A FRACTURE MECHANICS ASSESSMENT METHOD FOR OLDER STRUCTURAL STEELS

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A method for calculation of fracture toughness requirements for ordinary structural steel is described. The method is based on typical loading cases and takes distributed load and fracture toughness into account while the size of an assumed crack is kept deterministic. Transferability has been studied by means of full scale testing of structural elements and laboratory specimens extracted from these. Particulary for older and inhomogeneous steels transferability is obtained under similar constraint and for full thickness laboratory specimens. Examples of toughness as a function of required safety are given.

INTRODUCTION

Of a total of some 3000 bridges in the Swedish national railway network some 1200 bridges of steel were erected before 1960. The mean age of these bridges is around 75 years. The question of the toughness of older steels in such and similar structures of the same age in public service, e.g. road and street bridges, has been adressed from time to time in Sweden as well as many other countries during the last decades. Often, suprisingly low notch toughness figures are reported, of the order 2-5J, in view of the fact that Williams & Ellinger (1) reported brittle fracture in base materials from fractured ships with up to some 15J Charpy notch toughness. Such low toughness values seem to be common knowledge of many investigators and also of those responsible for the use of older steels. This and the observations a) that the common toughness requirement, 27J Charpy notch toughness according to the Bonhomme-recommendation,

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strictly applies to structural steels intended for welding and b) that failures due to fracture in bridges of riveted and rolled members on the whole are very rare, have risen the question "What minimum toughness is necessary to ensure present safety requirements in older non-welded structures?"

With the possible exception of the most brittle older steels, fracture is preceded by plastic deformation to such an extent that the conditions for small scale yielding fracture mechanics are not fulfilled (2). The path independent J-integral and its value at incipient crack extension are used here as measures of the crack driving force and fracture toughness respectively.

OLDER, INHOMOGENEOUS STEELS

Fracture toughness testing according to the ASTM standard E813 (3) of a random sample taken from some 20 older steel bridges erected 1900-1940 showed that the corresponding steels could roughly be divided into two groups (2). From an engineering point of view the steels in the first group did not meet present safety requirements at the testing temperature -30° C, while those in the second group most probably were ductile enough to remain in service. Both groups were roughly of equal size. The Charpy notch toughness of all steels were typically some 5-7J at this temperature. Notch and fracture toughnesses agreed resonably for the steels in the first group but not for those in the second.

It is found that the steels in the second group are much more inhomogeneous with typically a ductile surface layer and a brittle core, Fig 1. The random sample notch toughness specimens were almost always taken from core material. In view of this toughness variation it was concluded that specimens whose thickness equals that of the rolled material were required to correctly estimate the effective toughness of a structural element and that Charpy testing alone might be misleading.

TRANSFERABILITY

Subsequently testing of full scale rolled broad flange beam elements and of full thickness SENB and CT specimens, taken from the flanges, was carried through in order to check transferability. Beam elements with an edge crack in one of the bottom flanges were slowly loaded to failure in four point bending at a testing temperature as above. A testing program comprising five beams is fully described in (4).

One beam, a HEB400 of steel St37 class D, is recently produced and has not been in service. An older DIP 42 1/2 beam of a killed but inhomogeneous steel and three older DIP 42 1/2 beams of rimmed steel have been in service 1962-1987 and 1919-1987 respectively. The crack plane in the SENB and CT specimens was perpendicular to the rolling direction of the parent material and the tip of the crack at the same distance from the flange edge as in the beams.The result of the fracture toughness testing is summarized in Table 1.

From an engineering point of view the first two beams were ductile and the latter three brittle. The fracture toughness of the ductile beams is typically greater than 100 kN/m and of the brittle beams typically 10-50 kN/m. The fracture toughness scatter is considerable and of the same order for almost all steels, new or old, ductile or brittle, although the fracture toughness mean values are very different. The constraint measure Q, as defined by Dodds et al (5), is very close to zero for the brittle beams and the corresponding specimens. Experimentally determined values of J and Q for the ductile beams and their specimens are shown in Fig 2. Q has been determined from diagrams given by O'Dowd and Shih in (6).

TABLE 1 - Fracture Toughness Testing of Older Steels

Beam No	Fracture toughness J_c (kN/m)	Number of specimens	Mean fracture toughness J_m (kN/m)	Std.dev. J_s (kN/m)	Coeff Of Variance $v_J = J_s/J_m$
1	93	14	142	44	.31
2	-	11	159	75	.47
3	35	9	12	3	.25
4	23	11	38	27	.72
5	20	10	45	25	.56

The HEB400 beam with a crack length of 70 mm failed at a J-Q-point which agrees reasonably with the J-Q-locus of the SENB and CT specimens. The 1962 DIP 42 1/2 beam with crack length 38 mm did not fail through fracture but buckling. The loading point in Fig 2 is constructed by assuming that the toughness of the beam was equal to the lowest of its specimen values. Obviously fracture did not occur in this beam due to loss of constraint for the short crack.

The fracture thoughness of the beam elements fall reasonably within the scatterband of the corresponding SENB and CT specimens. Thus the behaviour of the beams can be predicted from extracted specimens provided the constraint is similar in both cases.

THOUGHNESS REQUIREMENTS

The results so far have been used to develop a method for calculation of fracture toughness requirements. The basic idea is that an assumed crack in the most stressed cross section of a structural element must not reduce the safety against fracture below what is required by present "crack-free" design codes. The loading is based on typical strain histories as measured on railway bridges in service (7). Loading and fracture toughness are assumed distributed while the crack size is kept deterministic. Estimates of scatter are based on measured strain histories and fracture toughness data given above. A typical example of a crack geometry is the broad flange beam element loaded in bending, Fig 3. The J-integral has been calculated with a general purpose FEM-code for one typical crack length and estimated with the R6-method (8) for other crack lengths. This example has also been used for determination of J_C for Beam No 1.

An iterative procedure, Fig 4, is used to calculate a required fracture toughness mean value for a given load distribution and crack size. The "resistance" is simply the equivalent of J along the load axis in Fig 3. The derivative $\partial J/\partial\sigma_o$ is evaluated analytically, as this is found to give better accuracy than numerical differentiation of the R6 J-expression. The required difference between resistance and load is obtained by using the expression (notations are given in Fig. 4):

$$\beta = \frac{\mu_R - \mu_L}{\sqrt{(\sigma_R^2 + \sigma_L^2}} \tag{1}$$

according to the the prescribed procedure in the current national design code (9). It is exact only if load and resistance are normally distributed. The result of a typical calculation is shown in Fig 5. In this example ν_L = 0.1 and ν_J is chosen to 0.5 based upon the data given in Table 1.

The method is used to calculate mimimum toughness requirements for a small number of specimens, extracted from a full scale structure. Because of the considerable scatter two fracture toughness levels are suggested. The lower level is chosen such that replacement should be done as soon as possible. Above the upper level safety is satisfactory. The upper level is approximately equivalent to to 27J Charpy notch toughness. Between the two bonds further testing and individual judgement are allowed. The fracture toughness requirements for the load case shown in Fig 3 are given in Table 2.

In practice only very small volumes of sampling material can be extracted from structures in service. If the minimum toughness of three specimens is taken as a characteristic value then the risk to overestimate the mean toughness is 12.5%. The safety of this estimation procedure used in conjunction with the thoughness calculation metod has been checked through direct application of the Monte-Carlo method.

The Monte-Carlo verification is based upon direct comparisons of the crack driving force and fracture toughness.The fracture toughness of a given material is assumed Weibull-distributed. The shift value is chosen

TABLE 2 - Fracture toughness requirements

a (mm)	L_m (MPa)	ν_L	ν_J	J_m (kN/m)	β
50	50	.1	.5	20	2.75
100	50	.1	.5	50	3.25

to half the 5%-fractile value of the empirical distribution curve. The Weibull shape and scale parameters are then determined according to the maximum likelihood method. The lowest of three fracture toughness values, chosen at random from the distribution curve, is taken as a characteristic or estimated mean fracture toughness value. Together with data given in Table 2 the estimation procedure is used to calculate a value of J which is compared to a random value on the fracture toughness distribution curve. As the fraction of failures at both the upper and lower fracture toughness limits is smaller than 10^{-4}, these limits are considered safe. This method is also slightly more conservative than that given by BSI (10), provided that the CTOD partial safety factor also applies to J.

The metod has been tentatively used since 1991. Specimen material has been extracted from more than 50 bridges yearly and the replacement rate is some 30 bridges per year. Full scale testing is performed on structural elements taken from dismantled bridges which do not fulfill the thoughness requirements described here. Also, that bridges which fulfill the toughness (and other) requirements are not taken out of service irrespective of their Charpy notch toughness, is the subject of an ongoing study.

In conclusion it has been found experimentally that Charpy testing alone of older and inhomogeneous steels might be misleading. Full thickness specimens are requried to obtain the effective thoughness of a structural element. The fracture toughness of a full scale structure can in the present case be predicted from small specimens provided the constraint is similar in both cases.

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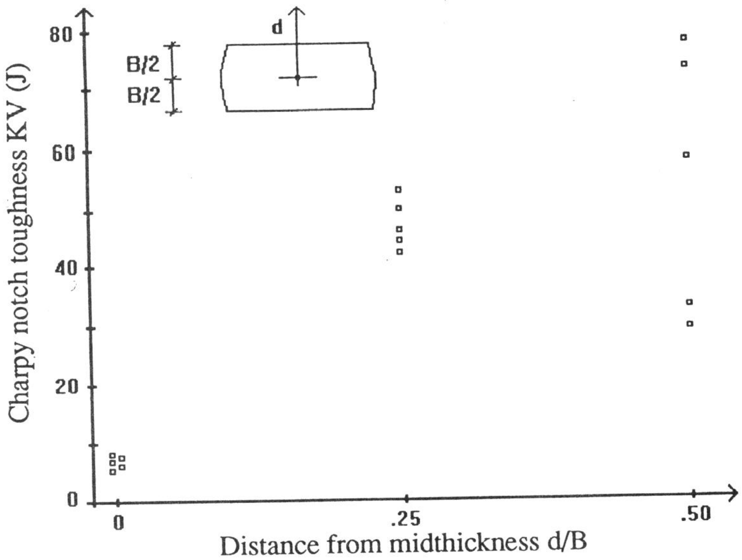

Figure 1. Charpy notch toughness of an inhomogeneous steel
Plate thickness B = 25 mm.

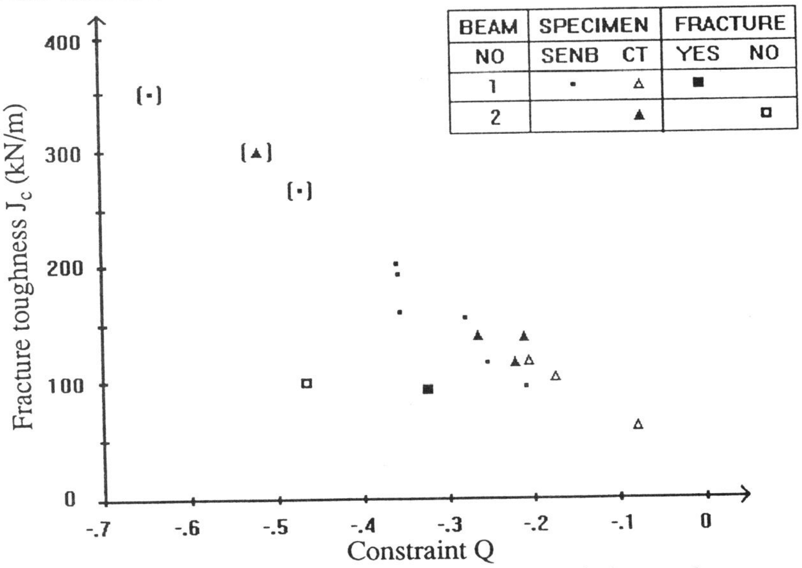

Figure 2. Fracture toughness J-Q-locus for full scale beam elements and extracted specimens.

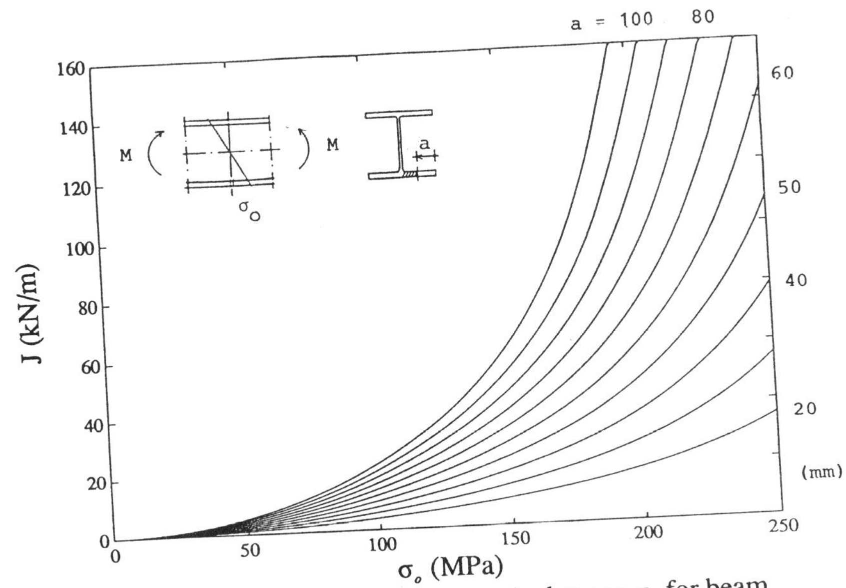

Figure 3. Crack driving force J versus nominal stress σ_o for beam HEB400. Yield stress 300 MPa.

μ mean
σ standard deviation
β safety index
 	
L load
R resistance
a crack length
 	
Given $\mu_L, \sigma_L, \sigma_J$
Guess $\mu_R, \Delta\mu_R$
	$n = 0$
	A $\mu_R = \mu_R + n\,\Delta\mu_R$
	$n = n + 1$
	$s = \mu_R / \mu_L$
	$\mu_J = J(\mu_R, a)$
	$\overline{\sigma}_R = (\partial J / \partial \sigma_o)^{-1}_{J=\mu_J}\ \sigma_J$
	$\overline{v}_R = \overline{\sigma}_R / \mu_R$
	$\overline{s} = \overline{s}(\overline{v}_R, \beta)$
	if $\lvert \overline{s} - s \rvert \leq \varepsilon$ *then*
	μ_J *ok*
	else
	goto A
	end

Figure 4. J-integral iteration procedure

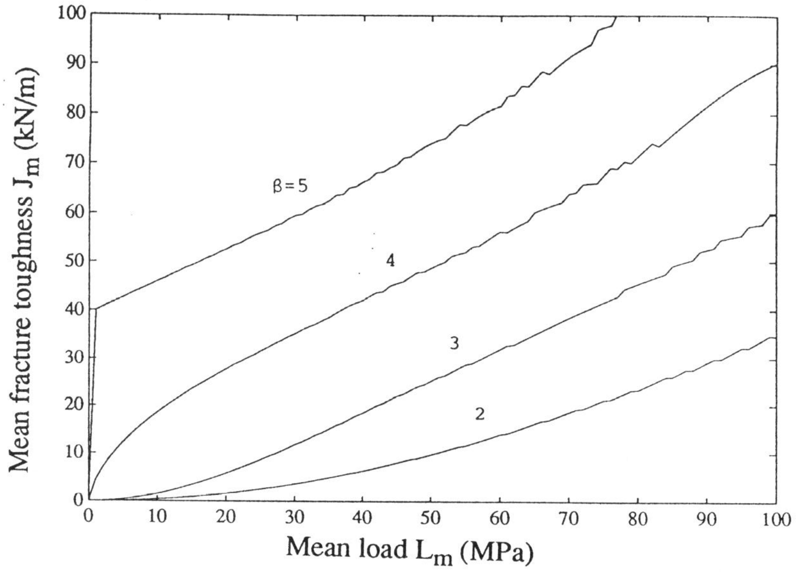

Figure 5. Mean fracture toughness versus mean load and safety index β. Crack length a = 50 mm.