Fatigue of bracing in buildings

BS5950 states that buildings subject to fluctuating wind loads do not need to be checked for fatigue but EC3 contains no such statement. Richard Henderson of the SCI considers the issues and illustrates a fatigue check of wind bracing in a conventional building.

Introduction

Clause 2.4.3 Fatigue in BS5950-1:2000, a code specifically for the design of steelwork in buildings, states "Fatigue need not be considered unless a structure or element is subjected to numerous significant fluctuations of stress. Stress changes due to normal fluctuations in wind load need not be considered". The ANSI/AISC 360-16 Specification for structural steel buildings Chapter B clause 11 states "… Fatigue need not be considered … for the effects of wind loading on typical lateral force-resisting systems …". BS EN 1993-1-1 and BS EN 1993-1-9 (Part 1-9) include no such clause but BS EN 1993-1-1 forms the foundation for a series of codes for the design of bridges, towers and other structures. Bridges are routinely checked for fatigue. Other structures such as chimneys and masts may be subject to wind-induced oscillations and need to be checked for fatigue.

The connections at the ends of wind bracing are often made using gusset plates, fillet welded to end plates and beam flanges. Tubular tension/compression bracing members may have bolted spade-end connections fillet welded to end plates.

Fatigue Strength Curves

An introduction to fatigue design was published in NSC magazine last year. Part 1-9 clause 7.1 gives the fatigue strength for nominal stress ranges for a range of details, identified in Tables 8.1 to 8.10. The fatigue strength is defined by a (log$\Delta\sigma_R$) – (log$N$) curve for each detail category as shown in Figure 1. For a constant amplitude nominal stress range, the curve gives the number of cycles to failure or endurance. The curve number is the detail category and is the constant amplitude nominal stress range that will result in failure after 2 million cycles. The curves change in slope at N = 5 million cycles. For nominal stress ranges lower than a certain value known as the cut-off limit $\Delta\sigma_L$, fatigue damage is considered not to occur. The curves are based on the results of tests on large-scale specimens collected over several decades.

Fatigue damage can be calculated for a given detail using the relevant fatigue curve from Part 1-9 to determine the number of cycles to failure N_i for a given stress range i and using Miner's summation for fatigue damage $\Sigma n_i/N_i$ where n_i is the number of occurrences of this stress range over the life of the structure. The fatigue damage should be less than or equal to 1.0 for the detail to be acceptable (see Part 1-9, Annex A clause A5). Some fillet welded details are in the lowest classes of detail identified in Part 1-9 Table 8.5: either detail category 36* or 40.

Wind loads

BS EN 1991-1-4 Annex B includes a graph of the number of times in 50 years that a wind gust load equals or exceeds a given proportion of the once in 50 year gust load, expressed as a percentage (see Figure 2). This curve is introduced in Annex B for use in the procedure for determining the structural factor c_sc_d in wind load calculations. BS EN 1991-1-4 gives no guidance on the use of the curve for fatigue calculations due to gust loads.

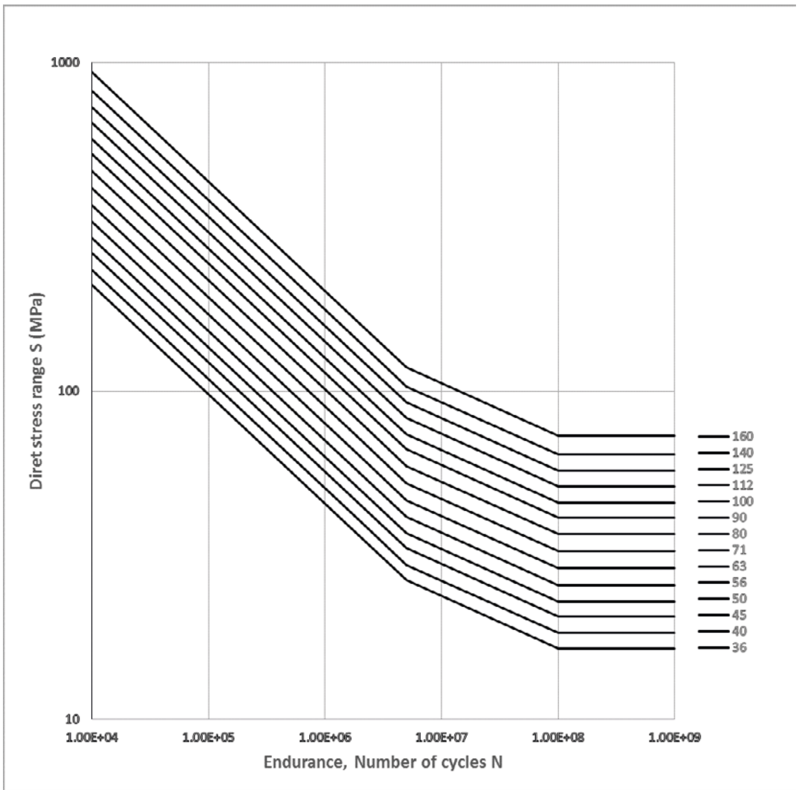

Figure 1: Fatigue strength curves

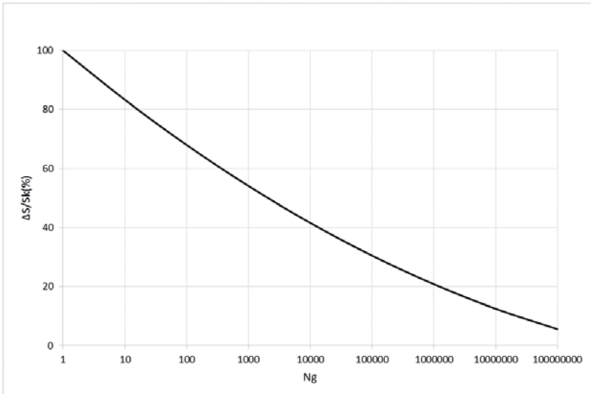

Figure 2: Number of gust loads N_g during a 50 year period

The relationship between the quantities is given as:

$$\frac{\Delta S}{S_k} = 0.7\left(\log_{10}(N_g)\right)^2 + 17.4\log_{10}(N_g) + 100$$

This graph provides the spectrum of stress ranges to which a detail is subjected. An unconsidered examination of the graph suggests that a load equal to 15% of the once in 50 year load ($\Delta S/S_k$ = 15%) occurs about 5 million times during the 50-year design life of the building. If the design load results in a stress equal to yield, a characteristic stress range of 36 MPa (about 355 × (0.15/1.5)) occurs enough times to cause a fatigue failure in a class 36 joint, for which a constant amplitude stress range of 36 MPa causes failure after 2 million cycles. ▶26

▶24 A crude examination such as this neglects a proper assessment of the stress ranges to which the bracing connection details are subjected. The bracing members are usually designed for wind loads and equivalent horizontal forces (EHF). These forces may also be amplified by a factor based on the elastic critical load factor of the building. Although fatigue is an ultimate load case, the load factor on the wind load is defined as unity instead of 1.5. Also, the EHF and amplification factor are intended to allow for global imperfections and second order effects respectively and are therefore not included in fatigue calculations. The stress ranges for the fatigue check are therefore significantly smaller than might initially be imagined.

Design Example

An example fatigue check on a connection detail for a bracing member taken from the design example in SCI's publication P365 Steel building design: medium rise braced frames is illustrative.

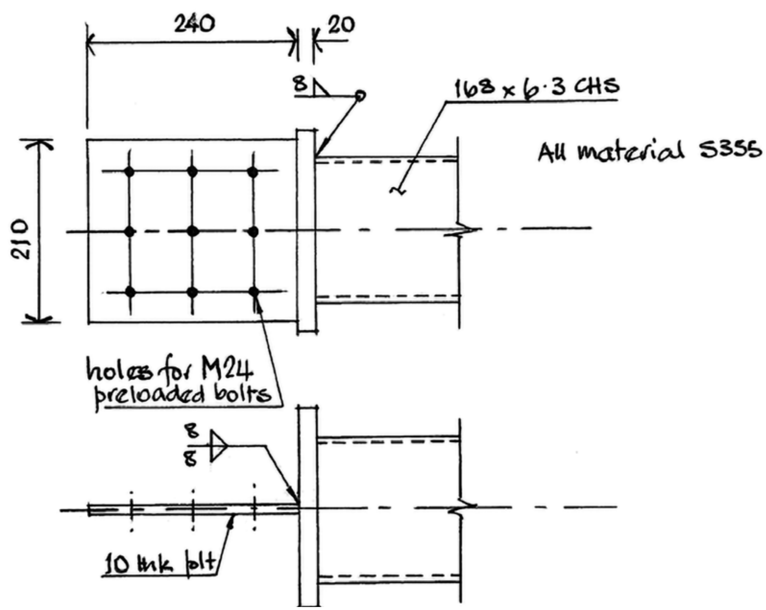

Figure 3: Bracing connection

The ultimate design load in the bracing member from ground to first floor is 539 kN. 60.9% of this force is due to wind load and it includes an amplification factor of 1.17. The unfactored load due to wind alone is therefore:

$$\frac{537.4}{1.17} \times 0.609 \times \frac{1}{1.5} = 187.2\ \text{kN}$$

The bracing member chosen is a 168 x 6.3 CHS in S355 material. A Tee or spade end connection is adopted and the double-sided fillet weld between the end plate on the tube and the projecting plate is designed in accordance with clause 7.6 of BS EN 1993-1-8 which determines the effective lengths of the weld. If the welds are sized according to the design load, as allowed in clause 7.3.1(6) of BS EN 1993-1-8, 8 mm leg fillet welds are adequate (weld throat = 5.7 mm). The connection detail is illustrated in Figure 3.

Fatigue Check

Checks on two welds are necessary for the end connection: the tube to end plate and the end plate to spade end welds. The relevant detail categories are 40 and 36*; the latter category has a modified curve in accordance with clause 7.1(3) Note 3 in Part 1-9. The curves are shown in Figure 4.

Fatigue damage is defined in Annex A para. A.5 of Part 1-9 as:

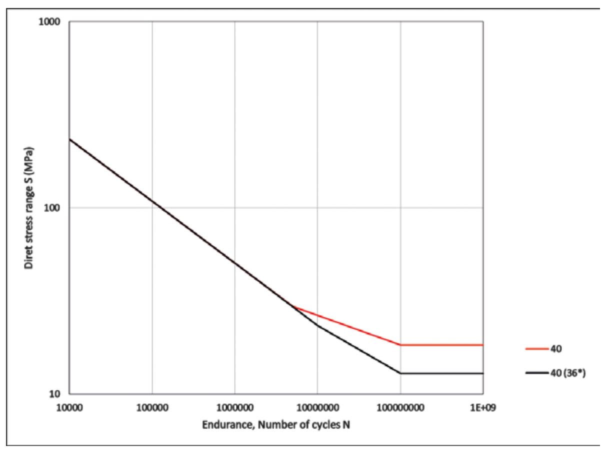

Figure 4: Fatigue strength curves

$$D_d = \sum_i^n \frac{n_{Ei}}{N_{Ri}}$$

where n_{Ei} is the number of cycles associated with the stress range $\gamma_{Ff}\Delta\sigma_i$ for band i in the factored spectrum and N_{Ri} is the endurance in cycles from the fatigue strength curve for a stress range of $\gamma_{Mf}\gamma_{Ff}\Delta\sigma_i$. According to the UK National Annex, $\gamma_{Mf} = 1.1$ and $\gamma_{Ff} = 1.0$.

The factored stress range spectrum is found from Figure 2. Stress ranges $\Delta\sigma_i$ corresponding to equal intervals of $\log_{10}N_g$ along the horizontal axis are considered in calculating the fatigue damage. The values of N_g range between 1.0 at 100% of S_k multiplied by the partial factors and the value of N_g at the factored cut-off limit $\Delta\sigma_L$. 100 intervals are chosen to achieve good convergence. The number of cycles n_{Ei} of the occurrence each stress range is calculated from the spectrum and the number of cycles to failure N_{Ri} for the stress range is calculated from the fatigue strength curve (Figure 3). The ratio of n_{Ei}/N_{Ri} is summed to calculate the fatigue damage.

Taking the details in turn, the effective length of the 8 mm fillet weld between the tube and end plate is 334 mm. The force /mm is:

$$\frac{187}{334} = 0.56 \text{ kN/mm}$$

The throat thickness is 5.7 mm. The fatigue direct stress is:

$\sigma_r = \frac{0.56 \times 10^3}{5.7} = 105 \text{ MPa}$. This stress factored as described corresponds to S_k in the curve in Figure 2. The weld detail class is 40, described as "circular structural hollow section fillet welded end to end with an intermediate plate" in Table 8.6 of Part 1-9.

An example of the steps in the summation are given in the Table 1 for 10 intervals.

Using 100 intervals gives cumulative damage of 0.320.

For the tube to end plate weld, the damage summation equals 0.32 < 1.0 so the detail is satisfactory.

The second detail is the double-sided fillet weld between the end plate and the spade-end. The effective length of the weld between the tube and end plate is 388 mm. The force /mm is:

$$\frac{187}{388} = 0.48 \text{ kN/mm}$$

Index	$\log_{10}N_g$int	n_i	$n_{Ei}=n_i+1-n_i$	$\gamma_{Mf}\gamma_{Ff}\Delta S$	$\Delta\sigma_i$	N_{Ri}	n_{Ei}/N_{Ri}	cum n_{Ei}/N_{Ri}
0	0	1	1	116	116	83000	0.0	0.0
1	0.68	4	3	104	110	97200	0.0	0.0
2	1.36	22	18	90.0	96.8	141000	0.0	0.0
3	2.04	109	87	77.9	83.9	216000	0.0	0.0
4	2.72	526	417	66.8	72.4	338000	0.001	0.002
5	3.40	2520	1997	56.5	61.6	546000	0.004	0.005
6	4.08	12100	9567	46.9	51.7	926000	0.010	0.016
7	4.76	57900	45831	38.1	42.5	1660000	0.028	0.043
8	5.44	277000	219568	30.1	34.1	3230000	0.068	0.111
9	6.12	1330000	1051898	22.8	26.4	8660000	0.121	0.233
10	6.80	6370000	5039407	16.2	19.5	39700000	0.127	0.356

Table 1: Calculation steps for 10 intervals

The fatigue direct stress is: $\sigma_r = \frac{0.48 \times 10^3}{5.7} = 85.2 \text{ MPa}$. This stress when factored corresponds to S_k in the curve in Figure 2. The weld detail class is 36*, described as "root failure in partial penetration Tee-butt joints or fillet welded joint …" in, Table 8.5 of Part 1-9.

For the spade end to end plate weld, the damage summation equals 0.296 < 1.0 so the detail is satisfactory.

Conclusion

The foregoing examples indicate that for a bracing end connection, the predicted fatigue damage according to EC3 Part 1-9 indicates a fatigue life in excess of the normal 50 year design life of a building. This supports the inclusion of clause 2.4.3 in BS 5950:2000 and suggests that following the historical practice in the UK of not carrying out fatigue checks on bracing in conventional buildings is justified when designing to BS EN 1993-1-1 and Part 1-9.

References

1 Introduction to fatigue design to BS EN 1993-1-1, New Steel Construction, September 2018