RICERCA

Web breathing in slender plate girders

"Web breathing" nelle travi a parete piena sottile

If a steel girder with slender web panels is subjected to cyclic loading fatigue cracking can occur due to repeated web buckling deflections, so called web breathing. Normally the cracks initiate in the vicinity of the boundaries of the web panel, i.e. close to the connections with the flanges or vertical stiffeners. The results of a series of fatigue tests on four full-scale slender plate-girders with medium to large panel aspect ratios subjected to repeated combined action of bending and shear, are presented here. The plate girders (span: 7500 mm, depth: 1200 mm, web thickness: 5 mm) were tested under constant amplitude fatigue loading up to failure. The girders tested were also analysed by means of the finite element method (FEM). The laboratory test data was evaluated and compared with results from numerical simulations performed by means of the finite element method showing a satisfactory agreement. The results show that, for the girders of the type presented in this paper, fatigue cracking due to web breathing occurs at stress ranges comparable to those caused by common bridge service loads.

Sottoponendo una trave con anima a parete piena sottile ad un carico ciclico, a causa dei ripetuti imbozzamenti dei pannelli d'anima, si possono innescare cricche dovute a fatica. Tali cricche, normalmente, hanno luogo lungo il perimetro del pannello d'anima, in prossimità dei giunti saldati tra anima e ali o tra anima e irrigidimenti. Il fenomeno dei ripetuti imbozzamenti dei pannelli d'anima è comunemente indicato nella letteratura tecnica anglosassone con il termine "web breathing" (italiano: "respirazione dei pannelli d'anima"). In questa memoria vengono illustrati i risultati di una serie di prove di fatica condotte su quattro travi in acciaio ad anima sottile (luce: 7500 m, altezza: 1200 mm, spessore dell'anima: 5 mm). I risultati delle prove di laboratorio sono stati successivamente validati per mezzo di modelli agli elementi finiti (FEM). Si è osservato che il collasso da fatica indotta dalla "respirazione dei pannelli d'anima" può avvenire, per travi del tipo considerato in questa ricerca, per delta di tensione comparabili a quelli causati da normali carichi di esercizio nei ponti.

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1. INTRODUCTION

Slender plate girders are used in a large variety of engineering applications, such as bridges, ships and aerospace structures. If furnished with sufficiently rigid boundary constraints, slender steel plates may exhibit significant postbuckling reserve of strength. This fact is normally taken into account, for instance in limit state design methods for structural steel codes. Owing primarily to the unavoidable presence of initial out-of-flatness, slender web plates may undergo relatively large out-of-plane displacements, even under relatively small applied loads. Furthermore, when the theoretical critical load for the actual plate is exceeded, these displacements start increasing "faster". This nonlinear behaviour means, in other words, that for a given increment of in-plane load (e.g. shear force or bending moment) the corresponding increment of the web out-of-plane displacement in the postcritical domain is larger than in the precritical domain. These displacements will induce, in turn, relatively high secondary bending stresses σ_b and surface stresses σ_s along the boundary of the web panel, cf. Figure 1. Under cyclic loading, e.g. induced by road or rail traffic on bridges, this can result in premature fatigue failure (see Figure 1).

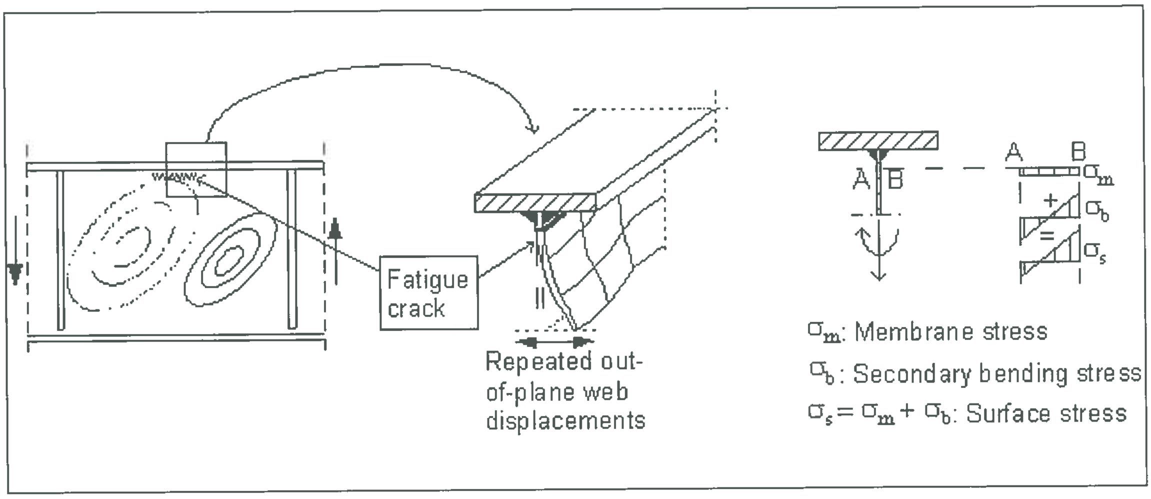


Figure 1: Fatigue cracking due to breathing in a web panel subjected to predominant shear action.

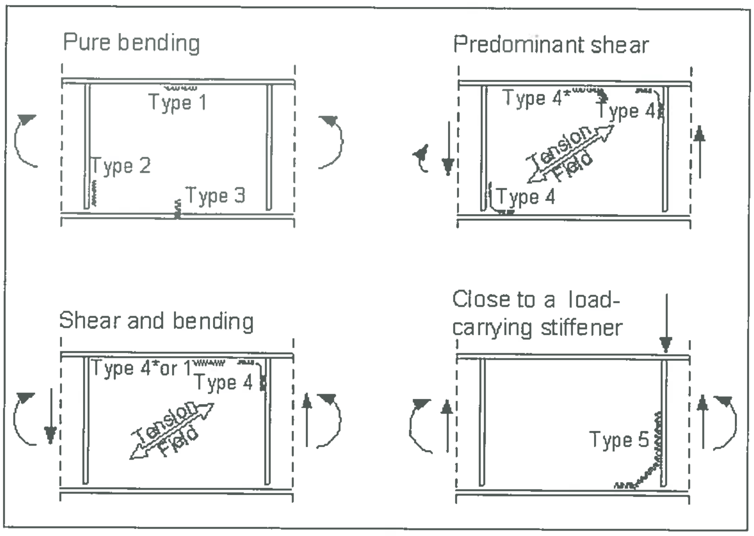


Figure 2: Typical fatigue cracks observed in plate girders with slender webs.

When a slender plate girder is subjected to repeated loading, there are possibilities of initiation and propagation of fatigue cracks at the toes of the fillet welds connecting the girder web to the flanges or the vertical stiffeners to the web. As shown in Figure 2, the fatigue cracks are classified in five types, associated with one of the following load conditions, namely:

- Pure bending;
- Predominant shear action;
- Combined shear and bending;
- Combined shear and bending in the vicinity of a load-carrying stiffener.

a) Girder web panels subjected to pure bending

In girder web panels subjected to pure bending, the following three types of cracks may occur:

- Type 1 crack, which is initiated at a weld toe on the web side of the fillet weld between the web and the compression flange. The crack is caused by the cyclic plate-bending stress (i.e. the secondary bending stress range) at the weld toe which is induced by out-of-plane deflections of the web under in-plane bending. It propagates along the weld toe with a relatively low speed.
- Type 2 crack, which is initiated at the web side of the fillet weld that connects the vertical stiffener to the web. This crack is not related to breathing (i.e. repeated out-of-plane displacements) of the web. It propagates towards the zone with higher tensile stress. When it has penetrated into the tension flange it may lead to fracture of the flange, hence to a sudden collapse of the girder. The rate of propagation of this crack (i.e. the ratio of the increment of crack length during a certain number of cycles) is generally greater than that of Type 1 crack. This is due to the fact that Type 2 crack is "driven" by pure membrane tensile stress range rather than plate-bending stress range, as it is the case in Type 1 crack. Thus, the range of stress intensity factor DK – i.e. the relevant parameter which governs the rate of propagation of a crack – is greater at the tip of the former crack than at that of the latter.
- Type 3 crack, which is initiated at the web side of the fillet weld at the tension flange. This crack is not related to breathing of the web. It is normally caused by weld imperfections, such as stop/start points.

b) Girder web panels subjected to predominant shear action

In girder web panels predominantly under shear, the buckling mode consists of one or more buckles, depending on the panel aspect ratio (length/depth) as well as on the shape and the magnitude of initial web deflections. The inclination of the buckles can vary within the range of approximately 20° to 45° to the longitudinal edges, depending on the "degree of restraint" against in-plane displacements (i.e. the possibility of the flanges to move towards each other). For web panels with relatively small aspect ratio (say, $\alpha \leq 1.0$), the buckling mode is normally just a major diagonal buckle parallel to the direction of the principal tensile stress. In this case, a Type 4 crack may be initiated at toes on the web side of the fillet welds near the corners where the diagonal tension field is anchored. This is primarily due to the large local curvature of the web deflection – accompanied by high plate-bending stresses – caused by the shear buckle in these corner zones. For web panels with relatively large aspect ratio, however, two or more "inclined" buckles may develop in the postcritical range. In this case, Type 4 or 4* cracks may be initiated at toes on the web side of the fillet welds in the zones close to the vertex of the buckles. However, it should be remarked, that if too high levels of loading are used in fatigue testing, then Type 4 crack is more likely to occur than Type 4*. In fact, for loads close to the collapse load of the girder, the web tends to assume a configuration of equilibrium with only one major buckle along the "tensile diagonal" of the web panel.
Both Type 4 and Type 4* cracks propagate along the weld toes and then branch out into the web in the direction approximately perpendicular to the longitudinal direction of the buckle. Finally, when the cracks are sufficiently long, the girder panel loses its load-carrying capability because of the diminished tension field action.

c) Girder web panels subjected to combined shear and bending

In girder web panels subjected to combined bending and shear, Type 1, Type 4*, and Type 4 cracks have been observed during laboratory testing.

d) Girder web panels subjected to combined shear and bending in the vicinity of a load-carrying stiffener

In girder web panels subjected to combined shear and bending in the vicinity of a load-carrying stiffener, a Type 5 crack may be initiated at a toe on the web side of the fillet weld that attaches the bearing stiffener to the web. This crack is usually initiated close to the neutral axis of the girder, where the bending stresses determined according ordinary beam theory are small. In the zone close to the point of load application, the web plate is subjected to relatively high compressive stresses. This is due to the fact that the applied concentrated load is not entirely taken by the vertical stiffener, since a part of it is "absorbed" by the web by compressive stresses. Although the part of the concentrated load taken by the web is generally rather small, it contributes to reduce the plate flexural stiffness making the web more prone to buckle under the action of bending and shear. Therefore, relatively large out-of-plane displacements accompanied by plate-bending stresses, are usually observed in the compression zone of a web panel close to a load-carrying stiffener. Type 5 crack propagates along the weld toe and then branches out into the web in a direction approximately perpendicular to the tension field direction of the shear buckle. When the crack has reached the tension flange, it continues to propagate along the toe of the fillet weld at the web to tension flange. Finally, when such a crack is sufficiently long, the girder panel loses its load-carrying capability because of the diminished shear buckling resistance of the web.

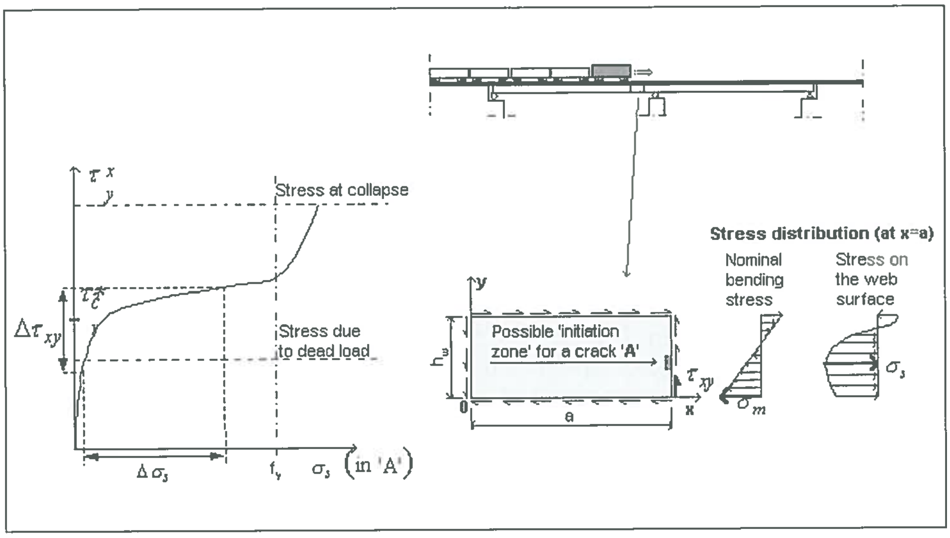


Figure 3: Schematisation of the phenomenon of web breathing for a bridge with slender steel I-girders. Note that τ_{cr} is a function of τ_{xy} and σ_m.*

Figure 3 illustrates schematically the phenomenon of web breathing for a continuous steel bridge with slender I-girders with only transversal (vertical) stiffeners. The web panels which are most prone to breathing – in the hypothesis that the web thickness is not dramatically increased at the support zones - should be those located in the vicinity of the inner supports where both bending moment and shear force due to the self-weight of the superstructure are largest. Therefore, in this zone traffic loads may be such that, when added to the permanent loads, they cause stresses that exceed the theoretical critical stress of the web panel. Hence, large out-of-plane deformations - accompanied by large secondary stresses at the boundary of the web panel - are expected in these zones.

As can be observed in Figure 3, the relationship between the in-plane stress (i.e.τ_{xy} or σ_m) and the surface stress σ_s ($\sigma_s = \sigma_b + \sigma_m$, where σ_b is the secondary bending stress and σ_m is the membrane stress, i.e. in the middle of the web thickness) is non-linear. Moreover, for surface stresses smaller than the yield stress f_y the gradient of the curve becomes smaller when the theoretical critical stress is exceeded. This means that – for a given increment of applied in-plane stress τ_{xy} - the correspondent increment of surface stress σ_s becomes greater when the critical stress is exceeded. For common traffic loads, the maximum surface stresses and also the maximum stress ranges due to out-of-plane displacements of the web are expected along the web boundaries with the vertical stiffener or with the compression flange in the zones of the web where bending moments and shear forces are largest. In web panels with relatively large aspect ratio, maximum surface stress at the corners of the web panel where the "tensile diagonal" is anchored should be expected only in the case when the in-plane shear (i.e.τ_{xy}) approaches the ultimate shear capacity of the panel. However, this circumstance is not likely to occur in real bridge girders. Here, in fact, due to the different safety factors used in design, stresses seldom exceed the theoretical critical stress much, thus they are generally far below the stress level which would cause static collapse of the girder.

Fatigue cracks due to web breathing generally initiate at a point (or a small zone) where the range of "nominal" surface stresses, i.e. the surface stresses calculated without consideration to welding imperfections, is maximum. (This point often coincides with – or is close to - the point of maximum surface stress.) However, due to welding imperfections, fatigue cracks may also occur elsewhere along the boundary of the web panel. In fact, due to the large stress concentration factor, the actual range of surface stresses, for example, at a stop/start point (which is the source of a rather common defect in welded girders) may be greater than the largest "nominal" range of surface stresses.

2. AIM AND SCOPE OF THIS RESEARCH

Research in the field of web breathing of thin-walled plate girders seems to have started in North America in the 1960's by Yen and Mueller (1966). Successively, a significant number of researchers have contributed to a better understanding of this phenomenon by both theoretical and experimental studies. The most relevant experimental results were obtained by Yen and Mueller (1966), Patterson, Corrado, Huang and Yen (1970), Toprac and Natarajan (1971), Maeda (1971), Huang (1994), Juhas (1995), Aribert, Remadi, Raoul and Carracilli (1996), Roberts, Davies, Osman and Skaloud (1997), Kuhlmann and Günther (1999), Kuhlmann, Spiegelhalder and Günther (2000).

However, most of the fatigue tests performed with regard to web breathing found in the literature, with only few exceptions have the following characteristics in common:

a) the girders were subjected to predominant shear;
b) the girders had rather small panel aspect ratios $\alpha = a/h_w$ (often $\alpha < 1.5$);
c) the maximum load in the fatigue tests was often chosen relatively close to the collapse load for the actual girder and the minimum load close to zero;

d) the specimens used were small-scale girders.

However, the results obtained from such tests are perhaps not directly applicable to the design of, for example, bridge girders. Below, some critical observations for each of the four above points (a to d) are presented:

1) In bridge structures, the girders are often continuous over several supports. Therefore, the web panels which are most prone to breathing should be those located in the vicinity of the inner supports (not exactly above the supports, where vertical stiffeners and thicker webs are generally used in order to avoid instability problems), where both bending moment and shear force are large;
2) Due to economical reasons (i.e. high labour costs) the tendency is nowadays to reduce the number of stiffeners; thus, panel aspect ratios of $\alpha > 1.5$ are rather common in bridge structures;
3) Actual traffic and dead loads are far below the load necessary to cause the static collapse of a bridge girder. In fact, even for the most severe traffic loading, the girder has still a great amount of load-carrying capacity left, due to the partial coefficients (used in order to take into consideration uncertainties concerning both material and loads) adopted for design with respect to the ultimate limit state. On the other hand, the minimum load can never be zero (or close to), because of the dead load of the superstructure.
4) It has been observed that secondary bending stresses, and thus the fatigue behaviour of girder webs with regard to breathing, is strongly dependent upon the size and shape of initial imperfections. When using small-scale testing specimens, all the dimensions should be scaled down, including those of the welds. However, it is almost impossible to reduce the weld throat size proportionally to the other dimensions of the girder. In other words, when using small-scale girders, the size of the weld throat may often be exagerated with regard to the web thickness. This may result, among other things, in introducing unrealistic initial geometrical imperfections in the web (i.e. out-of-flatness) due to the relatively excessive heat input.

The choice of specimens and loading conditions presented in this paper was "inspired" by the above-mentioned observations and one aim is to furnish some further information concerning web breathing, especially with regard to its application to the design of steel bridges.

3. LABORATORY TESTING

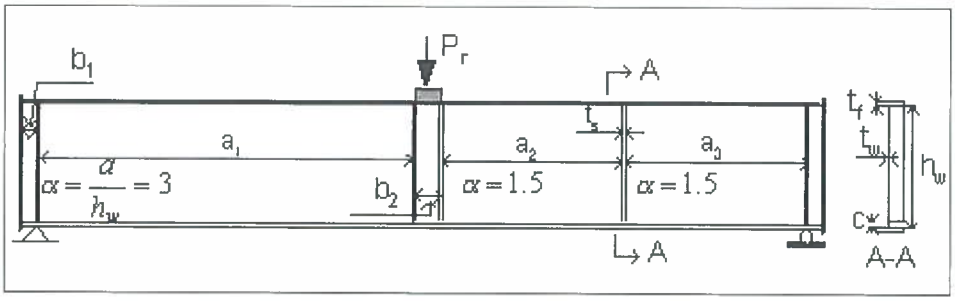
A series of fatigue tests was conducted on four full-scale plate girders, all with the same nominal dimensions. Fabrication details of the test girders are presented in Figure 4 and Table1.

The four girders were labelled G0, G1, G2 and G3. Each girder was provided with vertical stiffeners which created three sub-panels: one with an aspect ratio of α=3 and two with α=1.5. The non-load-carrying stiffener was at only one side of the web. The sub-panels were given the names Gk0, Gk1 and Gk2 respectively, where k (0,1,2,3) is a number, which identifies the actual girder. Fillet welds with throat size of about 5 mm were used for the flange-to-web connections and of about 8 mm for the stiffener-to-flange and stiffener-to-web connections. The steel used for both flanges and web was SJ 355, with characteristic yield strength f_y=355 MPa. Tensile tests on specimens taken from each girder (flange and web) were performed. The average yield stress was f_{yw}=428 MPa for the webs and f_{yf}=394 MPa for the flanges. The average elongation at the rupture of the specimen was ε_{uw}=25.2% for the webs and ε_{uf}=27.5% for the flanges. The average value for the modulus of elasticity was 2.0×10^5 MPa. Three girders were tested at identical load levels, namely P_{min}=160 kN and P_{max}=400 kN, while the remaining girder (G2) was tested at P_{min}=350 kN and P_{max}=600 kN. It should be noted, however, that the load ranges was nearly the same for all the girders.

The girders were tested using a servo-controlled dynamic pulsating machine (type: Losenhausenwerk). Each girder was supported on a roller and a bearing pin; the pulsating concentrated load was applied over a length of 300 mm at mid-span on the upper flange above a double stiffener (see Figure 5). Out-of-plane displacements were measured by means of a specially fabricated transducer bar at a static loading $P=P_{min}$ and $P=P_{max}$. Readings were taken at a number of 150x150 mm² equally spaced grid points. It should be noted that girder G0 had a maximum initial out-of-flatness of about 31 mm, which is much larger than the maximum amplitude usually allowed by codes. Due to this reason, it was decided to use this girder as a "pilot test" and thus, no strain gauges were applied there. The purpose of this test was to investigate the location where fatigue cracking may occur and thus gain useful information for placing strain gauges in the next girders.

For three of the girders tested, strains were measured by means of 55 strain gauges per girder, of which 54 were placed in pairs,

Figure 4: Fatigue test set-up and details (after Journal of Constructional Steel Research).



Girder	a_1	$a_2(=a_3)$	b_1	b_2	b_f	t_f	h_w	t_w	t_s	c	f_y
G0-G3	3600	1800	150	300	250	15	1200	5	15	15	355

Table 1: Details of the test specimens. The units are [mm] and [MPa].

one on each side of the web. In this way, both secondary bending strains and membrane strains could be captured. The distance between the centre of a gauge and the closest surface of either the flange or a stiffener was 15 mm. Two of the tested girders were also furnished with a 45° rosette gauge, which enabled to detect possible changes in the direction of the principal strains during loading.
Prior to each fatigue test, the out-of-plane deflections were measured at the load levels P=0, $P=P_{min}$ and $P=P_{max}$. Figure 5 shows the measured out-of-plane deflections of one of the tested girders. A complete set of strain gauge readings was also recorded for each of the aforementioned loads. Strains were recorded for a couple of minutes under pulsating loading in order to detect possible dynamic effects. This procedure of strain gauge readings was repeated every day, when the testing machine was activated (after having been turned off during the nights). The frequency of the pulsating load was approximately 3 Hz.

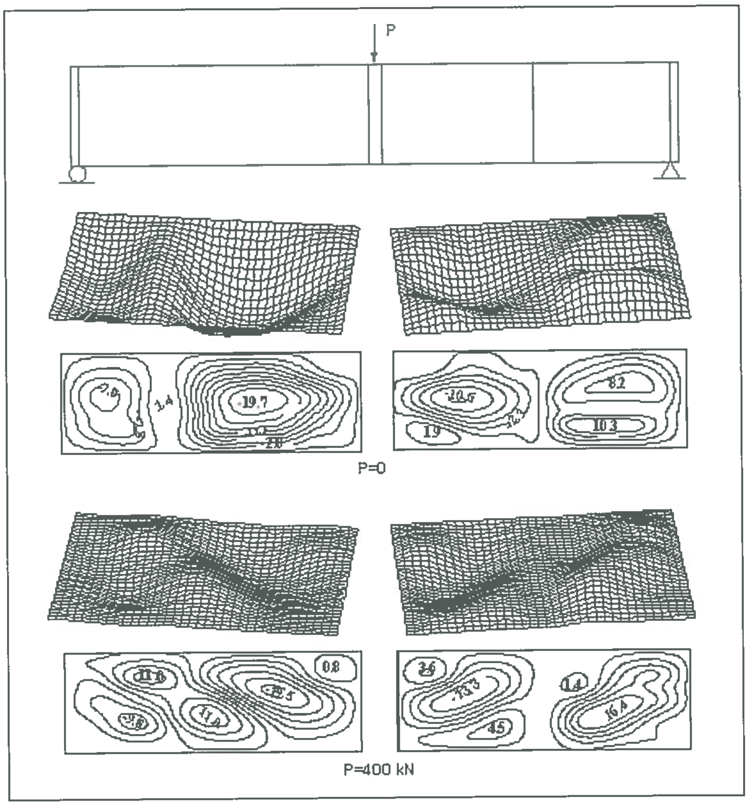


Figure 5: Measured out-of-plane displacements of girder G3 at P=0 and at $P= P_{max}$=400kN (after Journal of Constructional Steel Research).

4. LABORATORY TEST RESULTS

The stresses which are primarily responsible for the initiation of fatigue cracks due to web breathing are those that occur at the surface (σ_s) of the web, perpendicularly to the longitudinal direction of the welds connecting web and flange and web and stiffeners. These stresses consist of the sum of two contributions ($\sigma_s=\sigma_m+\sigma_b$), i.e.

- the membrane stress, or normal stress (σ_m), i.e. the stress at the mid-surface of the web plate;
- the secondary bending stresses (σ_b), due to out-of-plane web deflections.

For small loads, σ_m is usually larger than σ_b. However, especially for very slender webs, as the load increases past the critical load σ_b starts to become predominant and may become several times σ_m already at not too high loads (say less than one half of the load carrying capacity of the girder).
Fig. 6 shows the variation of the secondary bending stresses, "measured" in different zones of girder G2, with the applied load. The σ_b-P curves relative to girders G1 and G3 showed similar trends as those obtained for girder G2. However, G1 and G3 were only loaded up to 400 kN.

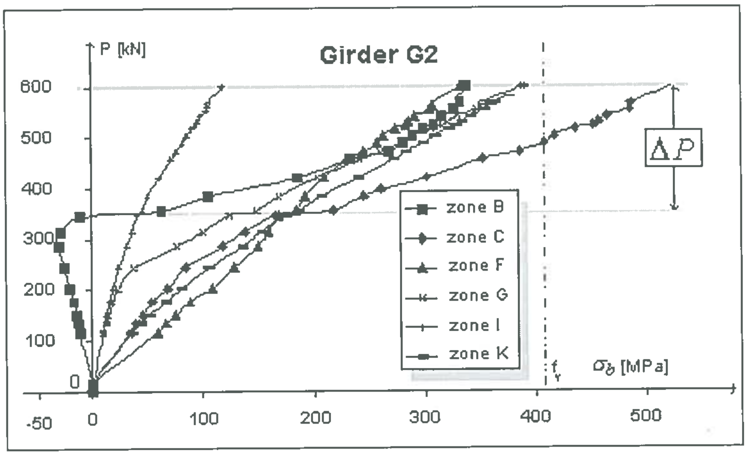


Figure 6: Secondary bending stress (obtained from strain measurements in different zones of girder G2) versus load (after Journal of Constructional Steel Research).

As a first observation, it should be noted that even at relatively small applied load the web can start to yield at some points. In fact as shown by the curve for zone C in Fig. 6, yielding occurs at a load level of about one half of the load carrying capacity of the girder (which is about 900 kN).
Figure 6 also puts in evidence the importance of the choice of load levels for fatigue testing. As an example, suppose that girder G2 shall be fatigue tested at a load range of 200 kN. If a minimum load of 400 kN is chosen, then the maximum stress range (disregarding membrane stresses) is attained in zone B. However, if a minimum load of 300 kN is chosen, then the maximum stress range is attained in zone C.
It should also be noted that - despite that the load range used during testing was approximately the same for all the girders - girder G2 exhibited the first crack and failed at a much smaller number of load cycles than the other girders. This is due to the fact that, due to the change in slope (flattening) of the σ_b-P curves with increasing P, for a given load range the corresponding range of secondary bending stresses is larger if a higher level of applied loads is chosen.
All the curves had a non-linear behaviour already at a low level of the external load. No clear sign of instability phenomena in terms of an evident sudden change of the slope of the σ_b-P curves were observed in any of the tested girder, with only one exception, namely girder G2, gauge location G2,B. However, a slightly perceptible decrease of the gradient $\partial P/\partial\sigma$ within the load range of about 200-300 kN could be observed in several of the strain gauges used during testing. This may depend upon the fact that elastic buckling for web panels Gk0, in the hypothesis that they were perfectly flat, is expected in the range P=172-263 kN, depending on the assumptions made regarding the boundary conditions of the web plate (i.e. simply supported or clamped along their edges).
All the fatigue cracks observed during the experiments, initiated along the toe of the fillet weld between the web plate and the

Web Panel	$P_{cr,h}$	$P_{min}/P_{cr,h}$	$P_{max}/P_{cr,h}$	N_i (x 10^6)	N_f (x 10^6)
G00	183	0.87	2.19	0.693	0.919
G01	207	0.77	1.93	0.447	-
G02	263	0.61	1.52	-	-
G10	183	0.87	2.19	0.226	1.93*
G11	207	0.77	1.93	-	-
G12	263	0.61	1.52	-	-
G20	183	1.91	3.28	0.119	0.493**
G21	207	1.69	2.90	0.321	-
G22	263	1.33	2.28	-	-
G30	183	0.87	2.19	-	-
G31	207	0.77	1.93	1.371	2.52
G32	263	0.61	1.52	-	-

* *vertical stiffeners were welded at approximately 300 mm from the weld toe along which the crack was propagating in order to prolong the fatigue life of the girder.*

** *The failure was caused by a second crack which occurred in another part of web panel G20.*

Table 2: Summary of the fatigue test results. The units are [mm], [kN] and [MPa]. P_{cr} is the critical load (theoretical) for a web panel subjected to combined shear and bending hinged along its edges; N_i and N_f are the numbers of cycles to crack initiation and to failure of the girder respectively. The index "h" means hinged web boundary conditions.

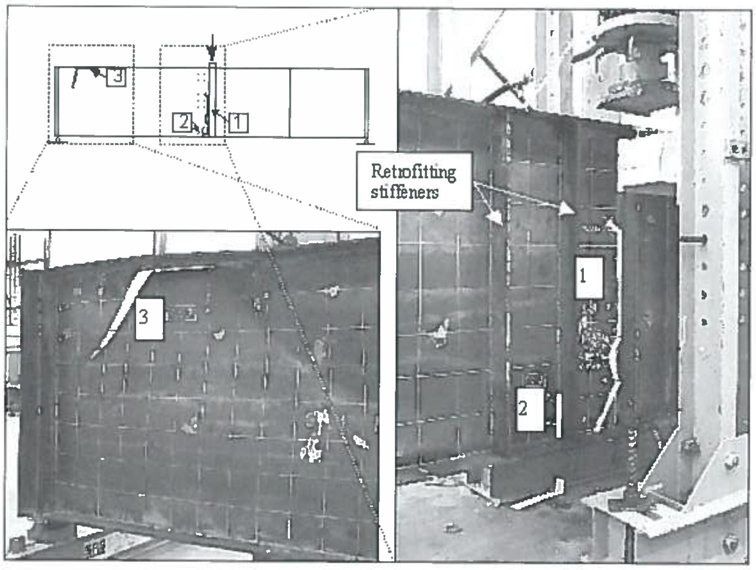


Figure 7: Retrofitting of girder G1.

boundary members, either close to the neutral axis of the girder (for cracks parallel to the vertical stiffener) or close to the upper flange. In these regions secondary bending stresses are generally very high whilst membrane stresses are rather small. Despite visual controls were performed approximately every two hours and the actuator was turned off during unmanned periods, fatigue cracks could not be detected before they had reached a length of 60-70 mm or more. There are reasons to believe, therefore, that the first phase of propagation of such cracks occurs in a rather sudden manner.

Table 2 shows a summary of the fatigue tests results. It should be remarked that web panel G10 was strengthened two times by means of vertical stiffeners welded at approximately 300 mm from the weld toe where a crack had been observed. Each stiffener was welded in turn when the crack had propagated to such an extent as to be considered harmful for the safety of the girder (see Figure 7).

The magnitude of the initial out-of-flatness of the web plate is not always a "sign" of bad fatigue performance with regard to web breathing. In fact, for example in web panel G00 (which had a maximum initial out-of-flatness of about 31 mm) the first fatigue crack was initiated at a considerably larger number of load cycles than in web panel G10 (which had a maximum initial out-of-flatness of only about 12 mm). The reason for this could be the fact that the "membrane action" in a web panel with large initial out-of-plane deflections subjected to in-plane loading is larger than in a nominally identical plate with small initial deflections subjected to the same loading. Consequently, the relative out-of-plane deflections and thus the secondary bending stresses in the former plate will be smaller than in the latter one.

The complete crack scenario for the tested girders is shown in Figure 8. The numbers within squares indicate the order in which the observed fatigue cracks initiated.

It is worth noting that three out of four cracks started at the weld toe at one of the vertical stiffeners directly below the load, where the effect of the combined action of bending and shear is largest.

All the fatigue cracks observed in the testing propagated, during a first phase, parallel to the weld toe. Successively, during the second phase, the cracks changed their direction of propagation, turning from the edge to the inner part of the web. This was probably due to one - or the combination of - the following reasons:

i) when the crack is long enough, the transmission of shear stresses between web and stiffener or between web and flange will be limited to a small area (due to the opening caused by the crack). Furthermore, at the tip of the crack a concentration of shear stresses will occur. The high level of shear stresses will cause the direction of the principal tensile stress (i.e. the stress which governs the direction of crack propagation) to rotate, from a direction nearly perpendicular to the longitudinal edge of the weld to an inclined direction;

ii) the change of direction of crack propagation generally occurred in a zone at a certain distance from the corner of the web. The web in this zone is very stiff and thus not sensible to "breathing". In other words, the triangle of web in the corner can be regarded as a "clamp"; hence, the largest secondary bending stresses, which cause fatigue cracks, are to be expected along the inclined side of this imaginary triangle, i.e. along the line delimiting the "flexible zone" from the "stiff zone" of the web.

Further, it was observed that when a crack reached a certain length in the second phase, collapse of the girder occurred due to shear buckling of the web.

It should be pointed out that no fatigue cracks were observed in the web panels subjected to predominant shear action (Gk2).

Figure 8 shows the rate of growth of fatigue cracks with increasing number of cycles, up to collapse.

As shown in Figure 9 cracks that originated at the weld toe of the vertical stiffener placed below the actua-

tor in the panel with aspect ratio α=3 generally propagated faster than cracks which started elsewhere. It should be noted that the curve representing crack "1" in G1 becomes flatter (i.e. the "velocity" of crack propagation decreases) at about 0.38×10^6 cycles. The reason of this behaviour was that the girder, at that number of load cycles, was retrofitted for the first time by means of a vertical stiffener (see above). The slope of the curve representing crack "2" in G2 is rather steep, even though this crack originated from the weld toe at the upper flange and not at the vertical stiffener. However, it should be remembered that girder G2 was subjected to greater loads than the other three girders; hence the secondary bending stresses (and even the ranges of these stresses) here were significantly larger than in the other tested girders.
Further, cracks that initiated in web panels with smaller aspect ratio (Gk1) seemed to have a more "stable" behaviour. In fact, after a certain number of load cycles, the gradient of the a-N curves representing the propagation of cracks in Gk1 panels, tended generally to reduce.

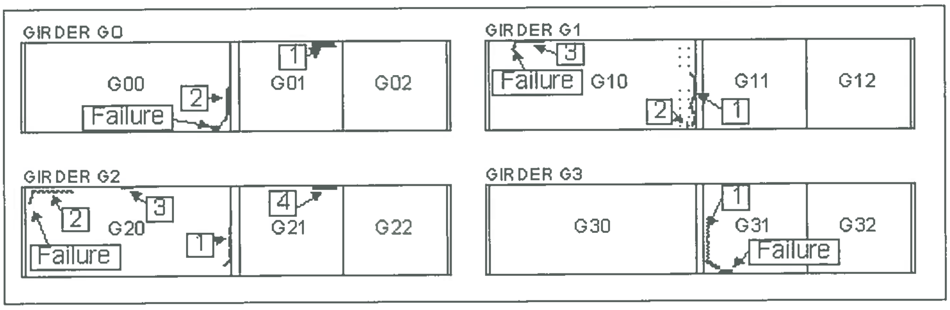


Figure 8: Crack scenario observed in the tested girders after failure(after Journal of Constructional Steel Research).

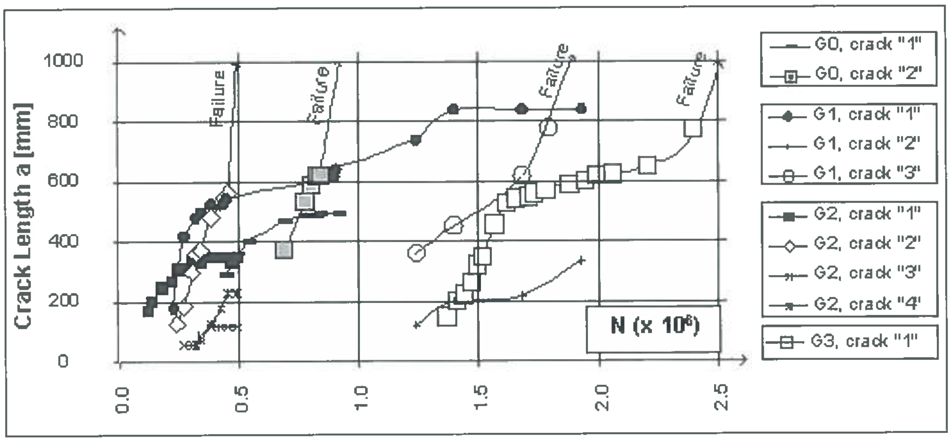


Figure 9: Crack length vs. number of load cycles for the observed fatigue cracks (after Journal of Constructional Steel Research).

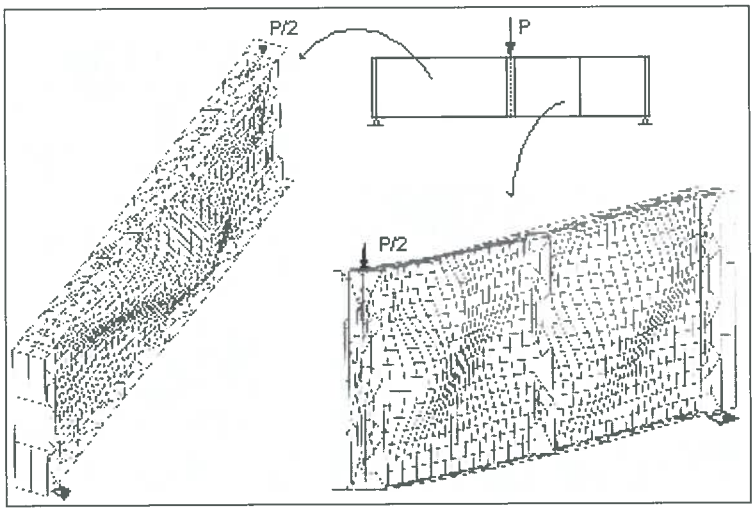


Figure 10: FE-models of girder G1. Note that the girder halves are subjected to $P/2=P_{max}/2=200$ kN. The out-of-plane deflections are amplified by a multiplication factor m=20.

5.FINITE ELEMENT MODEL

The main purpose of the FE-analyses was to gain a better understanding of the postbuckling behaviour of the tested girders and to investigate the stress state in those areas of the boundaries of the web panels which were not covered by strain gauges. Only girders G1 to G3 were FE-analysed. In fact, girder G0 - due to too large initial out-of-flatness - was only used as a "pilot test" and thus no strain gauges were applied on it. Therefore, it was decided to exclude girder G0 from numerical analyses, since no "measured" stresses were available to check the reliability of the FE results for this girder.
The test girders have been analysed with the commercial FE code ADINA. Eight-node shell elements were used in the FE model, with element size of 75 x 75 mm^2 for the web. For the quadrature, four Gauss points have been used in the plane of each element. Moreover, three integration layers (i.e. three sectional points) through the thickness of the shell were adopted in order to "capture" the secondary bending stresses at the boundary of the web panel. Non-linear large displacement theory was employed in the FE-analyses.
The structural steel was modelled as an isotropic material and a von Mises yield surface was used. The yield stress and the ultimate strength were taken from the tensile test results. The material curve was chosen according to BSK 99. Such a curve has the first "knee" at the point with co-ordinates (f_y/E, f_y). It should be noted that the branch of the material curve with abscissas greater than f_y/E is normally of minor importance for web breathing analyses. In fact, as shown by both experiments and FE-analyses, local yielding in the web is achieved only for relatively large applied loads. These loads give rise to a state of stress along the boundary of the web panel, which is believed to be considerably greater than the state of stress caused by common service loads in a bridge. The webs of the girders were given the geometrical imperfections (out-of-flatness) obtained from laboratory measurements.
Girders and loads - disregarding the non-load-carrying stiffener placed in one of the web panels (see Figure 10) and the web imperfections - are symmetrical with respect to the vertical plane through the mid-span of

the girder and orthogonal to the girder longitudinal axis. Therefore, in order to reduce the time of analysis, it was decided to model each girder as two separate halves (see Figure 10) using adequate boundary conditions. Thus, it was implicitly assumed that shape and magnitude of the initial web imperfections (and eventually the presence of a non-load-carrying stiffener) at one half of the girder (i.e. the part of girder at the left or right hand side of the mid-span) have a negligible influence on the behaviour of the other girder half.

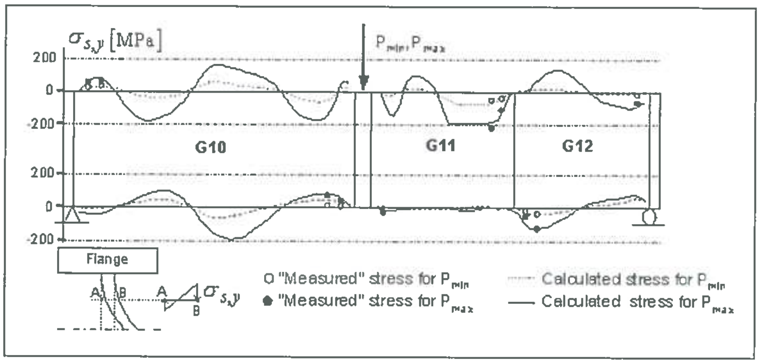


Figure 11: Variation of the surface stresses perpendicular to the flanges along the boundaries of the web panels in girder G1: largest "measured" values for P_{min} and P_{max} respectively in comparison with calculated values.

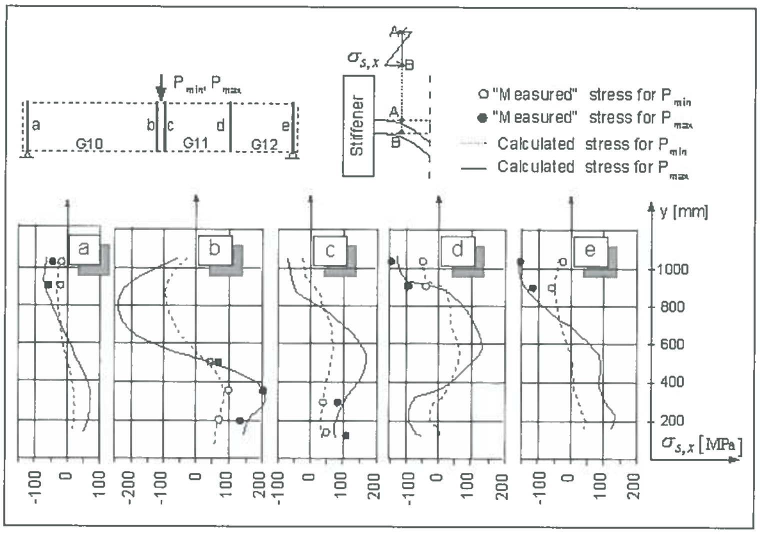


Figure 12: Variation of the surface stresses perpendicular to the vertical stiffeners along the boundaries of the web panels in girder G1: "measured" values in comparison with calculated values.

5. ANALYSIS OF STRESSES ALONG THE WEB BOUNDARIES

In general, satisfactory agreement between measured and calculated stresses was achieved for all the analysed girders. Figure 11 shows the variation of the surface stresses perpendicular to the flanges along the boundaries of the web panels of girder G1. Similar trends for the stress curves were observed for the other analysed girders.

Membrane stresses along the web panels perpendicular to the flanges were generally relatively small, except at some corner areas, i.e. at the connection between flanges and stiffeners. The number of peaks in the stress curves along the flanges, see Figure 11, coincides with the number of buckles (i.e. "half waves") that form in the web panel during loading. While girders G1 and G2 exhibited four peaks in the web panel with α=3, only two major peaks were observed in girder G2, web panel G20. This is due to the fact that only two major buckles formed in the postbuckling range. Stresses perpendicular to the fillet weld at the tension flange, were generally rather small, probably due to the beneficial effect of the bending moment. In the lower part of the girders, in fact, the bending moment "stretches" (read: "stabilises") the web longitudinally and hence limits the out-of-plane deflections induced by shear buckles. However, at a certain distance from the mid-span of the girder, the bending moment has only a small influence on preventing out-of-plane deflections. As a consequence, here relatively large secondary bending stresses perpendicularly to the fillet weld may occur, even in the tensile zone of the web. Due to the particular inclination of the shear buckles, these stresses are usually greatest in the central zone of the web with larger aspect ratio (i.e. α=3).

Figure 12 shows the variation of surface stresses perpendicular to the stiffeners along the boundaries of the web panels in girder G1 . Similar trends were observed for the other girders.

Both FE analyses and strain gauge measurements showed that membrane stresses at the boundary of the web panel – perpendicular to stiffeners "b", "c" and "d" – is a non-linear function of y. Furthermore, membrane stresses taken at a given point in the tension zone of the web were generally much larger than membrane stresses taken at the corresponding point in the compression zone of the web. This is due to the well-known "bowing effect", which causes a redistribution of the longitudinal web stresses in the postbuckling range and thus a shift of the neutral axis towards the tension flange. However, even though in theory this phenomenon should only occur when the stresses exceed the theoretical critical stresses, in practice it occurs as soon as the girder starts to be loaded. This is due to the initial out-of-flatness of the web, which – especially in very slender girders – makes the web plate deflect already at small applied stresses.

The maximum stress ranges along the boundaries of the web panels, perpendicularly to the longitudinal direction of the welds, were generally found, either close to one of the transverse stiffeners placed directly below the pulsating load (locations "b" and "c") or close to the compression flange, see Figure 13. These stress ranges were normally largest in the panel with aspect ratio α=3 and in one case, i.e. girder G3, in a panel with aspect ratio α=1.5 in the zone of larger bending moment. Figure 13 gives a picture of the magnitude of minimum and maximum surface stresses computed or observed in "critical" zones of the girder. Girder G2 had levels of maximum and minimum loads (i.e. P_{max} and P_{min}) higher than the other girders. This explains the fact that the highest stress levels were normally observed in girder G2 (see Figure 13).

It is worth pointing out the importance of the level of maximum and minimum loads that were used during experiments. It is easy to convince oneself about this importance, by studying, for example, Figure 14.

As can be seen in Figure 14, at location "b" (i.e. along the boundary of the panel with larger aspect ratio, close to a load-carrying transverse stiffener) the surface stresses in the web, perpendicular to the transverse stiffener, and especially the stress ranges, may increase considerably at higher load levels. In fact, at an ordinate y between 400 and 600 mm from the bottom flange, for two similar load ranges (i.e. 250 kN and 240 kN), the corresponding stress range $\Delta\sigma_{s,x}$ is significantly greater in the case of higher load levels.

Moreover, although the maximum load adopted during the fatigue test of girder G2 was considerably smaller than the theoretical collapse load, surface stresses in this girder exceeded the yield stress (i.e. f_{yw}=420 MPa, for the web in G2) at locations "b" and "f0". Thus, at these locations the web was subjected to a local low-cycle fatigue loading, which caused a relatively premature cracking of the girder. However, it is doubtful – at least for the working (or service) loads currently encountered for normal steel girders – that such high load levels as those used in girder G2 are likely to occur.

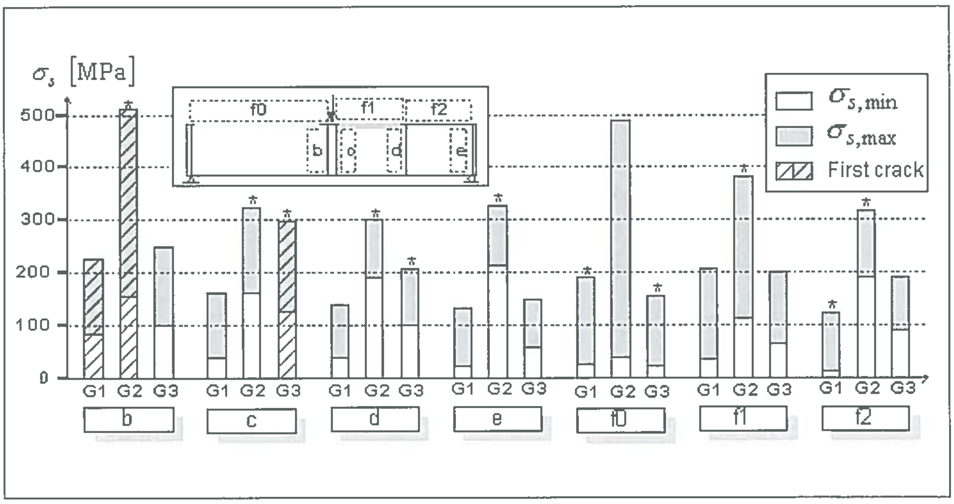


Figure13: Maximum and minimum surface stresses calculated along the boundaries of the web panels, in the zones most susceptible to fatigue cracking due to breathing. The star (*) indicates that at this location strain gauges were applied at -or adjacently to - the node were the stresses were computed.

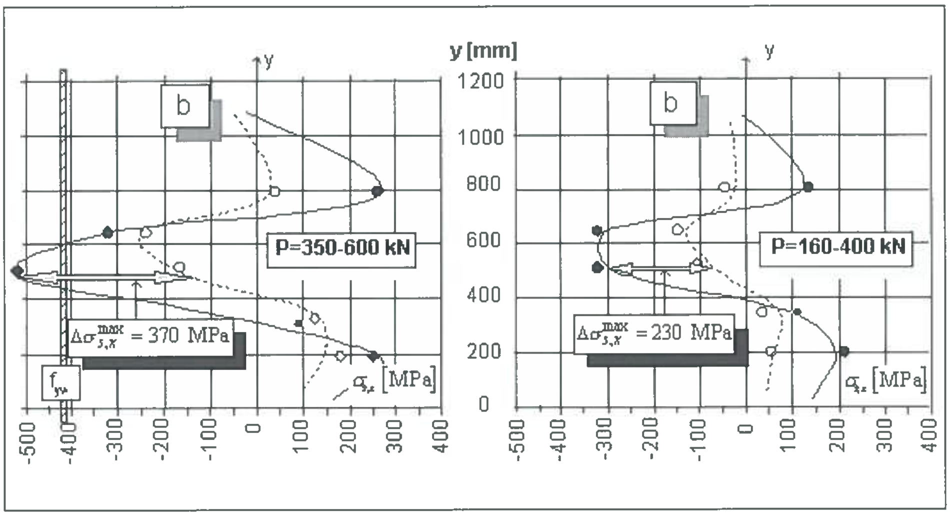


Figure 14: Computed and "measured" surface stresses of girder G2 at location "b". Dotted and continuous lines represent the computed stresses under the action of minimum and maximum load respectively.

6. CAUSES OF FATIGUE CRACKS AND S-N CURVE

Fatigue cracks normally occurred in zones were surface stresses at the boundary of the web panel had a peak. However, although in some cases the recorded stress ranges at locations "f0"-"f2" were larger than at locations "b"-"e" (see Figure 13), the first crack in girders G1-G3 was always observed at one of the latter locations. The reasons of this behaviour are presumably two, namely:

i) the fillet welds between flange and web were performed mainly automatically, whilst the fillet welds between stiffener and web were performed manually. Normally, the fatigue performance of the former is better than the fatigue performance of the latter;
ii) at the connection with a flange, the web is subjected to largely predominant bending stresses. On the other hand, at the connection with a stiffener – even though most of cracks initiated close to the neutral axis – nominal postbuckling membrane stresses in the web are generally larger than at the connection with a flange. The fatigue performance of a cruciform welded joint subjected to bending action is generally significantly better than an identical joint subjected to pure membrane action.More detailed information about crack initiation and propagation of the test girders are reported in Crocetti, R. 2001.

For each analysed girder, i.e. G1 to G3, surface stress ranges are plotted in Figure 15 against the number of cycles to crack initiation, i.e. an S-N diagram. When a crack starts propagating, the state of stresses around the boundary of the web panel is modified. In a girder subjected to web breathing, this may contribute to the initiation of a second fatigue crack at another zone of the web boundary. In fact, in some circumstances two or three different cracks initiated at the same web panel. Moreover, the stress ranges at a given point of the web boundary will not be constant, but it will change as the size of the crack - propagating in another zone of the web boundary – is increasing. On the other hand, the state of stress in a given web panel is not significantly influenced by the presence of a crack in another web panel. Due to these reasons, the surface stress ranges in Figure 15 were computed (or "measured") at two different locations, namely:

i) where the first fatigue crack initiated;
ii) in the point - at a web panel different from the one where the first crack started - which exhibited the maximum surface stress range.

As proposed by some authors , the design curve for a fillet weld detail subjected to pure bending falls

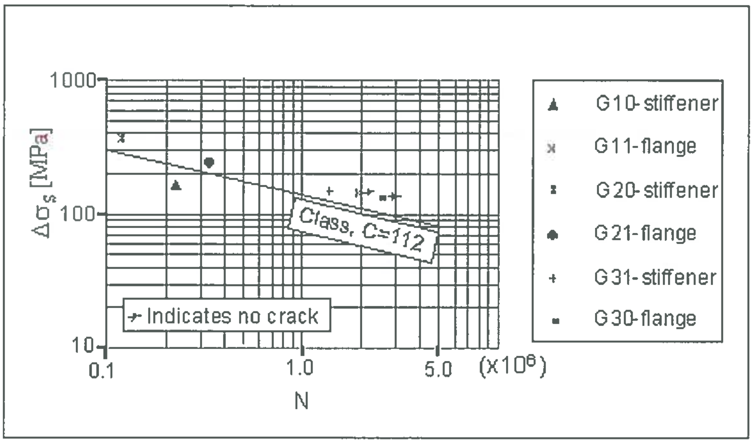


Figure 15: Fatigue test results for crack initiation in comparison with the EC3 design fatigue curve for detail class, C=112.

between the classes C=104 and 116 (where C is the design value of fatigue strength at $2x10^6$ cycles). The web plate at the fillet welds joining flange and web or stiffener and web - at the location where cracks caused by breathing usually occurred – are subjected to predominant bending. Thus, it may be assumed that detail class C=112 of Eurocode 3, for example, can conveniently be adopted for fatigue assessment of fillet welds subjected to web breathing. As shown in Figure 10, the curve C=112 is "safe" for all the surface stress ranges - computed in the three analysed girders – except one, namely the stress range close to the vertical stiffener in web panel G10. Here, the discrepancy is probably due to the presence of a stop/start point in the manually executed fillet weld, which caused an "abnormal" stress intensity factor, hence a premature crack initiation. The S-N curve which best represents the fatigue crack in web panel G10 is the one for class C=80.

7. CONCLUSIONS

A series of fatigue tests was conducted on full-scale, slender plate girders subjected to repeated combined shear and bending loading. Moreover, extensive FE analyses were carried out in order to gain a better understanding of the postbuckling behaviour of the laboratory tested girder. The main findings, based on the results obtained from this research, are summarised below.

i) The maximum loads used in the fatigue testing were well below the ultimate load carrying capacity of the girder. Nevertheless, the process of forming and reforming shear and bending buckles was clearly visible during the tests and fatigue cracks originating from the weld toes, often occurred at a relatively small number of cycles.
ii) The web panels subjected to combined action of high shear and bending are more prone to fatigue cracking due to web breathing than web panels subjected to predominant shear action.
iii) Web panels with large aspect ratios seem to be more susceptible to failure due to web breathing.
iv) Due to the non-linear relationship between load and secondary bending stress at the boundary of the web plate, the load range alone is not sufficient to describe the fatigue behaviour due to web breathing. Thus, the levels of both minimum and maximum load are decisive for the initiation of a fatigue crack.
v) When a relatively low level of the maximum applied load is used in fatigue testing, no significant post-buckling tension field action can develop. Thus, no initiation of fatigue cracks at the corners of the web panel where the tension diagonal is anchored is expected. This is in contrast with some previous research works conducted using a maximum load close to the load-carrying capacity of the girder which showed that fatigue cracking due to breathing occurs mainly at these corners.
vi) The secondary bending stress ranges, which cause fatigue cracking, are strongly depending upon the shape and the magnitude of the initial imperfections. These are of rather stochastic nature and hardly resembling ordinary trigonometric functions.
vii) cracks at a given zone (i.e. close to a stiffener or a flange) always initiated where the range of surface stress perpendicularly to the longitudinal direction of the fillet weld had a maximum. However, in some instances, this stress range was not the largest for the whole girder. In fact, even though the stress range was greatest close to the compression flange, for example, the crack initiated close to a vertical stiffener. The reason of this behaviour is due to different actual quality classes of the fillet welds at different regions of the web boundaries

Finally, it is the opinion of the author, that the approach to the problem of web breathing should be essentially based on experimental data rather than numerical simulations. Hence, the latter should be used only as a qualitative rather than a quantitative tool (for example, in order to estimate the influence of a certain parameter on the magnitude of stresses in a given zone). In fact, the fatigue performance of girders subjected to web breathing depends, among other things, upon weld size and weld imperfections, especially on the location of such imperfections. An appropriate modelling of welds would result in an enormous increase of complexity of the global numerical model of the girder. Therefore, the details of the welds are generally completely disregarded in such kind of models.

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