

Using fracture mechanics principles in steel bridge renovation projects

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Steel bridges built in the post war period were not built for the current traffic intensities and are therefore prone to fatigue damage. The combination of high traffic intensities, low quality steel and suboptimal weld detailing can generate fatigue problems in existing bridges that could compromise the structural integrity of the bridge. Fracture mechanics principles can be used in steel bridge renovation projects, when S-N based calculation procedures do not predict sufficient structural capacity, as an alternative to potentially unnecessary strengthening of the considered detail. These principles have been used in the renovation of a 231 m long, plate-girder bridge in The Netherlands. Inspection intervals were derived for a non-load carrying, double fillet welded connection between transverse stiffeners and an I-shaped main girder. Such an analysis requires a detailed insight of the material's condition and therefore should be completed in close cooperation with the inspection operator to verify calculation assumptions. Using fracture mechanics allows the bridge owner to choose between inspecting a detail at a certain frequency or completing preventative strengthening.

1. Introduction

The Managing Contractor (MC), a combination of Royal HaskoningDHV, Arup, and Greisch was appointed by Rijkswaterstaat, The Dutch national bridge authority, in 2009, to refurbish 8 long span steel bridges in The Netherlands. The scope of the MC project is to recalculate the capacity of the existing bridges and to design strengthening schemes to increase the capacity of the bridges for both fatigue and static strength. The MC has also a leading role in the procurement and construction management.

The bridges in the contract were built between 1969 and 1990, during a period of rapid infrastructure development. They were not designed to withstand the current traffic intensities and are therefore prone to fatigue damage. The span length of the bridges varies between 100 and 300 m and they are all located in strategically important locations in the Dutch highway network. All 8 bridges have steel orthotropic decks, with trapezoidal longitudinal stiffeners, but have different types of superstructure e.g. tied-arch, beam-girder and cable-stay. Cracks in the deck plates have been observed since 1997. Additionally, certain of the bridges have other fatigue problems in the deck and superstructure.

To assess the fatigue capacity of the bridges, the MC used fatigue load models that were derived during traffic measurements on Dutch highway bridges, discussed in [1] and implemented in NEN8701 [2]. These loads, in combination with a detailed finite element model of the deck and the experimental results



of [3] were used to calculate the predicted fatigue damage in the orthotropic deck. The bridge decks were strengthened with a High Strength Concrete (HSC) overlay initially developed as part of [4]. The asphalt is replaced during the renovation with this new, stiff overlay, reducing the stress in the deck plate and increasing the fatigue life of the orthotropic deck.

The MC used the same fatigue load model, in combination with a finite element model of the entire bridge, to determine the fatigue capacity of the welded joints in the superstructure of the bridges. The derived stress ranges were then used in a S-N based approach, using detail categories from [5]. Where insufficient fatigue capacity was predicted, measures to guarantee the structural safety of the superstructure were developed. The corresponding structural upgrading can cause excessive hindrance for the traffic using the bridge and the surrounding infrastructure, i.e. roads, railway or waterways, during the execution of the works and could also be expensive.

An alternative solution is presented in this paper which is based on fracture mechanics and inspections to demonstrate adequate structural capacity of the bridge without any structural upgrading of the bridge.

2. Fracture mechanics principles

Fracture mechanics assesses the propagation of cracks within structures. These cracks, or flaws, can be attributed to various processors including: the welding practice, voids produced during the production process of steel or propagating fatigue cracks due to cyclic loading conditions. In most cases a Linear Elastic Fracture Mechanics (LEFM) based approach is used to assess these existing cracks. This method is valid as long as the plastic zone around the crack tip is relatively small. This approach uses the Stress Intensity Factor (SIF) which defines the crack tip conditions and relates internally or externally applied stress states to the region of the structure containing the flaw. The SIF is an important parameter in LEFM and extensively discussed by Anderson [6].

Fracture mechanics can be used to determine the fatigue crack growth (da) per stress induced cycle (dN) using the SIF approach for cyclically loaded structures, such as bridges. The most simple, and frequently applied crack growth law is the one derived by Paris and Erdogan [7], shown in Equation 1. This equation uses material parameters (C and m) which can be found in guidelines, such as BS7910 [8]. This guideline also includes parametric equations for the SIF (ΔK), as a function of the load configuration, crack geometry and flaw size.

$$\frac{da}{dN} = C \Delta K^m \quad (1)$$

A second application of LEFM is to determine the critical flaw size before failure is predicted under given static loading conditions. This procedure often uses a Failure Assessment Diagram (FAD), which relates the SIF to the fracture toughness of the material (the fracture ratio) and the static stress level to the yield stress (the load ratio) to determine the acceptability of a flaw. The flaw is deemed to be unacceptable whenever the combination of the fracture and load ratio exceeds the FAD assessment line.

A key step in this approach is the Probability of Detection (PoD) of the applied Non Destructive Testing (NDT) technique. The PoD is defined as the capability of a particular NDT technique to find a crack like flaw, of a particular size, during inspections. The PoD curves can be based on a significant amount of testing and are included in certain fitness-for-service guidelines, e.g. BS7910 [8].

Fracture mechanics can be used to determine the fatigue design life of a structural member containing a flaw until failure (i.e. fast fracture or plastic collapse) is predicted. The fracture resistance assessment determines after each crack increment (da) if the flaw size is acceptable or not using the FAD. Based on the outcome of this procedure, a certain inspection frequency can be calculated which allows for

sufficient time for taking measures if a crack is found on site. This approach can be utilised even when a S-N based approach results in predicted fatigue damage exceeding one.

3. Case study: Suurhoff bridge

A fracture mechanics analysis was carried out for the Suurhoff bridge, which is one of the 8 bridges within the MC project. The Suurhoff bridge is a 231 m long beam-girder bridge with an orthotropic, steel deck, spanning the Hartel canal in the Port of Rotterdam area. This bridge was open for traffic in 1972 and carries 4 lanes of traffic of the A15 motorway.

The original contract requirement for the Suurhoff bridge was to strengthen the steel deck and superstructure to allow for a remaining service life of 30 years. The HSC overlay is however less effective for this bridge as the longitudinal, trapezium shaped stiffeners are discontinuous and welded in between the cross beams. Fatigue cracks are therefore expected in welds for which the stress range is not significantly reduced by the HSC overlay. Moreover, the increase in self-weight due to the application of HSC introduces a static capacity problem in the superstructure. Therefore the cost to strengthen the bridge for the full 30 year life extension is prohibitively expensive and would cause significant traffic hindrance.

Rijkswaterstaat has decided to replace the Suurhoff bridge in the future, but in the meantime, the functionality and structural safety of the bridge needs to be maintained. The necessary structural upgrading should be limited, because the bridge will be replaced. All connections with insufficient fatigue capacity have been identified and the MC has looked for solutions that are cost effective and do not cause excessive hindrance for traffic or the underlying canal.

The Suurhoff bridge contains two I-shaped main girders with cross beams welded to the main girder, typically spaced at 4.54 m. Transverse stiffeners are welded by means of two fillet welds to the web and bottom flange of the main girder at each cross beam location, shown in Figure 1. The transverse stiffeners increase the capacity against lateral torsional and local plate buckling of the main girder and are positioned either side of the main girder web at the location of interest. A fatigue crack initiating at the toe of the weld and growing into the main girder web or bottom flange could compromise the structural integrity of the bridge. Fatigue damage exceeded one was predicted at this detail using a S-N based approach, implying insufficient fatigue capacity.

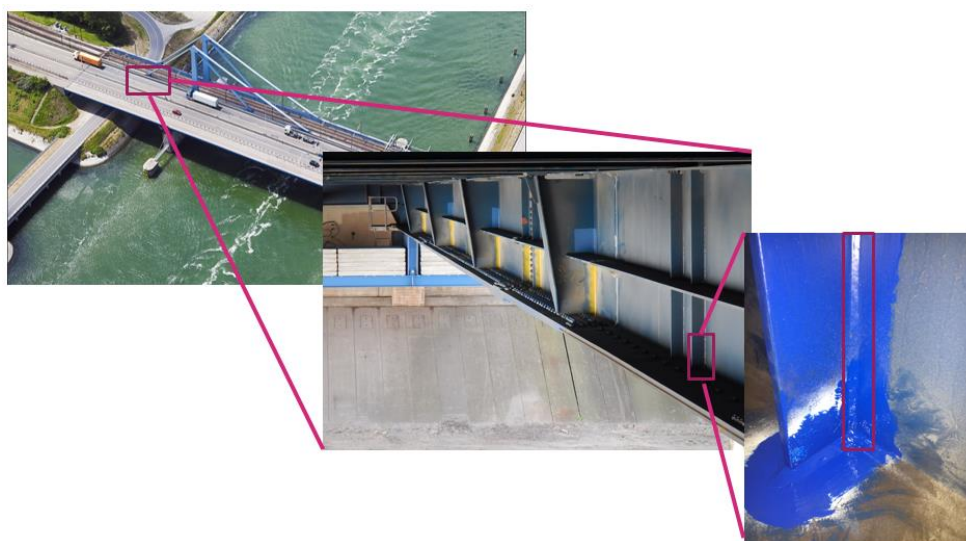


Figure 1. Encircled double fillet welded connection between transverse stiffeners and the I-shaped main girder, zoomed in to illustrate the location of the welded details in the bridge.

It was decided to inspect these connections for the remaining service life of the bridge until it is replaced. The inspection frequency was derived using fracture mechanics; a combination of fatigue crack growth and fracture as outlined in Section 2. This approach corresponds to a damage tolerant design philosophy, which requires a partial safety factor of 1.15 for the bridge superstructure [5]. This partial safety factor is normally applied on the resistance side, i.e. to the detail category, but has been applied to the stress ranges in the fracture mechanics approach. The calculation procedure was carried out for a connection between a transverse stiffener and the web of the main girder, referred to as 'detail I2' in the remainder of this paper, located in the bridge as shown in Figure 1.

Two inspection techniques were chosen to inspect the fillet welded connections between the transverse stiffeners and main girder; Time of Flight Diffraction (TOFD) and Magnetic Particle Inspection (MPI). TOFD uses ultrasonic principles to detect a flaw and is capable of providing crack length and depth information of planar flaws in a structure. MPI can detect surface breaking flaws in ferromagnetic materials, measuring the crack length of flaws only. MPI was chosen to verify whether indications picked up by TOFD, were surface breaking fatigue cracks, initiating at the weld toe.

The minimum flaw size that can be reliably detected using TOFD, when inspecting from the opposite side of the main girder web or bottom flange, is 2 mm deep by 10 mm long [8]. The position of the TOFD probes for a fatigue crack growing into the main girder web is shown in Figure 2. This detection threshold is determined using a probabilistic approach with PoD curves. This threshold indicates that a fatigue crack slightly smaller than 2 mm x 10 mm could remain undetected by an operator during a TOFD inspection. This therefore represents the initial flaw size, or starting flaw size, for the fracture mechanics assessment. The flaw size length that can be reliably picked up using MPI is 10-20 mm for as-welded joints [8], dependent on the weld quality and surface roughness.

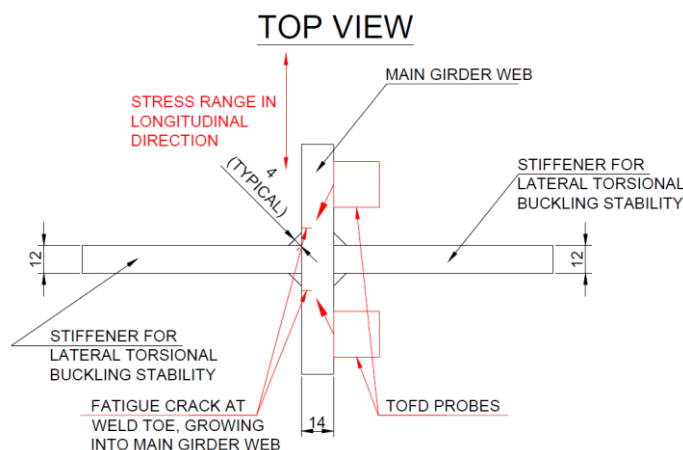


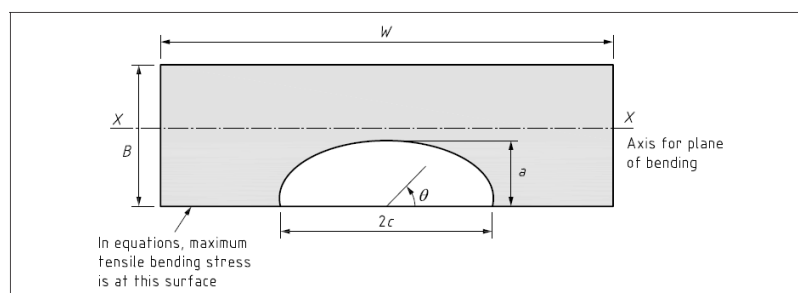
Figure 2. Fatigue crack initiation location for detail I2 and indicating the position of the TOFD probes during inspections

A two-staged crack growth law has been used for the fatigue crack growth calculation. This crack growth law accounts for decelerated growth of cracks in the near-threshold regime and uses two different sets of crack growth parameters (C and m , Equation 1). The usual practice for fatigue assessments of welded joints is to neglect the mean stress effect and to assume high, through-thickness, tensile residual stresses are present, generated by the welding process. This results in the simplification that the entire stress range ($\Delta\sigma = \sigma_{\max} - \sigma_{\min}$) is effective in terms of fatigue crack growth, not a function of the mean stress and the stress ratio ($R = \sigma_{\min} / \sigma_{\max}$). The crack growth parameters, corresponding to $R \geq 0.5$ and mean plus 2 times the standard deviation were taken from BS7910 and are shown in Table 1. A fatigue crack growth threshold, relating to $R \geq 0.5$ and equal to $63 \text{ MPa}\sqrt{\text{mm}}$ was also applied [8].

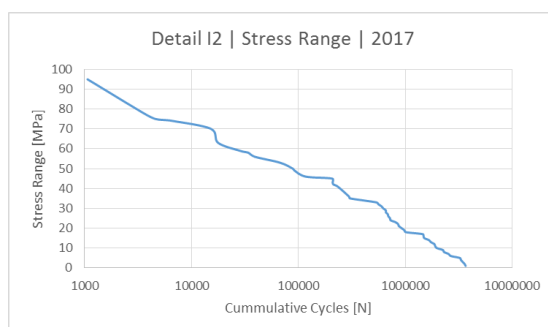
Table 1. Crack growth parameters for a two staged crack growth law

Crack growth parameter	Stage A	Stage B
C	2.10E-17	1.29E-12
m	5.10	2.88

The fatigue crack initiates at the weld toe for detail I2 and grows into the main girder web (da/dN) and along the weld toe profile ($d2c/dN$). This surface breaking flaw is often simplified as a semi-ellipsoid, shown in Figure 2. Annex M, Section M.4 of BS7910 [8] contains parametric equations for surface breaking flaws, applicable for membrane and local bending stress conditions. Local bending over the plate is considered negligible for this detail. The membrane stress contains the axial and bending stress components over the strong axis of the I-section main girder, these induce a stress range perpendicular to the crack (Mode 1 loading). Shear stresses have been included for detail I2, as recommended in [5] because the stiffener ends in the web of the main girder. The shear stress components have been added to the total membrane stress in which mixed mode loading has been disregarded. Parametric equations for the M_k factor, that represents the stress raising effect caused by the weld profile, were taken from Section M.11.1.3 of [8].

**Figure 3.** Semi-elliptical surface breaking flaw [8]

The Palmgren-Miner linear damage summation rule, used in S-N based approaches, disregards load sequence effects. A certain stress range results in a predicted damage, irrespective of the current crack length. Also the cut-off limit is not dependent on the current crack size. This simplification is not valid in fracture mechanics based analyses. Fatigue crack growth is only observed when the SIF, which is a function of the current crack size, exceeds the fatigue threshold. The fatigue load model of NEN8701 [2] includes various time-dependent factors, including: vehicle configuration, increase of heavy traffic numbers per year and an estimated axle load increase in the future. The stress ranges were therefore randomly ordered per year and applied in the fatigue crack growth assessment. Figure 4 and 5 show the stress ranges for detail I2 in 2017 and the Finite Element Model (FEM) used to derive the stress history.

**Figure 4.** Stress history for detail I2 in 2017**Figure 5.** FEM of the bridge

The fracture assessments are carried out at the lowest possible service temperature of the bridge, assumed to be -20°C in The Netherlands. This is because steel becomes more brittle at low temperatures, resulting in a lower fracture toughness. An accidental load combination [9], to combine (quasi-)static loads, may be used for fracture assessments due to the low probability of the combined occurrence of a low service temperature, large critically positioned flaws, poor material properties and onerous combination of actions [10]. The static stress level, using the accidental load combination for detail I2, is equal to 129 MPa.

A steel grade of Fe 510 D3 was used during the original construction of the bridge, this corresponds well with a modern S355 steel grade. The FAD assessment line has been created assuming a yield discontinuity in the stress-strain curve, a yield and tensile strength of 355 and 510 MPa respectively and a modulus of elasticity of 210000 MPa. The fracture calculation procedure verifies, for every crack increment (da) in the crack growth procedure, whether or not the total flaw size exceeds the assessment line which could correspond to failure of the remaining ligament.

NEN-EN 10025-2 [11] contains technical delivery conditions of structural steels, but these were not applicable during the original construction of the bridge. These standards require a minimum Charpy impact value of 27J at -20°C for low carbon steels. Other delivery standards were applicable in the late 1960's, and required comparable Charpy impact values, this value was therefore assumed in this assessment. The fracture toughness was calculated based on this minimum Charpy impact value using the master curve presented in Annex J of BS7910 [8]. A tensile residual stress distribution of yield magnitude (355 MPa) was also assumed for calculating the fracture ratio. The SIF equations for a semi-elliptical surface breaking flaw were used with the static stress level as input and the Mk factors based on 3D FEM calculations [8]. The load ratio relates the static stress level to the yield stress of the steel grade, accounting for the reduction in cross sectional area due to the propagating crack, using the reference stress equations in Annex P of BS 7910 [8].

An initial surface breaking flaw of 2 mm x 10 mm in the main girder web breaks through thickness for detail I2 in less than 5 years as shown in Figure 6. Brittle fracture is however predicted at smaller crack sizes using the fracture assessment, shown in Figure 7 and Table 2. The fracture and load ratio are a function of the crack size and exceed the FAD assessment line at a flaw size of 4.7mm by 21.8mm when fracture is considered in the through-thickness (a-)direction for a fracture toughness based on 27J. This crack size is predicted to be reached after 41 months, by using Figure 6. A less stringent critical flaw size is predicted for the (2c-)direction along the weld profile.

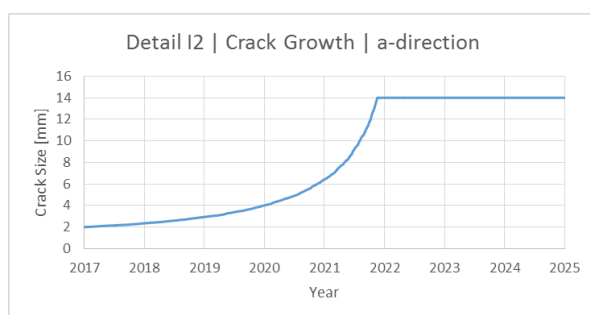


Figure 6. Through-thickness fatigue crack growth rate for a surface breaking flaw

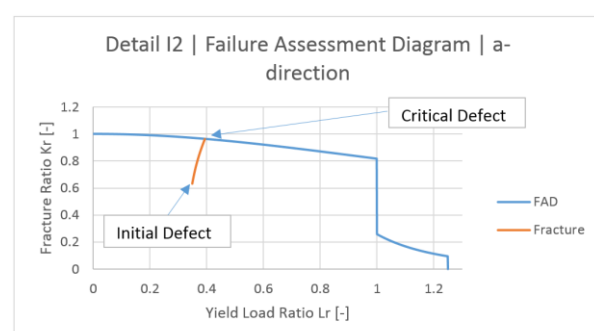


Figure 7. FAD for a surface breaking flaw fatigue crack growing through thickness

Table 2. Remaining fatigue design life of a surface breaking flaw for detail I2

	Charpy impact value [in J]	Fatigue Life [In Months]	(a) at brittle failure [In mm]	(2c) at brittle failure [In mm]
Brittle Fracture a-direction	27	40.9	4.741	21.772
Brittle Fracture 2c-direction	27	46.5	5.970	25.675

Based on the fracture mechanics analyses it is recommended that the details are inspected every two years with a back-surface TOFD procedure, combined with MPI. This provides Rijkswaterstaat sufficient time to react in case a fatigue crack is found during the inspections and to prepare necessary precautions. The inspection frequency also accounts for the fact that 1. the remaining fatigue life decays if for example 2023 is used as a starting point, due to the expected heavy traffic number and axle load increase and 2. the 27J Charpy impact value has not been proven by testing on heat affected base material. If cracks are found during inspection suitable measures could include repair welding the fatigue crack, by-passing the fatigue crack with a bolted splice connection or using burr grinding as post weld treatment to remove any historic damage and to improve the detail classification.

The assumptions taken for this analyses could be refined to predict a less stringent inspection frequency. Sensitivity results are given in Table 3 for various Charpy impact values and residual stress magnitudes. The fracture toughness can be measured by taking samples of the bridge. Residual stresses can be measured using various experimental techniques or studied numerically using non-linear finite element analysis. Arup is currently developing numerical welding simulations that can, amongst other things, predict residual stresses to provide benefit in asset management of existing, steel structures.

Table 3. Sensitivity results to determine the remaining fatigue life for detail I2

Residual Stress [in MPa]	Charpy impact value [In Joules]	Fatigue life [In Months]	(a) At brittle fracture [in mm]	(2c) at brittle fracture [in mm]
355	27	40.9	4.741	21.772
355	50	47.4	6.272	26.620
275	27	48.4	6.509	27.382
200	27	53.5	8.823	34.948
200	50	55.5	10.403	40.608

NEN-EN 1993-1-10 [10] and its background document [12] also provides guidance for deriving inspection intervals for joints that have expired their fatigue service life. The crucial assumption in the derivation of 'safe service periods' using this design standard is that the virgin, as-welded, situation may be assumed once the inspection has been carried with no fatigue cracks found. This approach does not assume that a crack may be left behind in the structure due to detection capabilities of the applicable inspection technique. This assumption may result in too optimistic inspection intervals and could potentially compromise the structural safety of the bridge.

4. Conclusion

The design life of existing steel bridges exposed to cyclic loading conditions induced by traffic is often assessed using a S-N based approach provided by various guidelines and standards. The outcome of such an analysis is a theoretical, predicted damage number which only indicates whether or not the fatigue capacity of the joint is according to standards used. This doesn't yield information to determine the inspection regime. Predictions of insufficient fatigue capacity can result in structural upgrading which is often expensive, could cause hindrance for traffic and/or underlying infrastructure and could be unnecessary due to model assumptions and conservatism.

An alternative approach is to derive bespoke inspection intervals using fracture mechanics and to inspect the joints predicted to have insufficient capacity. Sufficient time should be included in the inspection frequency for the bridge owner to prepare actions in case of detected fatigue cracks. Furthermore the measures to be taken when a crack is detected should be known on forehand. This alternative approach for joints with insufficient fatigue capacity is less robust than implementing a comprehensive strengthening, but could be considered a more cost effective solution for the bridge which reduces traffic hindrance. This option is especially relevant for bridges with a relatively short remaining service life.

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