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FATIGUE DESIGN OF WELDED JOINTS ACCORDING TO EUROCODE 3

by

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ABSTRACT

A survey is given about the factors influencing the fatigue of welded joints. Each factor is commented considering the rules of the Eurocode 3. Numerical examples illustrate the task of students concerning the home-work in this theme. Flow charts show how the numerical examples can be solved by a computer.

SYMBOLS

A	cross-sectional area
a_f, a_w	throat sizes of fillet welds
b	plate width, width of a square hollow section
E	modulus of elasticity
F	force
f_y	yield stress
h	beam height
I	moment of inertia
i	radius of gyration
K	factor for the end restraint of a compressed strut
L	beam or strut length
L_w	weld length
M	bending moment
N	number of cycles
t	thickness
t_f, t_w	flange and web thickness, respectively
t_g	gusset plate thickness
W_o	required section modulus
β	limiting web slenderness

γ_{Ff} safety factor for fatigue load
γ_{M1} safety factor for buckling
γ_{Mf} safety factor for fatigue strength
δ limiting flange slenderness

$\varepsilon = \sqrt{235/\sigma_{max}}$

λ column slenderness ratio
$\bar{\lambda} = \lambda/\lambda_E$ reduced slenderness ratio
$\lambda_E = \pi\sqrt{E/f_y}$ Euler slenderness
$\rho_\perp$ resultant of perpendicular stress components
σ, τ normal and shear stress, resp.
$\Delta\sigma$, $\Delta\tau$ normal and shear stress ranges, resp.
$\Delta\sigma_C$, $\Delta\tau_C$ reference values of the fatigue strength at 2×10^6 cycles
$\sigma_\perp$, $\tau_\perp$, $\tau_\parallel$ stress components in a weld
χ overall buckling factor

1. AN EXAMPLE OF STRUCTURAL FRACTURE DUE TO FATIGUE OF WELDED JOINTS

As a tragic case of a structural fatigue failure, for instance, the catastrophe of the semi-submersible Kielland offshore platform may be mentioned [1]. The platform capsized suddenly on March 27, 1980 with loss of 123 lives while 89 persons were rescued. Hydrophones were attached to the underside of tubular bracings (Fig.1, Fig.2.). Installing the hydrophones has been performed by cutting a hole in the brace and by poor quality fillet welds, from which fatigue cracks propagated in bracings, the fracture of bracings caused a complete separation of a main leg and the disaster of the platform.

2. FACTORS INFLUENCING THE FATIGUE OF WELDED JOINTS

1/ *Parent material:* steel, aluminium alloy, plastic. The yield stress does not affect the fatigue strength, the fatigue strength data given by Eurocode 3 [2] are the same for steels Fe 360 and Fe 510. Fatigue strengths of aluminium alloy welded joints are proposed e.g. by Ogle [11].

2/ *Welding technology:* for some new technologies fatigue strength data do not exist yet.

3/ *Residual welding stresses:* in our courses we treat the residual welding stress and distortions in a separate section showing that these stresses affect the brittle fracture, fatigue and buckling strength of structural elements. The fatigue is influenced by the high tensile residual stresses around the welds.

4/ *Type of joint:* a detailed categorization of joints with the corresponding fatigue strength is given by Eurocode 3.

5/ *Weld geometry:* the geometry of toes of butt and fillet welds can be improved by several methods such as grinding or TIG remelting, but an additional fabrication cost arises [6].

6/ *Weld defects:* at the ends of welds, at points of electrode changing, lack of penetration in cruciform joints. The weld quality and inspection /by non-destructive testing/ play an important role, since, if we have a reliable information on the weld quality produced by a technology, significant inspection cost savings may be achieved.

7/ *Load spectra:* the introduction of an equivalent constant wave load using the Palmgren-Miner rule enables designers to consider the load in a more realistic mode for the investigated structural application.

8/ *Stress range:* the most important factor for the fatigue crack propagation. The recent studies showed that the fatigue life depends on the stress range $\Delta\sigma = (\sigma_{max}-\sigma_{min})/2$ rather than on the σ_{max}, thus, the fatigue life is characterized by the $\Delta\sigma$ - N curves, where N is the number of load cycles.

9/ *Number of cycles:* recent experiments showed that the fatigue of some welded structural parts should be characterized by curves different from the classical $\Delta\sigma$- N curve having an asymptote at $N = 2 \times 10^6$ cycles. For instance, welded joints of hollow sections, Eurocode 3 gives a diagram which has a cut-off point at $N = 10^8$.

10/ *Plate thickness:* recent experiments with specimens of realistic dimensions showed that the older experimental results obtained with small size specimens should be corrected. Eurocode 3 prescribes a correction for thicknesses over 25 mm.

11/ *Stress state:* in most cases not only a normal stress, but also a shear stress component arises. For these cases Eurocode 3 gives an interaction formula.

12/ *Post-welding treatments:* used for the decreasing the residual welding stresses or for improving the weld toe geometry [7].

13/ *Special phenomena:* the interaction of fatigue crack propagation and local buckling due to the breathing of a thin web plate in a welded plate girder [10]; fatigue cracks arising in crane runway girders due to local pulsating torsional loading caused by the wheel load [4], [8].

14/ *Environmental effects:* the fatigue strength data given by Eurocode 3 are valid only for temperatures less than 150°C and for structures with suitable corrosive protection. The corrosion fatigue is a very dangerous failure mode, it should be considered using data from special publications.

15/ *Joints of hollow sections in tubular trusses* are treated in Eurocode 3, but improved international rules are in preparation considering the recent research results, thus, we are waiting for these new rules.

16/ *Economic aspects*: a welded structure should be not only safe, but also economic. These two main aspects can be fulfilled using the methods of optimum design, which is our main research theme [5]. The objective function (weight or cost of a structure) should be minimized fulfilling the design constraints. The constraints are related to static stress, stability, fatigue, vibration, stiffness and fabrication. In numerical examples it is shown how the fatigue constraint determines the optimal dimensions of a welded structure. We have adapted and applied several computer programs of mathematical optimization methods [9].

3. FATIGUE DESIGN RULES OF THE EUROCODE 3

In the Part 1.1 general rules and rules for steel buildings are given. A problem is that the other parts are now in preparation, so the special fatigue problems of bridges or cranes are not treated. The most important rules are the categories of welded joints and the related $\Delta\sigma$ - N diagrams for different categories.

4. NUMERICAL EXAMPLES

In the home-work students should calculate the design task of three different structural parts or simple structures. In the solution they can use hand or computer calculation.

4.1 Optimum design of a welded box beam (Fig.3.)

This example illustrates the effect of the fatigue constraint on the minimum weight of a welded box beam constructed with transverse diaphragms built-in by fillet welds.

The optimal dimensions of the cross-section should be calculated in two cases: without and with post-welding treatment (TIG remelting) of the toes of transverse fillet welds. The fatigue strength of a TIG remelted fillet weld is given from the published test results [7]. Besides the fatigue constraint other constraints are defined relating to the local buckling of flange and web (Fig.3.) plates according to the Eurocode 3.

The box beam (Fig.3) of span length L = 2 m is loaded by a force pulsating between 0 and F_{max} = 350 kN, thus $\Delta F = F_{max}/2$. The required section modulus without treatment for $N = 2x10^6$ is

$$W_O = \frac{M_{max}}{\Delta\sigma_C \;/\; \gamma_{Mf}} = \frac{175 \times 2 \times 10^6}{4 \times 80 \;/\; 1.35} = 1.4766 \times 10^6 \text{ mm}^3$$

The limiting web slenderness is

$$\frac{1}{\beta} = 69\ \varepsilon \quad ; \quad \varepsilon = \sqrt{\frac{235}{1.1 \times 160 \;/\; 1.35}} = 1.34 \;;\; \frac{1}{\beta} = 92.6$$

The optimal web height is

$$h_{opt} = \sqrt[3]{\frac{3\ W_o}{4\ \beta}} = \sqrt{0.75 \times 92.6 \times 1.4766 \times 10^{\ 6}} = 468 \rightarrow 470 \text{ mm}$$

$t_w \;/\; 2 = \beta h = 5$ mm.

$$b_{opt} = h\sqrt{\beta \;/\; \delta} = 468\ \sqrt{56.3 \;/\; 92.6} = 365 \text{ mm}$$

with the flange limiting slenderness $1/\delta = 42\ \varepsilon = 56.3$

$t_f = \delta\ b = 7$ mm (rounded value).

$A_{min} \doteq ht_w + 2\ bt_f = 9810$ mm².

Similar calculation for post-welding treatment, considering

$\Delta\sigma_C$ = 112 MPa, yields $W_o = 1.0547 \times 10^6$ mm³, $1/\beta = 78.3$, h_{opt} = 400 mm, $t_w /2 = 5$, b = 310, $t_f = 7$, A_{min} = 8340 mm², i.e. with post-velding treatment 18% savings in weight can be achieved. Fig.4 shows the flow chart of the calculation. The program runs under Borland C++.

4.2 Optimum design of a welded I-beam connected to a column by fillet welds (Fig.5.)

For the calculation of the optimal dimensions of the welded I-beam the constraint on fatigue of the parent material in the vicinity of the toes of fillet welds is used. The optimal fillet weld sizes are calculated considering the constraints on fatigue of roots of fillet welds with partial penetration. The local buckling constraints for flange and web are also used.

With data of F = 300 kN and L = 2 m and the maximal bending moment M_{max} = FL the required section modulus is

$$W_o = \frac{M_{max}}{\Delta\sigma_C \ / \ \gamma_{Mf}}$$

where $\Delta\sigma_C$ is the fatigue strength range, for N = $2x10^6$ cycles it is $\Delta\sigma_C$ = 71 MPa, and the safety factor for fatigue strength γ_{Mf} = 1.25. Thus

$$W_o = \frac{300 \times 2 \times 10^6}{71/1.25} = 10.56 \times 10^6 \ \mathrm{mm}^3$$

The limiting web slenderness ratio is

$$\frac{1}{\beta} = 69 \ \varepsilon; \quad \varepsilon = \sqrt{\frac{235}{\gamma_{M1}\Delta\sigma_C \ / \ \gamma_{Mf}}} = \sqrt{\frac{235}{1.1 \times 71/1.25}} =$$

$$= 1.94; \quad \frac{1}{\beta} = 134.$$

The optimal web height is

$$h_{opt} = \sqrt[3]{\frac{3 \ W_o}{2 \ \beta}} = \sqrt{1.5 \times 134 \times 10.56 \times 10^6} = 1285 \ \mathrm{mm},$$

rounded 1300 mm;

$t_w = \beta h = 9.6$ rounded 10 mm; $b = h\sqrt{\dfrac{\beta}{2\,\delta}} = 1285\sqrt{\dfrac{54}{2 \times 134}} = 579$ rounded 580 mm, since $1/\delta = 28$ $\varepsilon = 54$; $t_f = \delta b = 10.7$ rounded 11 mm.

The maximal normal tensile stress in the upper flange is

$$\sigma = \frac{M_{max}}{W_x} = \frac{300 \times 2 \times 10^6}{2\,\beta\, h^3\,/\,3} = 55 \text{ MPa}$$

To calculate the required fillet weld throat size a_f we reduce the σ in parent metal into the fillet weld based on the following equality (see Fig.3)

$$\sigma\, t_f = 2\,\rho\perp\, a_f$$

from which

$$\rho\perp = \sigma t_f\,/\,(2\, a_f)$$

and the two stress components

$$\sigma\perp = \tau\perp = \frac{\rho\perp}{\sqrt{2}} = \frac{\sigma\, t_f}{2\sqrt{2}\, a_f} = \frac{55 \times 11}{2\sqrt{2}\, a_f} = \frac{213.9}{a_f}$$

For root cracking of partial penetration fillet welds and for $N = 2 \times 10^6$ cycles we use $\Delta\sigma_C = 36$ and $\Delta\tau_C = 80$ MPa. According to the Eurocode 3, the fillet weld should be checked using the following interaction formula

$$\left[\frac{\gamma_{Ff}\,\Delta\sigma}{\Delta\sigma_C\,/\,\gamma_{Mf}}\right]^3 + \left[\frac{\gamma_{Ff}\,\Delta\tau}{\Delta\tau_C\,/\,\gamma_{Mf}}\right]^5 \le 1$$

With $\Delta\sigma = \Delta\tau = 213.9/a_f$, $\gamma_{Ff} = 1.0$ and $\gamma_{Mf} = 1.25$ one obtains

$$\frac{409.7}{a_f^3} + \frac{417.0}{a_f^5} \le 1$$

from which $a = 8$ mm /rounded value/.

In the fillet welds connecting the web plate, besides the perpendicular stress components

$$\sigma_\perp = \tau_\perp = \frac{\sigma\ t_w}{2\sqrt{2}\ a_w} = \frac{55 \times 10}{2\sqrt{2}\ a_w} = \frac{194.5}{a_w}$$

a parallel stress component should be calculated as well

$$\tau_\parallel = \frac{F}{ht_w} \cdot \frac{t_w}{2a_w} = \frac{F}{2ha_w} = \frac{300 \times 10^3}{2 \times 1300a_w} = \frac{115.4}{a_w}$$

In this case we use, according to the Eurocode 3, the following stress components in the interaction formula

$$\Delta\sigma = \sqrt{\sigma_\perp^2 + \tau_\perp^2} = \rho_\perp = \sqrt{2}\ \sigma_\perp = 275 / a_w$$

and $\Delta\tau = \tau_\parallel = 115.4 / a_w$

From $\left[\frac{275}{36\ a_w / 1.25}\right]^3 + \left[\frac{115.4}{80\ a_w / 1.25}\right]^5 \le 1$

we obtain $a_w = 9.5$, rounded 10 mm.

Fig. 6. gives the flow chart of the calculation.

4.3. Design of the connection of a compressed strut to a gusset plate (Fig.7.)

The strut of length L = 7.5 m is cyclic loaded in tension-compression by a pulsating force F = 190 kN. The cross-section should be a square hollow section designed for overall buckling. For maximal stress in overlapping plate elements $\Delta\sigma_c$ = 45 MPa should be used. The overall buckling constraint is

$$\frac{\gamma_{M1} F}{A} \le \chi\ \Delta\sigma_c \quad ; \quad A = 4\ bt$$

χ is the overall buckling factor. For the calculation we use as an approximation to the Eurocode 3 buckling curve "b" the simpler Japanese JRA buckling curve as follows

$$\chi = 1.109 - 0.545\ \bar{\lambda} \qquad \text{for} \quad 0.2 \le \bar{\lambda} < 1$$

$$\chi = 1/(0.773 + \bar{\lambda}^2) \qquad \text{for} \quad \bar{\lambda} > 1$$

where $\bar{\lambda} = KL/(i\ \lambda_E)$; $i = \sqrt{I/A}$; $\lambda_E = \pi\sqrt{E/f_y}$

For pinned ends K = 1, for square hollow sections $i = b/\sqrt{6}$ in the case of f_y = 235 MPa $\varepsilon = 1$, $\lambda_E = 93.9$

$$\bar{\lambda} = \frac{L\sqrt{6}}{b\ \lambda_E} = \frac{7500\sqrt{6}}{93.9\ b} = \frac{195.65}{b}$$

To avoid local buckling we use the limiting slenderness

$$\text{with}\quad \varepsilon = \sqrt{\frac{235}{45\ /\ 1.1}} = 2.4\ ;\ \delta_{sL} = \left[\frac{b}{t}\right]_L = 42\ \varepsilon = 100.$$

Assuming that $\bar{\lambda} < 1$ we obtain an equation for b as follows

$$\frac{1.1 \times 19000}{0.04\ b^2} = \left[1.109 - \frac{0.545 \times 195.65}{b}\right] 45$$

this yields the following quadratic equation

$$b^2 - 96.149\ b - 104700 = 0$$

from which one obtains b = 375 and t = 3.75 mm. Instead of these values we use a section of 350 x 350 x 6.3 according to ISO 4200 for cold-formed square hollow sections.

Calculation of the overlapping length L_w for the fillet welded connection according to the Fig.4, with the total weld length $2L_w + b$, for shear we take $\Delta\tau_C$ = 80 MPa and a_w = 5 mm

$$\frac{F}{(2L_w+b)\ a_w} \le \frac{\Delta\tau_C}{\gamma_{Mf}} = \frac{80}{1.25} = 64\ \text{MPa}$$

from which L_w = 122 rounded 150 mm.

Calculation of the required gusset plate thickness t_g according to Eurocode 3 with the category $\Delta\sigma_C$ = 63 MPa. From

$$\frac{F}{(L_w+b)t_g} \le \frac{63}{\gamma_{Mf}} = 50.4\ \text{MPa}$$

one obtains t_g = 7.5, rounded 8 mm.
Fig.8. shows the flow chart.

5. CONCLUSIONS

Since the fatigue design of welded structural components plays an important role in education, it is necessary to show the complexity of this problem by analysis of the main affecting factors and to give a detailed information about the design rules of the Eurocode 3. It was shown how to incorporate the fatigue constraints into the optimum design of welded structures which enables designers to develop safe and economic structural versions.

6. REFERENCES

1. A. Almar-Ness, P.J. Haagensen et al.(1984) Investigation of the Alexander L. Kielland failure - metallurgical and fracture analysis. J.Energy Resources Technology Trans. ASME 106, 24-31.
2. Eurocode 3 (1992) Design of steel structures. Part 1.1 General rules and rules for buildings. European Prestandard ENV 1993-1-1. Brussels, CEN - European Committee or Standardization.
3. J. Farkas (1993) Fémszerkezetek. Metal Structures - University textbook in Hungarian. Budapest, Tankönyvkiadó.
4. J. Farkas (1983) Optimum design of crane runway girders. Publ. Techn. Univ. Heavy Industry Miskolc Series C. 37, 233-246.
5. J. Farkas (1984) Optimum Design of Metal Structures. Akadémiai Kiadó, Budapest and Ellis Horwood, Chichester UK.
6. J. Farkas (1991) Fabrication aspects in the optimum design of welded structures. Struct. Optimization 3, 51-58.
7. V. Gregor (1989) The effect of surface preparation by TIG remelting on fatigue life. Zváracské Správy - Welding News 39, 60-65.

8. A.R. Ingraffea, K.I. Mettam et al. (1987) An analytical and experimental investigation into fatigue cracking in welded crane runway girders. In "Structural Failure, Product Liability and Technical Insurance. Proc. 2nd Internat. Conference Techn. Univ. Vienna 1986. Ed. H.P. Rossmanith. Inderscience Enterprises" pp.201-223.
9. K. Jármai (1989) Single- and multicriteria optimization as a tool of decision support system. Computers in Industry 11, 249-266.
10. Y. Maeda and I. Okura (1983) Influence of initial deflection of plate girder webs on fatigue crack initiation. Eng. Struct. 5, 58-66.
11. M.H. Ogle (1991) Weld quality specifications for steel and aluminium structures. Welding in the World 29, 341-362.

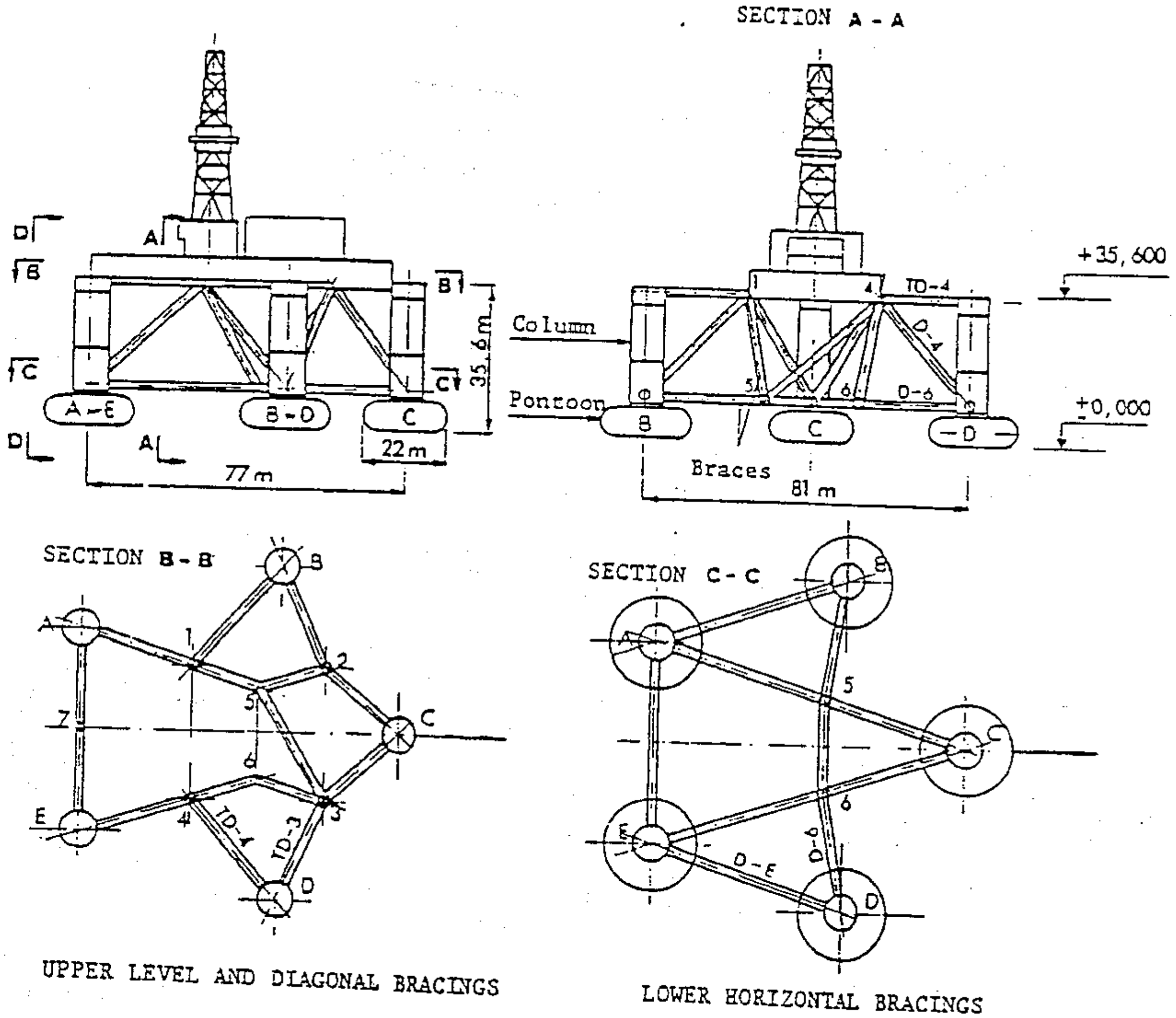

Fig. 1 — Structural arrangement of the Alexander L. Kielland platform

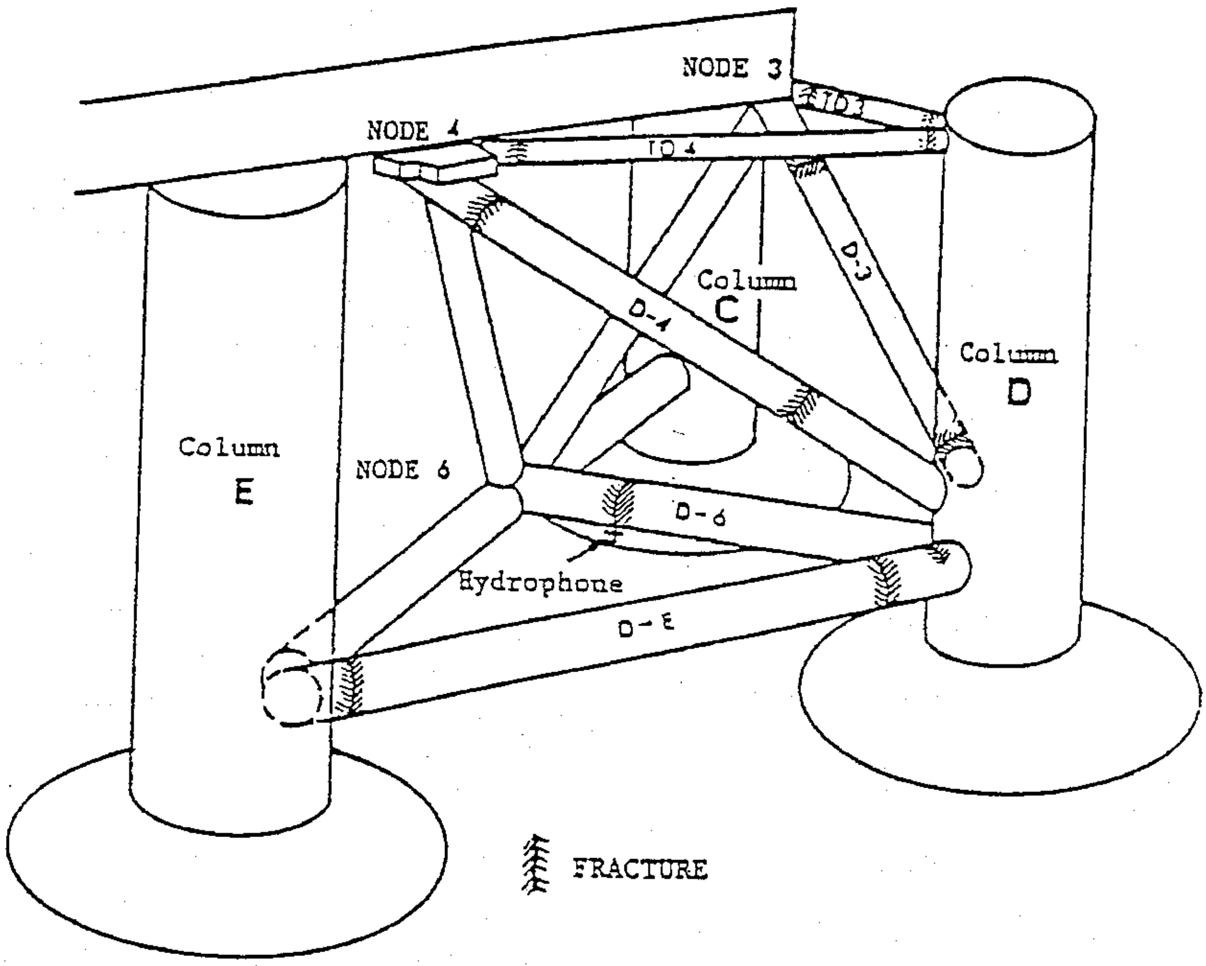

Fig. 2 — Fractures in bracings connected to leg D

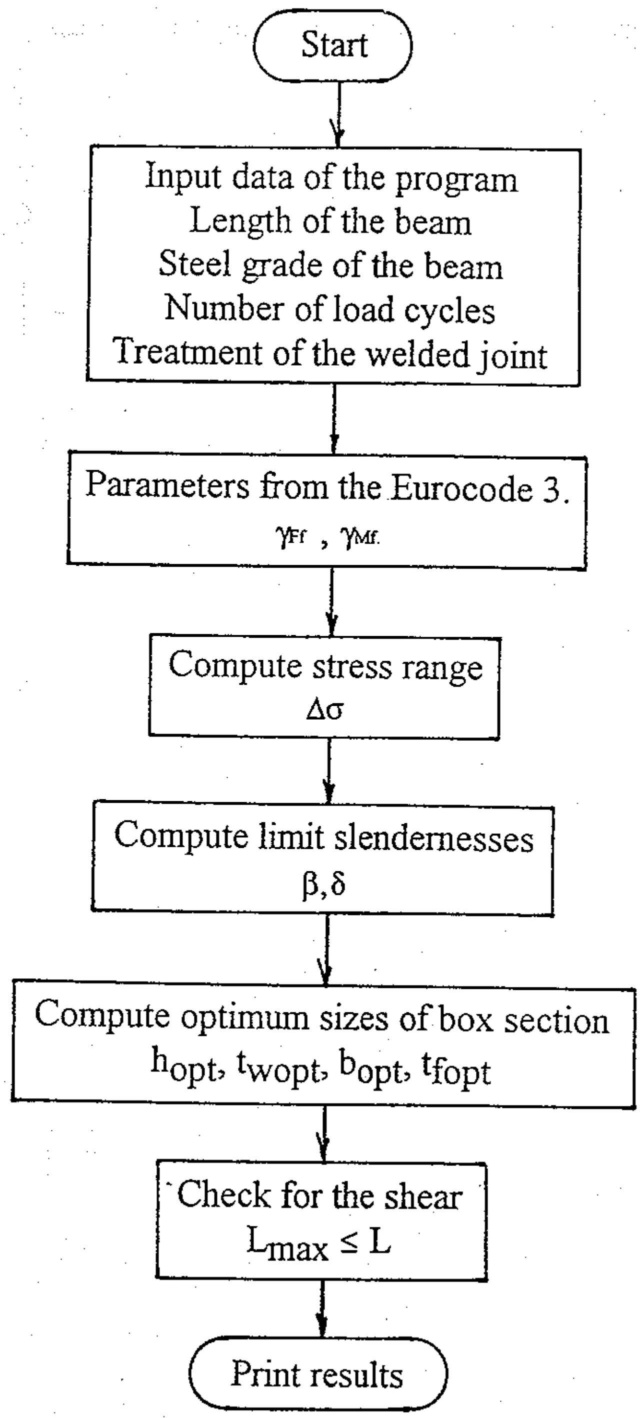

Fig. 4. Flow chart of box beam fatigue design program

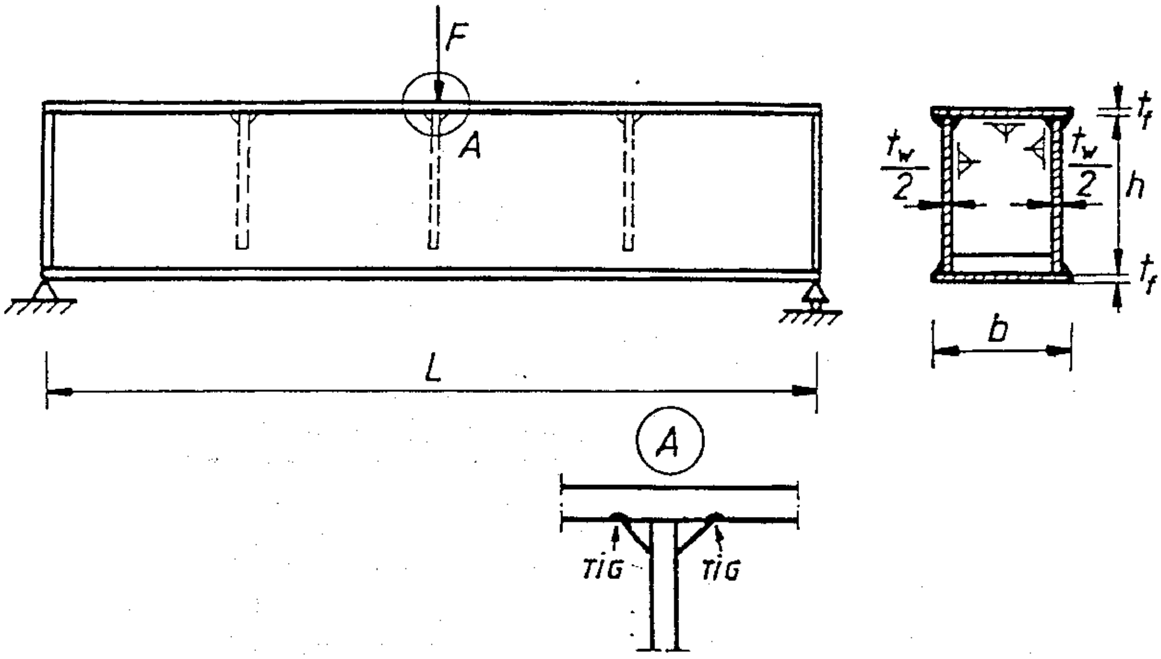
F
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Fig. 3.

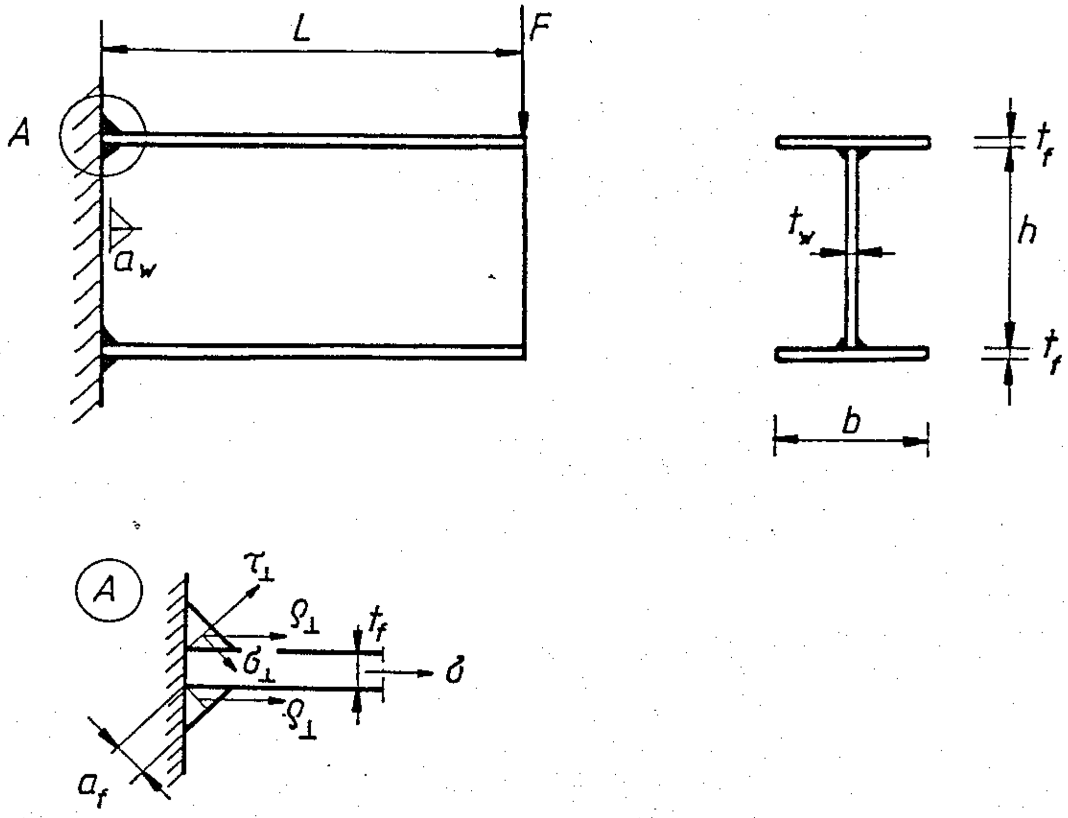
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Fig. 5.

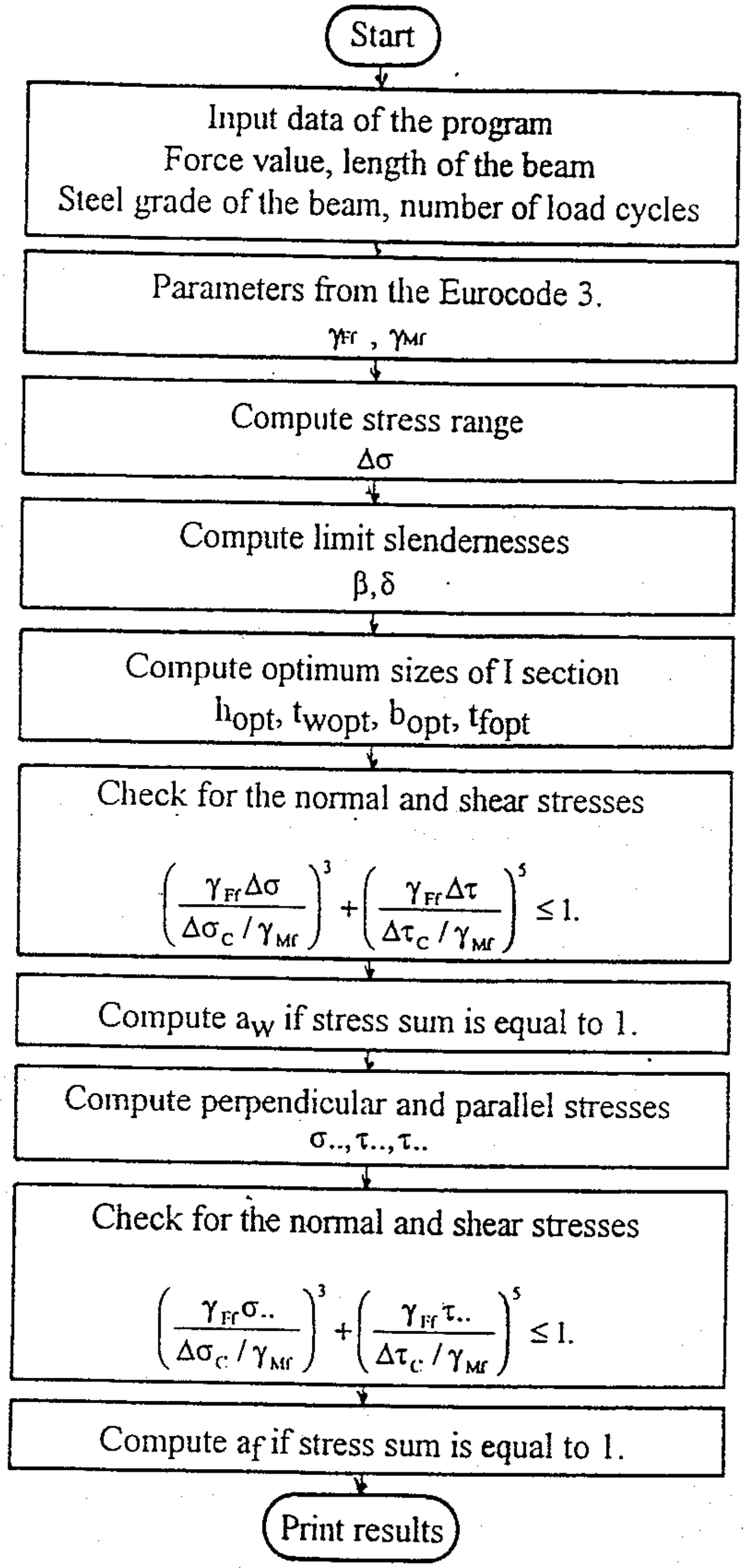

Fig.6 . Flow chart of I beam fatigue design program

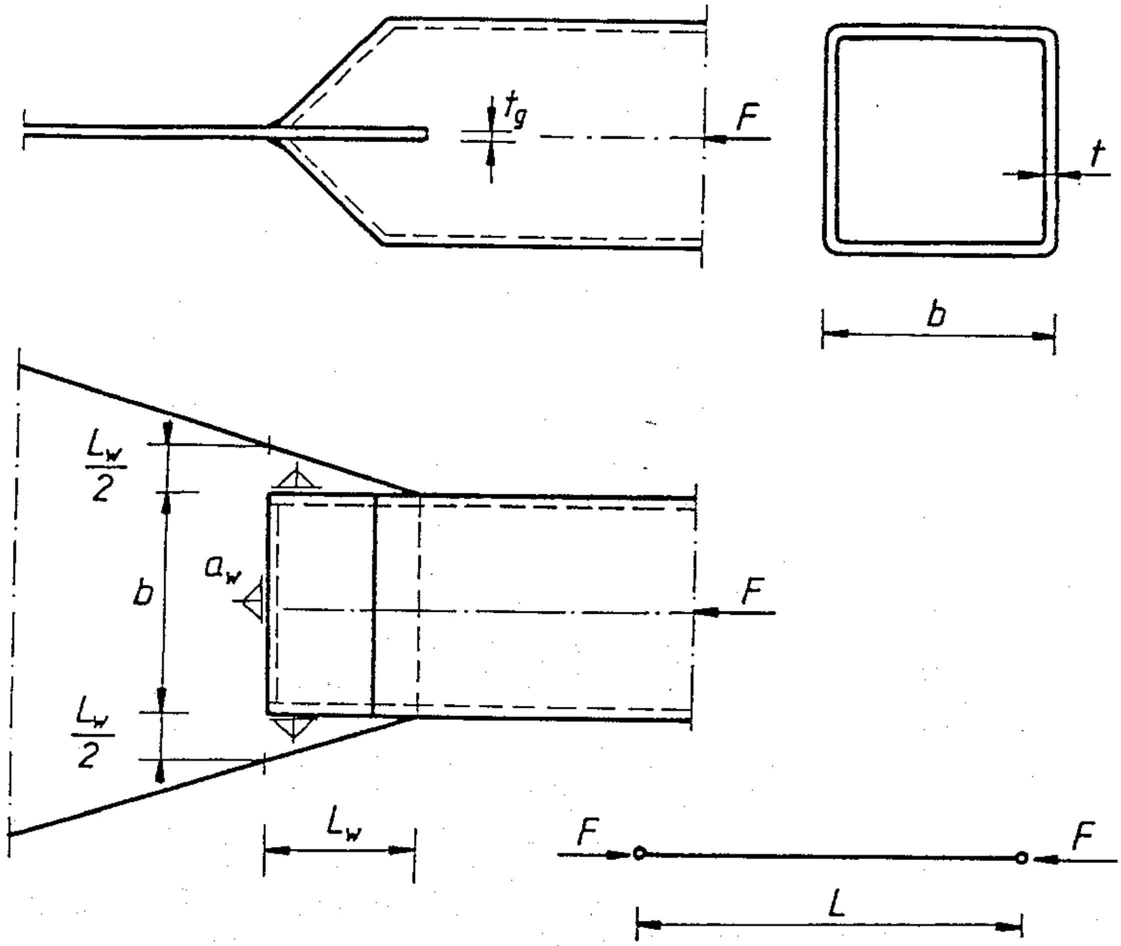

Fig.7.

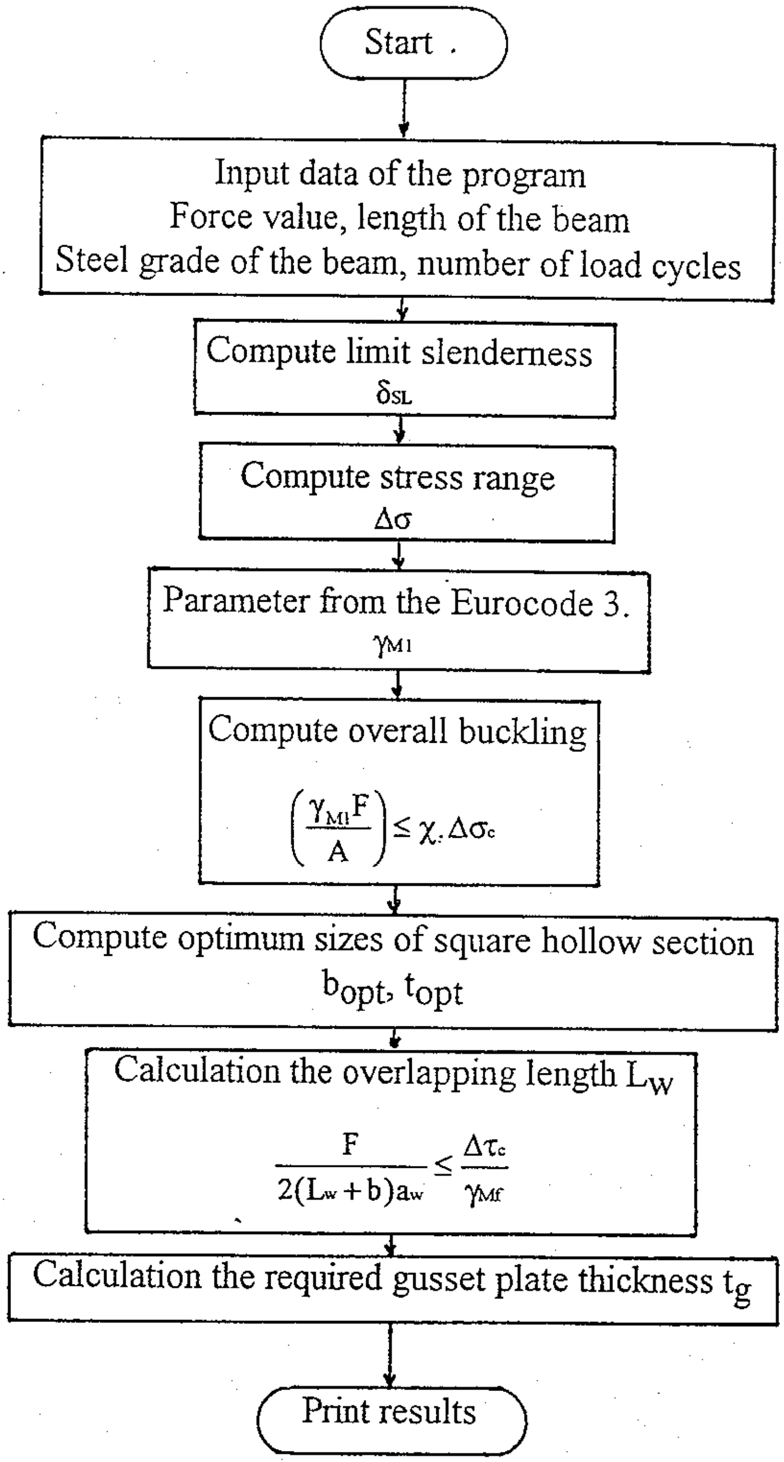

Fig. 8. Flow chart of the program for design of connection of a compressed strut to a gusset plate