions such as the choice of:

- An adequate fatigue prediction model (stress, strain energy based, fracture mechanics etc.)
- A damage accumulation index and the definition of a limit state function.
- Probability density function for the considered stress range. In the case study conducted by Deng [46], it appeared that the stress density distribution for the joints between longitudinal ribs would be best approximated by a Gaussian Mixture Model while a lognormal distribution is more suited for rib-deck joints.

Zhang [44] proved that accounting for the combined dynamic effects of wind and traffic loading can result in a significantly lower fatigue life than otherwise obtained from traditional design strategies where wind and traffic are considered separately. Zhang also developed a detailed framework to account both for the bridge-vehicle interaction and road surface deterioration leading to the conclusion that these parameters were of low significance for long-span bridges with orthotropic steel decks.

3.2. Selection of random variables

In addition to the uncertainties related to the loads acting on long-span bridge, material properties and fabrication processes should be also regarded as random variables influencing the fatigue reliability of an OSBD.

Zhao [52] proposed a framework for strain-based fatigue reliability analyses where the material parameters in the stress-strain relation as random variables. Jakubczak [54] proved through a series of fatigue tests that these material parameters exhibit a limited scatter and can be treated as uncorrelated random variables. Dong [55] discusses the importance of accounting for the randomness of additional variables when performing the fatigue assessment of a welded joint. After having considered a large number of random variables, Dong showed that the reliability index of welded joints was significantly influenced by the following parameters:

- Weld geometry (especially misalignments, weld toe radius). Hobbacher [17] suggests the use of stress magnification coefficients to account for the effect of misalignment when the expected misalignment exceeds the misalignment limits that have already been accounted for in the fatigue resistance S-N curves of welded details (in the case of butt welds between 5 and 10 % of the plate thickness).

- Presence and intensity of the residual stresses
- Presence and size of secondary notches resulting in secondary notch effects. Schork [56] describes a methodology to incorporate secondary notch effects in fatigue reliability assessments.

3.3. Efficient reliability assessment frameworks

Zhu [53] proposed an efficient numerical framework using machine-learning algorithms based on field monitoring data to predict the response of long-span bridges which are subjected to the combined action of wind and traffic. Subsequently, fatigue damages accumulating throughout the bridge service life can be estimated from the estimated structural response.

The wind field for a long-span bridge is difficult to describe and there are multiple sources of uncertainty. Consequently, in a reliability fatigue assessment accounting for damages induced by wind loading, many random variables would have to be defined resulting in a very large number of simulations if the traditional Monte Carlo approach was chosen.

4. Loads contributing to fatigue damage

4.1. Traffic loads

The European design standards [8] provide different Fatigue load models to estimate the fatigue life of critical details. The Fatigue Load Model 5 is based on the recorded or projected traffic data at bridge data and intends to represent realistic traffic conditions.

The repeated passage of heavy lorries is often considered as the principal source of fatigue damage on OSBDs. To predict accurately how the dynamic loads from traffic affect the fatigue reliability of a long-span bridge, the following random variables were studied by Zhang [44] and were found to have a limited impact on the fatigue reliability of the bridge:

- vehicle speed
- road surface deterioration (surface roughness)
- bridge-vehicle interaction

Zhang recommends neglecting the above parameters in further studies.

4.2. Welding residual stresses

Residual stress in welded joints result from local restrained contraction and expansion that take place during the welding process. The heated area cannot shrink freely during the cooling phase which causes the formation of tensile residual stresses in the vicinity of the welded area. Compressive forces appear outside the weld area to balance the tensile residual forces. Webster [63] emphasized the importance of taking residual stresses into consideration when performing fatigue life assessments. The residual tensile forces discussed above can have an unfavorable impact on fatigue performance while compressive stresses can improve locally the fatigue properties of a structural detail. Residual stresses exceeding the yield strength of the material were measured in the experiments carried out by Webster [63]. Webster also recommends the use of fracture mechanics approach for fatigue life estimations when the residual stresses are expected to be present throughout the thickness of the joint.

Cui ([47] and [48]) underlines the role played by welding residual stresses at the deck-to-rib joints in crack initiation. The presence of welding residual stresses combined with the action of other service loads is believed to be the main cause of fatigue crack damages at the deck-to-rib welded joints. Very few studies have considered the effect of residual stresses on fatigue life despite the experimental evidence of their importance. Cui [47] proposes an alternative approach to complex thermomechanical analyses to model residual stresses and analyze the nonlinear superposition of residual stresses and stresses from service loads. The proposed method is deterministic and neglects the uncertainties during the welding process.

Gu [51] proposed the Welding Distortion Source method to estimate deterministically the welding distortion from a thermomechanical analysis performed on a refined local model of the studied joint, represented by a hinge. The displacements estimated in the local model are then transferred to the full-

size finite element model. Dong [67] also developed a methodology that was not specifically designed to model the influence of residual stresses in the welded joints of OSBDs. However, its relevance to the study of OSBDs deserves to be further investigated. The method described by Dong aims to determine the duration of the fatigue crack initiation period by modifying the stress-strain relation of the material to incorporate the effect of residual stresses on the local stress-strain history. The implementation of Dong's method is less time-consuming than the more traditional method that consists of imposing the residual stress field in a Finite Element Model.

Van der Berg [66] investigated the role played by residual stresses on fatigue crack propagation. Using the extended finite element method (XFEM) to implement linear elastic fracture mechanics theory, van der Berg demonstrated that tensile forces in the transverse direction of cracks forming at rib-deck joints tend to accelerate the propagation rate while tensile forces in the longitudinal direction have the opposite effect. Mode I, causing the crack to open orthogonally to crack surfaces, appears to be the dominant mode of failure.

4.3. Wind loading
The effect of wind loading is often discarded in the fatigue design of orthotropic steel bridge decks. Bridge design standards such as [8] and [9] do not provide specific guidelines to estimate the fatigue damage induced by wind-loading of bridges.

Xu [49] presented a framework to investigate the fatigue damage caused by wind loading on a long-span bridge exposed to high-wind speeds. Damage accumulation is estimated using continuum damage mechanics. The cumulative fatigue damage is found to be very limited. Liu [50] conducted a similar study on a suspension bridge and found a significantly higher estimation of the expected accumulated wind-induced fatigue damage. Liu used, as a damage index, the PM rule whose limitations are discussed in the present paper.

The case studies published by Han [38] and Zhang [44] confirm that wind alone is unlikely to cause critical fatigue damage. However, it can be unconservative to ignore the contribution of wind in the fatigue assessment of long-span bridges. Zhang and Han showed that the study of combined effect of wind and traffic could reveal serious potential fatigue diseases on OSBDs even though these potential defects would not be significant if only traffic loads were considered.

5. Findings from fatigue tests performed on orthotropic steel bridge decks
Numerous fatigue tests have been conducted on OSBDs. The most comprehensive collection of fatigue test results can be found in the doctoral thesis of Kolstein [6]. The fatigue test data gathered by Kolstein was used by the authors of the European standards to assign adequate detail categories to the different welded joints in OSBDs.

The present paragraph presents some of the main conclusions that could be drawn from fatigue tests conducted on OSBDs.

5.1. Rib-deck welded joint
Several studies have shown that the rib-deck joint is the preferred location for the occurrence of fatigue cracks in OSBDs.

Ji [57] identified four types of fatigue cracks in the rib-deck longitudinal joints (Figure 4.):
- *Root-deck* cracks originating at the root of the rib-deck weld and growing though the thickness of the deck plate
- *Toe-deck* cracks originating at the toe of the rib-deck weld and growing though the thickness of the deck plate
- *Toe-rib* crack originating at the lower toe of the rib-deck weld and growing though the thickness of the rib
- *Weld root* crack originating at the root of the rib-deck weld and growing into the weld

Ji [57] focused on *root-deck* fatigue cracks. Because of limited accessibility, it is particularly challenging to detect their presence. Consequently, it is of utmost importance to ensure that *root-deck* joints have an adequate fatigue life.

Fatigue tests of rib-deck joints conducted by Kainuma [58], Ya [59] and Sim [60] revealed discrepancies in the fatigue strengths of the toe-deck and *root-deck* crack locations even though both crack locations are assigned identical fatigue properties in design standards [8] and [9].

Sim [62] conducted fatigue tests on OSBDs to investigate the effect of partial-joint-penetration (PJP) on the fatigue performance of *rib-deck* welded joints. A PJP of 80% yielded a better fatigue resistance than a full penetration weld, this could be due to lower welding residual stresses in the case of partial penetration.
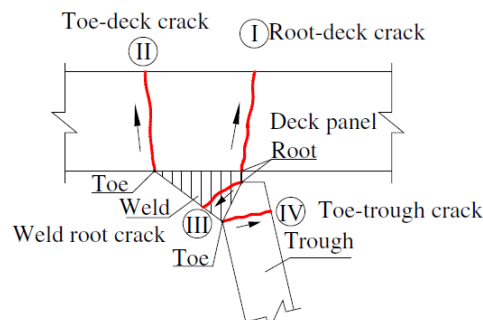

Figure 4. Fatigue cracks in rib-deck welded joint [57].

Ya [59] also investigated analytically the influence of the thickness of both the deck plate and the rib and concluded that an increase from the usual deck plate thickness of 12 mm has a noticeable positive effect on the fatigue life of the welded joint. However more recent studies are less conclusive as to the beneficial effects of an increased deck plate thickness.

Ji [57] proposed a mesh-insensitive finite element method to determine the reference stress at a check point located 6 mm from the weld root tip. Ji also showed that taking the pavement-deck interaction into consideration could help improve notably the fatigue life of *root-deck* joints.

The mechanisms behind the initiation and growth of cracks at the root of the rib-deck joint are not fully understood yet. The interaction between the weld penetration rate, deck thickness and the shape and depth of root cracks requires further attention. Kainuma [61] concluded, through fatigue tests and finite element analyses, that root gap shapes affect the direction in which the crack propagates and the initiation phase of the fatigue life. Kainuma also revealed that the weld penetration rate could prevent crack initiation but its effect on crack propagation appears to be limited.

Cracks in the deck plate of OSBDs typically originate at the root or the toe of the rib-deck joints and grow into the deck plate. Maljaars [64] has studied the fatigue performance of deck plates in OSBDs. Maljaars presents a linear elastic fracture mechanics model to predict the fatigue performance of deck plate cracks. Both experimental and analytical results document a high fatigue capacity of deck plates. It was also found that deck plates are most prone to fatigue cracks at their intersection with diaphragms and the longitudinal ribs. After an initially high crack growth rate, the crack propagates at a slower rate given that the stress at the crack tip decreases as the crack expands away from the diaphragm.

Double-sided welds are regarded as a potential technique to improve the fatigue properties of rib-deck welded joints in OSBDs. But the lack of experimental data or established design guidelines constitute good reasons to remain cautious. Fang [65] studied the eight potential failure mechanisms of double-sided welds. As shown in Figure 4, only four possible modes of failures are identified for traditional single-sided welds. Despite their higher structural complexity, double-sided welds allow a 20% reduction of the notch stress resulting in a significantly improved fatigue life. Through sensitivity analyses, Fang showed that the recommendations from [17] in terms of mesh refinement to evaluate notch stresses, originally formulated for single-sided joints, apply to double-sided joints.

5.2. Diaphragms – ribs

Several fatigue cracks at the connection between the diaphragms and the longitudinal ribs of OSBDs have been reported. Notorious examples can be found in [69] and [70].

The connection between the longitudinal ribs and the diaphragm is complex as it is affected by the flexural behavior of the rib and the out-of-plane deformation of the diaphragm. Zhang [3], through a literature review, concluded that the fatigue performance of the joints between transverse panels and longitudinal ribs is mainly affected by their stiffness ratio, the quality of welding and the geometry of the cutout details.

Diaphragm cutouts are not welded areas, but they are an area of high stress concentration and complex structural interaction. Several fatigue cracks have been reported at diaphragm cutouts.

Zhu ([71] and [72]) studied the fatigue behavior of two different types of diaphragm cutouts and found that small alterations of the cutouts could result in significant variations in the stress concentration effects and affect the fatigue life of the cutout details. Zhu studied the fatigue life of cutouts based on field measurements of both traffic, strain monitoring and FEM. Zhu concluded that the fatigue life of cutouts was governed by their geometry, the thickness of the diaphragm and the weight of the heavy vehicles. Zhu also insists on the difficulty of estimating nominal stresses at cutouts.

Zhang [73] also studied the suitability of different stress approaches. The hot spot, the effective notch and the nominal stress methods were implemented to study the fatigue strength of the welded joints between the ribs and diaphragms. The nominal and modified nominal stress approaches fail to capture the complex stress field expected at the diaphragm-rib joints. The hot spot and the effective notch stress approaches require a much more refined mesh near the areas of stress concentration than the nominal stress method, but their accuracy is significantly higher.

5.3. Splice joints – longitudinal ribs

Fatigue cracks in the butt welds joining the longitudinal ribs have been reported. Xiao [74] investigated the impact of incomplete weld penetration on the fatigue durability of butt welds joints between longitudinal ribs. Results from constant amplitude fatigue tests were compared with the findings from analyses based on the theory of LEFM. Based on the results presented by Xiao, incomplete weld penetration is the main cause of fatigue cracks at butt weld joints

6. Summary and future research

Summary of the relevant key findings and reflections regarding future work within the fatigue life assessment methods for welded joints in orthotropic steel decks of long-span bridges is presented in this section.

Fatigue pathologies in Orthotropic Steel Bridge Decks have been reported. From the observed damages, it appears that the fatigue resistance of several details (deck plate, rib-deck joints, rib-splice joints, rib-diaphragm joints, and diaphragm cutouts) require special attention.

The present paper illustrates the multitude of fatigue prediction models whose reliability is well documented in the specialized literature. The design standards currently in practice cover only some of the stress-based models. The other models mentioned in this paper have already been used to estimate the fatigue life of OSBDs and showed promising results. Clear guidelines on the selection of appropriate prediction models are absent from design standards. It appears that prediction models are chosen on the basis of engineering judgement, availability of material parameters that are required by advanced models such as continuum damage models and the fatigue regime under consideration. A unified framework would be of interest to the practicing community for the fatigue assessment of existing and future long-span bridges.

The combined effect of traffic, wind and welding residual stresses on the orthotropic steel decks of long- span bridges deserve further attention. The spectral content of the fatigue loads mentioned above differ significantly. Consequently, the effect of non-proportional loading should be taken into consideration. It is already established that some details of OSBDs experience multiaxial fatigue under

traffic loading. It could be relevant to determine whether the multiaxiality of stress is enhanced by the action of other service loads.

In a fatigue assessment procedure, multiple sources of uncertainty can affect its outcome. Reliability-based approaches are commonly used to evaluate the impact of selected sources of uncertainty on the fatigue reliability of long-span bridges. Traditionally, reliability-based analyses are carried out by means of time-consuming Monte Carlo simulations. However, recent research has proven the efficiency of machine-learning algorithms. It is the authors' opinion that the applicability of these numerical tools to the field of fatigue should be further studied.

Concern about the limitations of the Palmgren-Miner rule based on the linear accumulation of damage has been raised. Given the complex nature of the loads acting on a long-span bridge, it seems essential to consider alternative damage accumulation indexes.

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