Crack propagation in a stiffener-to-deckplate connection of an orthotropic steel bridge deck

Wim Nagy[1,a], Hans De Backer[1,b] and Philippe Van Bogaert[1,c]

[1] Ghent University, Dept. of Civil Engineering

Technologiepark 904, 9052 Zwijnaarde, Belgium

[a] Wim.Nagy@UGent.be, [b] Hans.DeBacker@UGent.be, [c] Philippe.VanBogaert@UGent.be

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Abstract. Orthotropic decks are an indispensable structural element in long span steel bridges. This type of bridge deck consists out of a complex network of longitudinal stiffeners and transverse stiffeners and is extremely lightweight, durable and efficient for carrying large traffic loads. Due to the important number of welding details, it is obvious that these bridges are very sensitive to fatigue damage. In addition, traditional fatigue analysis of these bridges has now reached its limits which leads to overestimating the necessary dimensions. A possible design improvement could be the use of fracture mechanics since this is a more in-depth analysis. In the fatigue details of orthotropic bridge decks, micro cracks are caused by the parent material, the geometric discontinuities, as well as to the welding operations. Therefore, the duration of the crack initiation phase in these decks is significantly reduced. When focussing on the stiffener-to-deckplate detail, the question remains how fast the crack will grow and if it will grow through the deck plate, through the weld throat or through the trough stiffener. To investigate this problem, a FEM-model is developed with the finite element software Samcef. In this paper, crack propagations are studied in a stiffener-to-deckplate detail of the Temse bridge in Belgium where fatigue cracks occurred and progressed rapidly. Approximately $38{,}5.10^6$ load cycles were needed for a crack length of 600 mm. This is more than the traditional calculations would predict because the FEM-model does not take residual stresses and other welding defects into account. This could be a possible improvement in future research.

Introduction

Orthotropic plated decks are widely used in long span steel bridges since they are extremely light weighted and very efficient for resistancing localized traffic loads. As this bridge deck consists of a complex network of longitudinal and transverse stiffeners, it is obvious that these bridges are very sensitive to fatigue damage. Nowadays fatigue behaviour of steel bridges is mainly analysed by applying SN-curves as well as through the Palmgen-Miner hypothesis. However, there is a need for relevant test data for the used geometrical conditions and welding configurations. In addition, the background tests for the SN-curves do not exclusively use present construction technology, resulting in a conservative approach and thus leading to an overestimation of the necessary dimensions. Furthermore, classical fatigue calculations assume damage accumulation to be a strictly linear phenomenon, which implies that the load sequence has no effect. Questions have been raised in recent research about the validity of classical fatigue philosophy. A possible design improvement could be the use of fracture mechanics since this is a more in-depth analysis.

This fatigue design method is well known in the automobile, marine and aeronautics industries. Unfortunately it is not frequently used in civil engineering since it is a more complex and labour-intensive method. Fracture mechanics can deal with fatigue crack initiation, crack propagation and

subsequent failure of the structure. Therefore, the method could be used for the lifetime prediction of orthotropic decks and the remaining strength of existing steel bridges.
In this research project, the focus is on the fatigue problems of the stiffener-to-deck plate connection mid-span between two crossbeams. This was also the location of a fatigue problem in the Temse bridge across the river Scheldt in Belgium. To verify the problem, a comparison is made between the classical fatigue analysis and the fracture mechanics method.

Fatigue of welded connections

Generally fatigue can be seen as cracks developing until complete fracture after a sufficient number of stress fluctuations. The lifetime of the structure then consists of three subsequent stages: fatigue crack initiation, fatigue crack propagation and final fracture.
The fatigue crack initiation stage is defined as the period of crack propagation from an initial defect until the first detectable crack. According to [1], this stage can be subdivided into basically two subsequent phases: microstructural and mechanical (Fig. 1). In the first phase, the crack has to conquer microstructural barriers such as twin or grain boundaries. Only when the applied stress range is sufficiently high, the crack will continue to extend. Once the crack encloses a number of grains, it is becoming mechanical. This is a more steady crack propagation stage. However, the crack still does not behave by the linear elastic K factor because of crack closure effects.

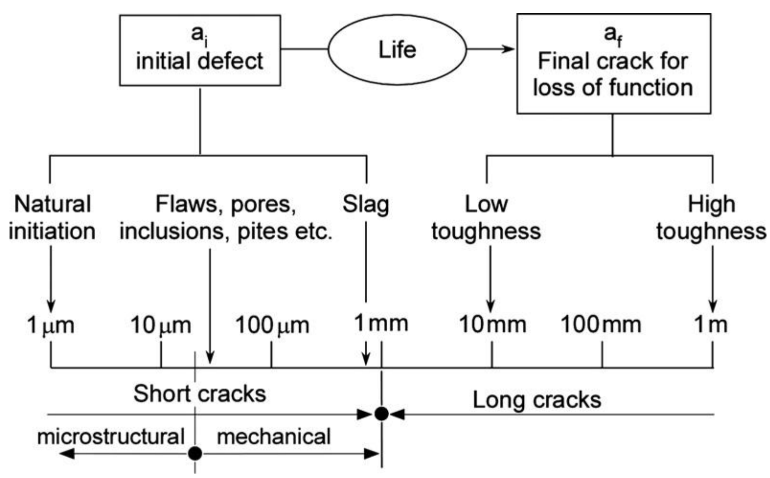
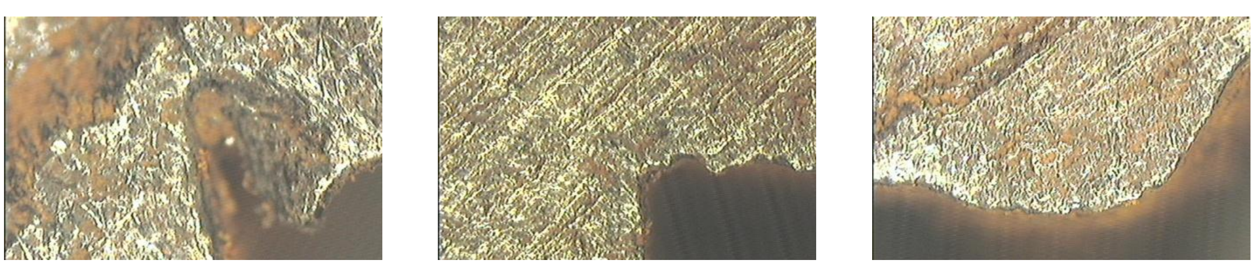


Fig. 1: Length scales of the life cycle of a component subjected to cyclic loading (according to [1])

This crack initiation phase in unwelded components can be very large compared to the total fatigue life. In welded connections, this crack initiation stage is relative small due to the welding defects, stress concentrations and the residual stresses. Lack of penetration, lack of fusion, slag inclusions and porosity are some of the possible welding defects. It is obvious that welds cannot be perfect and will always contain initial micro cracks which decrease the total fatigue crack initiation stage.



(a) Lack of penetration (b) Full penetration (c) Excessive weld penetration

Fig. 2: Possible welding defects

In addition, the period of crack initiation in smooth specimens without defects is less than 5-20% of the total fatigue life [2]. In materials containing defects, the crack initiation period is even smaller. For this reason, the crack initiation phase in welded connections is mostly neglected. In the past years various technologies were studied to improve the fatigue performance of welded joints [3]. Therefore an important benefit can be realized concerning residual stresses. However, remaining residual stress can raise the effective stress intensity factor and thus make a large factor for the crack initiation process [4].

In Belgium, the welds of a stiffener-to-deck plate in an orthotropic bridge deck comply with EN 1993-2: 2006/AC:2009 [5]. This implies that the edges of the closed stiffeners are not prepared (machined). In case these edges of the troughs are prepared (manual and partially machined), the butt welds must be chamfered and the lack of penetration is limited to a maximum of 2 mm. For these types of connections, the possible cracks paths are through the deck plate starting from the weld root (Fig. 3: (a)), through the deck plate starting from the weld toe (Fig. 3: (b)), through the stiffener (Fig. 3: (c)) or through the weld throat (Fig. 3: (d)).

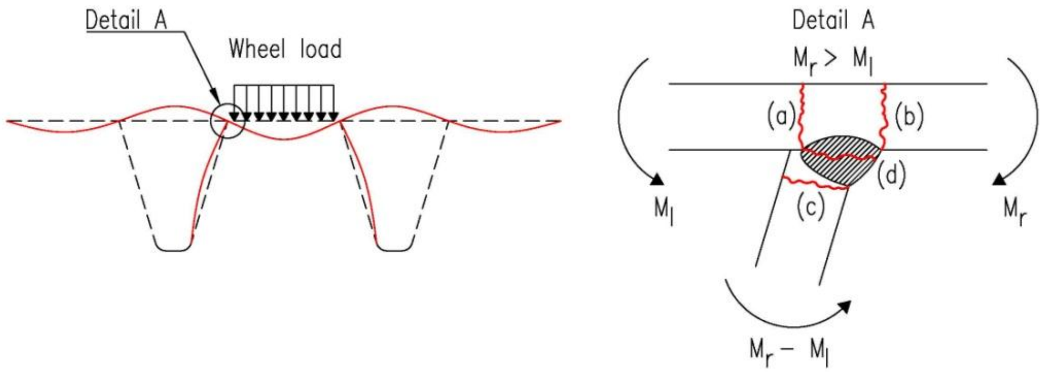


Fig. 3: Schematic deformation under wheel loading and the corresponding crack directions

Case study: Temse bridge across the river Scheldt

Fig. 4: Movable part of the Temse bridge across the river Scheldt (Belgium)

Introduction. An example of a fatigue crack propagation in an orthotropic steel bridge deck is the Temse bridge across the river Scheldt in Belgium. This bridge is part of an important transport route for heavy lorries to the Port of Antwerp. The Temse bridge was built in 1955, the movable part having initially an aluminium deck plate. After many repairs, the movable part however was rebuilt in 1994 using a steel orthotropic plated deck.

The deck plate of this movable part of the bridge is 12 mm thick and the stiffeners are 8 mm thick. The overall dimensions of the traffic lane are 53,90 m by 7,00 m. The trapezoidal stiffeners are 350 mm high and 300 mm wide on top and 90 mm width of the lower soffit. The distance between the longitudinal stiffeners equals 300 mm.
In 2004, a 60 cm long crack was detected in the stiffener-to-deck plate detail mid-span between two crossbeams (Fig. 5, Fig. 6). The main reason for this crack is the occurrence of multiple welded joints intersecting each other [6]. In addition, these welds were not chamfered and even the prescription of lack of penetration can be questioned here. In this particularly case, the crack propagated through the deck plate starting from the weld root. Because of the proportional propagation of the crack front due to the high traffic loads, which occurs close to the crack front, the crack has been repaired [6].


Fig. 5: Longitudinal crack through the deck plate at a stiffener-to-deck plate connection


Fig. 6: Multiple welds intersecting at the crack location

Analytical study: an alternative approach with fracture mechanics techniques. To investigate the fatigue problem for this particularly case of the Temse bridge, an alternative approach of fatigue life predictions has been proposed. Traditionally fatigue problems of civil constructions are being studied through the Palmgren-Miner hypothesis and SN-curves. For orthotropic bridge decks, fatigue analyses result in a conservative approach since the load sequence does have an effect on the total fatigue life of the structure. In addition, the SN-curves do not always represent the true cut-off-limit of the detail being studied. The alternative approach could be the use of fracture mechanics. This method has already shown its reliability in the aerospace and automobile industry but never really found its way in civil engineering. One of the advantages of this method, is that there is no need for relevant test-data except for the parameters C, m, K_{th} and K_{IC} in the Paris law. But they are only material dependent, so less test-data is needed for a wide variety of fatigue problems. In addition, even load histories and load sequences can be studied. However, this method is more complex and labour-intensive since it has a need of a FEM-model. Depending on the detail being studied, this model could be very large which has an effect on the total computation time.

Analytical study: the FEM-model. To investigate the fatigue problem in a stiffener-to-deck plate connection mid-span between two crossbeams, a FEM-model was developed in the finite element software Samcef. The software can deal with different crack propagation simulation techniques such as the XFEM (eXtended Finite Element Method) method. With the XFEM method, the crack growth can be assessed without re-meshing the model in the vicinity of the crack. For the computation of the stress intensity factors in a 3D field, the software uses the calculation of a so called interaction integral J [7]:

$$J = J^{(a)} + J^{(b)} + I^{(a,b)} \tag{1}$$

In this technique, particular values of the J-integral are used that include two stress states:

- the real one (a):

$$\begin{cases} \sigma_{11}^{a} = \sigma_{11}^{real} = FEM\ solution \\ \sigma_{22}^{a} = \sigma_{22}^{real} = \ldots \end{cases} \tag{2}$$

- the auxiliary ones (b):

$$\begin{cases} \sigma_{11}^{b} = \sigma_{11}^{aux} = \dfrac{K_I^{aux}}{\sqrt{2\pi r}} \cos\dfrac{\theta}{2}\left(1 - \sin\dfrac{\theta}{2}\sin\dfrac{3\theta}{2}\right) + \dfrac{K_{II}^{aux}}{\sqrt{2\pi r}} \cos\dfrac{\theta}{2}\left(2 + \cos\dfrac{\theta}{2}\cos\dfrac{3\theta}{2}\right) \\ \sigma_{22}^{b} = \sigma_{22}^{aux} = \ldots \end{cases} \tag{3}$$

The auxiliary ones correspond to the asymptotic fields of modes I and II. Finally the classical term of the interaction integral $I^{(a,b)}$ is given by:

$$I_i^{(a,b)} = -\int_A \theta_{m,j} \left(\sigma_{kl}^{(a)} \varepsilon_{kl}^{(b)} \delta_{mj} - \sigma_{lj}^{(a)} u_{l,m}^{(b)} - \sigma_{lj}^{(b)} u_{l,m}^{(a)} \right) dV \tag{4}$$

For 3D problems, the interaction integral of each point i of the crack front is compared to the energy release rate. By doing this, the stress intensity factors can be evaluated.

$$I_i^{(a,b)} = G_i^{(a,b)} \int_{\Gamma_i} \theta ds \tag{5}$$

$$G^{(a,b)} = \frac{2(1-\nu^2)}{E}\left(K_I^{(a)} K_I^{(b)} + K_{II}^{(a)} K_{II}^{(b)} \right) + \frac{1}{\mu} K_{III}^{(a)} K_{IIII}^{(b)} \tag{6}$$

The advantage of using the J-integral and thus using ΔJ for evaluating ΔK consists in the ability to prescribe short crack propagation and long crack propagation as well [1]. The fatigue crack growth can be simulated through many crack propagation laws. In this case, the traditional Paris law is used:

$$\frac{da}{dN} = C\left(\Delta K_I^{eff}\right)^m \tag{7}$$

The stress intensity factor ΔK_I^{eff} is a function of ΔK_I, ΔK_{II}, ΔK_{III}, θ_P (angle of Sih or bifurcation angle) and the used material. The parameters C and m are also material dependent. For ferritic-pearlite structural steels, C is equal to $6{,}9.10^{-12}$ and m is equal to 3 (da/dN has units of m/cycle and ΔK is in $MPa.m^{1/2}$) [8].
To build up the FEM-model, the whole movable bridge is modelled with shell elements and beams (Fig. 7).

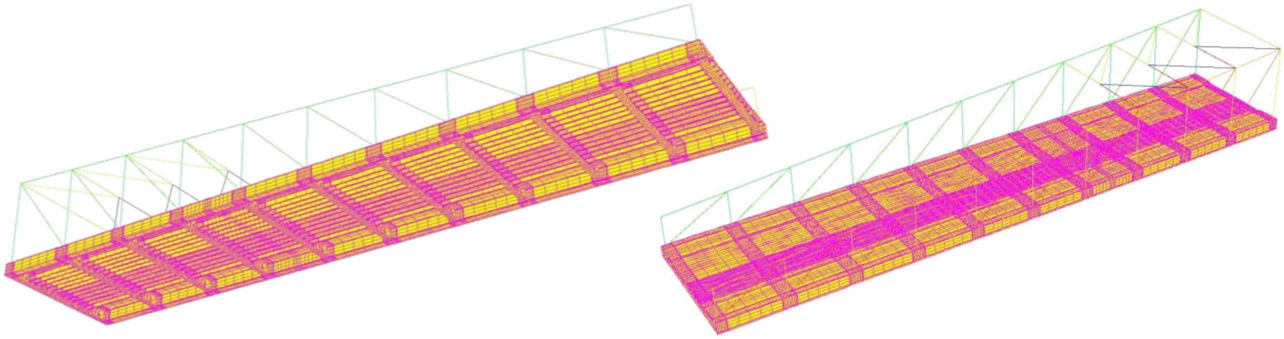


Fig. 7: FEM-model of the Temse bridge

At the crack location a part is cut out the model and replaced by volume elements for the XFEM calculations (Fig. 8). The load consists of a 90 kN axle load on a wheelprint type C (EN 1991-2:2003/AC:2010 [9]) . The position of this wheel load is given in Fig. 3. Concerning the initial crack length, a 1 mm semi-elliptical crack length has been chosen as a possible expected weld defect in the Temse bridge. This choice also relates to the computating time. The location of this initial crack length is chosen at the stiffener-to-deck plate connection inside the left through of the left wheel load (Fig. 3).

Results. During a first phase, linear elastic calculations of the uncracked bridge are carried out. This results in a maximum hot spot stress of 43,41 MPa and a fatigue life of $41{,}59.10^6$ cycles. Obviously this is only true in the case of the assumed 90 kN axle loads. Knowing that each day 2800 lorries are passing across the Temse bridge [10], the load cycle history in the past 10 years (1994-2004) is approximately:

$$N = 10\,years \cdot 250\,days \cdot 3\frac{axles}{lorry} \cdot 2800\frac{lorries}{day} = 21 \cdot 10^6 \qquad (8)$$

This indicates that according to the traditionally fatigue calculations, already 50% of the total fatigue life is reached, which can be true based on the observation of the progressive propagation of the crack.

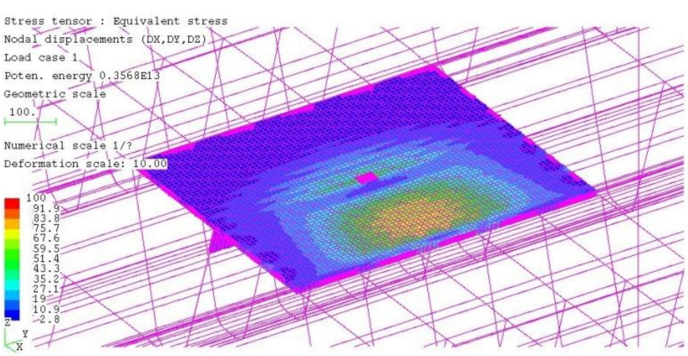


Fig. 8: Stresses in the 3D volume elements for the XFEM calculations due to an axle load of 90 kN

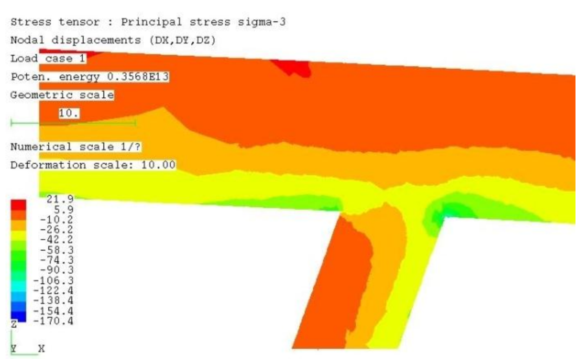


Fig. 9: Stresses in the weld detail without any cracks

If calculations are repeated with the insertion of a 1 mm crack in the model, the crack seems to propagate through the weld throat and not through the deck plate. At first, the crack propagates longitudinally along the outer surface of the stiffener before fully penetrating the weld throat. This is also seen in Fig. 10 where a comparison is made between the longitudinal crack propagation and the propagation through the weld. After $3{,}7.10^6$ cycles, the crack propagates 22,30 mm in the longitudinal direction. This contradicts the propagation through the weld throat where a propagation of 5,65 mm can be found after $3{,}69.10^6$ cycles.

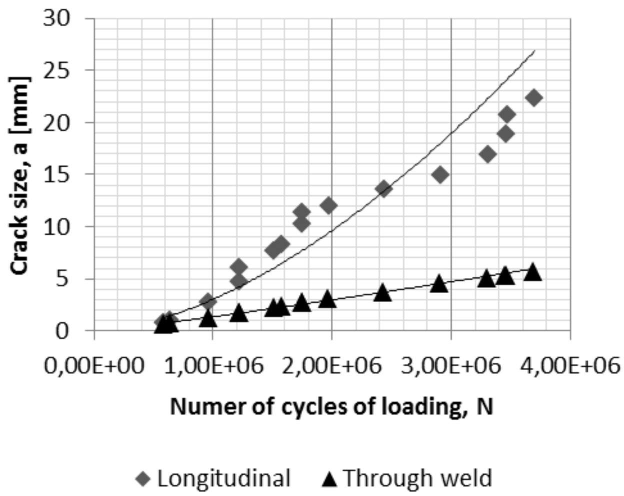


Fig. 10: Comparison of the longitudinal crack propagation and the propagation through the weld

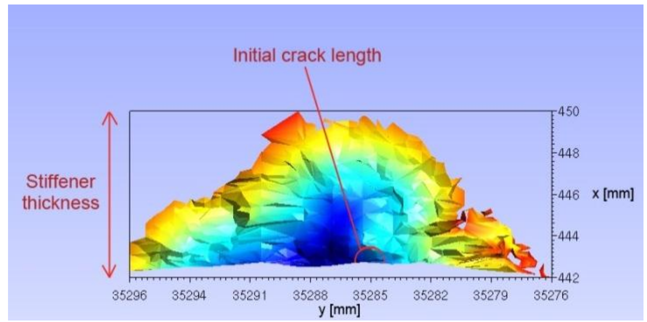


Fig. 11: Crack front dimensions

Fig. 11 displays the simulated crack front after $3{,}7.10^6$ load cycles. The y-axis shows the longitudinal coordinates in mm along the stiffener-to-deck plate connection in the whole model, while the x-axis shows the coordinates in mm of the stiffener thickness. At this time, it is not possible to simulate the crack until a crack length is found like the Temse bridge of 600 mm due to the heavy computation time. The XFEM method is a less labour-intensive tool to simulate crack growth, but it has a need for a very course mesh in front of the crack. Otherwise, the result loses its accuracy.

If an exponential extrapolation is made of the results of Fig. 10 the crack will reach a length of 600 mm after $38{,}5.10^6$ load cycles. This is more than the indicated 21.10^6 load cycles in the traditional fatigue assumptions, but it must be remembered that the model does not take into account the residual stresses. These could be high in the detail of the Temse bridge because of a lot of welds intersecting each other at the crack location. Also, the welds are not chamfered or treated at the surface to reduce any residual stresses. In addition, the weld material is homogenous in the FEM-model without any other defects than the inserted crack. Hence, a different crack propagation then observed in the Temse bridge is perfectly possible.

Conclusions

The use of fracture mechanics as an alternative approach to the traditional fatigue calculations has many advantages. There is no need for relevant test-data except for the parameters C, m, K_{th} and K_{IC} in the Paris law. This means that the fracture mechanic techniques method makes it possible to evaluate every detail separately, this being less conservative. On the other end, a more labour-intensive FEM-model is needed. Still, once this method is used for many fatigue details in orthotropic bridges, the traditionally SN-curves could be adapted to the results of this alternative approach and the calculations would be less intensive and still less conservative.

Application of the method to the case of the Temse bridge in Belgium, the crack length seems to propagate through the weld throat of the stiffener-to-deck plate detail and not through the deck plate as observed. Approximately $38{,}5.10^6$ load cycles are needed for a crack length of 600 mm. This is more than the traditionally calculations would predict because the FEM-model does not take residual stresses and other welding defects into account. This may be a possible improvement in future research. Nevertheless, facture mechanics seem to be an appropriate method to improve the results of traditional calculations.
A second advantage is that fracture mechanics can calculate the whole crack propagation. This implies that the total fatigue life of the structure becomes known for every crack length. Therefore, remaining lifetime predictions could be made for existing steel bridges. Although not studied in this research, fracture mechanics can also deal with load histories and load sequences. This is much more accurate than the Palmgren-Miner hypothesis which assumes that the load sequence has no effect at all on the total fatigue life of the structure.

Nomenclature

a	crack length
A	surface of the domain being studied
C,m	constraints of the Paris law
E	Young's modulus
G	energy release rate
I	classical term of the interaction integral
J	J integral
K_I	stress intensity factor of mode I (opening)
K_{II}	stress intensity factor of mode II (sliding)
K_{III}	stress intensity factor of mode III (tearing)
N	number of loading cycles
r	distance crack front to the point being studied
u	displacement vector
Γ	path for integral determination
ε	strain
θ	angle between the crack front and the point being studied
ν	Poisson coefficient
σ_{ii}	stress in the ii-plane

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