Benchmark Example No. 19

Fatigue of a Rectangular Reinforced Concrete CS

SOFiSTiK | 2018
1 Problem Description

The problem consists of a simply supported box girder beam of reinforced concrete, as shown in Fig. 1. The structure’s resistance to fatigue shall be verified.

![Figure 1: Problem Description](image)

2 Reference Solution

This example is concerned with the verification to fatigue. The content of this problem is covered by the following parts of DIN EN 1992-1-1/NA [1] [2]:

- Verification conditions (Section 6.8.1)
- Internal forces and stresses for fatigue verification (Section 6.8.2)
- Combination of actions (Section 6.8.3)
- Verification procedure for reinforcing and prestressing steel (Section 6.8.4)
- Verification using damage equivalent stress range(Section 6.8.5)
- Verification of concrete under compression or shear (Section 6.8.7)

3 Model and Results

The properties of the simply supported beam of reinforced concrete with a box cross-section are defined in Table 1. The beam is loaded with three combinations of load cases with calculatoric forces and moments, as presented in Table 1. A verification of its resistance to fatigue is performed at $x = 5 \, \text{m}$ with respect to DIN EN 1992-1-1/NA [1] [2]. The results are given in Table 2.
### Table 1: Model Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Geometry</th>
<th>Loading (at x = 5 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C 35/45</td>
<td>$h = 200.0 \text{ cm}$</td>
<td>LC 911: $V_x = 610 \text{ kN}, M_y = 4575 \text{ kNm}, M_t = -0.19 \text{ kNm}$</td>
</tr>
<tr>
<td>S 500</td>
<td>$b = 600.0 \text{ cm}$, $t = 400.0 \text{ cm}$, $L = 20.0 \text{ m}$</td>
<td>LC 912: $V_x = 660 \text{ kN}, M_y = 4950 \text{ kNm}, M_t = -50.20 \text{ kNm}$</td>
</tr>
<tr>
<td></td>
<td>$A_{s1} = 60 \text{ cm}^2$</td>
<td>LC 913: $V_x = 710 \text{ kN}, M_y = 5325 \text{ kNm}, M_t = 99.78 \text{ kNm}$</td>
</tr>
<tr>
<td></td>
<td>$A_{s2} = 60 \text{ cm}^2$</td>
<td></td>
</tr>
</tbody>
</table>

### Table 2: Results

<table>
<thead>
<tr>
<th>Result</th>
<th>SOF (FEM).</th>
<th>Ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta \sigma_{s,equ}(N^*) \text{ [MPa]}$</td>
<td>74.04</td>
<td>76.98</td>
</tr>
<tr>
<td>$f_{cd, fat} \text{ [MPa]}$</td>
<td>17.06</td>
<td>17.0567</td>
</tr>
<tr>
<td>$\sigma_{cd, max, equ, top} \text{ [MPa]}$</td>
<td>$\leq 14.33$</td>
<td>$\leq 14.33$</td>
</tr>
<tr>
<td>$\sigma_{cd, max, equ, shear, cut} \text{ [MPa]}$</td>
<td>$\leq 10.39$</td>
<td>$\leq 10.34$</td>
</tr>
<tr>
<td>$\frac{\Delta \sigma_{pitk} (N^*)}{\gamma_{s, fat}} \text{ [MPa]}$</td>
<td>152.17</td>
<td>152.2</td>
</tr>
</tbody>
</table>
4  Design Process\footnote{\textsuperscript{1}}

Design with respect to DIN EN 1992-1-1/NA \cite{1} \cite{2}:

STEP 1: Material

Concrete: C 35/45

\[ f_{ck} = 35 \text{ N/mm}^2 \]

\[ \gamma_c = 1.50 \]

\[ f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 0.85 \cdot 35 / 1.5 = 19.83 \text{ MPa} \]

STEP 2: Cross-section

\[ 1/W_{Vz} = 0.8177 \text{ 1/m}^2 \]

\[ 1/W_{Vy} = 0.371 \text{ 1/m}^2 \]

\[ 1/W_T = 0.3448 \text{ 1/m}^3 \]

Minimum reinforcements:

\[ A_{s1} = A_{s2} = 6 \cdot 10 = 60 \text{ cm}^2 \]

\[ A_{sl} = 8.22 \text{ cm}^2/m \]

STEP 3: Load Actions:

Permanent: Loadcase 1

Variable: Loadcase 2, 3

For the determination of the combination calculatoric forces and moments the following superposition types are chosen:

\begin{itemize}
  \item Quasi permanent combination for serviceability - MAXP
  \item Frequent combination for serviceability - MAXF
\end{itemize}

The following combination of actions scenario is investigated for serviceability:

\begin{itemize}
  \item LC 911 G
    \[ \text{MAXP} + \text{MY} : 1.00 \cdot G \]
  \item LC 912 G+2
    \[ \text{MAXF} + \text{MY} : 1.00 \cdot G + \psi_1 \cdot \text{LC 2} \]
  \item LC 913 G+3
    \[ \text{MAXF} + \text{MY} : 1.00 \cdot G + \psi_1 \cdot \text{LC 3} \]
\end{itemize}

\footnote{\textsuperscript{1}}The tools used in the design process are based on steel stress-strain diagrams, as defined in \cite{2} 3.3.6: Fig. 3.10

\footnote{\textsuperscript{2}}The sections mentioned in the margins refer to DIN EN 1992-1-1/NA \cite{1}, \cite{2}, unless otherwise specified.
Combination calculatoric forces and moments at \( x = 5.0 \text{ m} \):

<table>
<thead>
<tr>
<th>LC</th>
<th>( V_y ) [kN]</th>
<th>( V_z ) [kN]</th>
<th>( M_y ) [kNm]</th>
<th>( M_t ) [kNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>911</td>
<td>0</td>
<td>610</td>
<td>4575</td>
<td>−0.189</td>
</tr>
<tr>
<td>912</td>
<td>0</td>
<td>660</td>
<td>4950</td>
<td>−50.20</td>
</tr>
<tr>
<td>913</td>
<td>0</td>
<td>710</td>
<td>5325</td>
<td>99.78</td>
</tr>
</tbody>
</table>

**STEP 4:** Calculation of stresses at \( x = 5.0 \text{ m} \):

The resistance of structures to fatigue shall be verified in special cases.

This verification shall be performed separately from concrete and steel.

The following calculation corresponds to LC 911.

\( \tau_Q = \frac{1}{W_{V_y}} \cdot Q_y + \frac{1}{W_{V_z}} \cdot Q_z \)

where \( Q_y \) and \( Q_z \) are calculated through a proportionate factor \( f_V \), depending on the lever arm of internal forces and the elastic part of \( V_y \) and \( V_z \).

The proportionate factor \( f_V \) is obtained from the internal lever in cracked condition to the un-cracked condition.

\[
\begin{align*}
V_I &= \sqrt{V_y^2 + V_z^2} = \sqrt{0^2 + 610^2} = 610 \text{ kN} \\
V_{II} &= \left( \frac{V_y}{z_{y,II}} \right)^2 + \left( \frac{V_z}{z_{z,II}} \right)^2 \\
V_{II} &= \sqrt{\left( \frac{0}{3.369} \right)^2 + \left( \frac{610}{1.528} \right)^2} = 399.21 \text{ kN} \\
f_V &= \min \left( 1, \frac{V_I}{z_0}, \frac{V_{II}}{z_{II}} \right) = \min \left( 1, \frac{610}{1.782}, \frac{399.21}{399.21} \right) = 0.8576
\end{align*}
\]
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Figure 2: Stress distribution in un-cracked state - $z_1$

Figure 3: Stress distribution in cracked state - $z_{II}$

$Q_y = f_y \cdot V_y = 0.8576 \cdot 0.0 = 0.0 \text{kN}$

$Q_z = f_y \cdot V_z = 0.8576 \cdot 610 = 523.149 \text{kN}$

$\tau_Q = 0.371 \cdot 0.0 + 0.8177 \cdot 523.149 = 427.770 \cdot 10^{-3} \text{MPa}$

$\tau_T = -1/W_T \cdot M_t = -0.34484 \cdot 0.189 = 0.065 \cdot 10^{-3} \text{MPa}$

$\tau = \tau_Q + \tau_T = 427.770 \cdot 10^{-3} + 0.065 \cdot 10^{-3} = 427.835 \cdot 10^{-3} \text{MPa}$

$\tau_{II} = (\tau_Q + \tau_T) \cdot (1.0 + \cot^2 \theta)$

$\sigma_{II} = \frac{\tau_{II}}{\cot \theta + \cot \alpha}$

A rather nasty problem is the evaluation of the shear. The DIN design code allows a simple solution based on a corrected value for the inclination of the compressive struts:

$\tan \theta_{fat} = \sqrt{\tan \theta}$

Unfortunately it is nearly impossible to keep this value from the shear design for all individual shear cuts or transform it to different load com-

$\theta$: angle of compression struts

$\alpha$: angle of shear reinforcement

$\alpha = 90^\circ \Rightarrow \sin \alpha = 1.0, \cot \alpha = 0.0$

6.8.2(3): In the design of shear reinforcement the inclination of the compressive struts $\theta_{fat}$ may be calculated by Eq. 6.65

6.8.2(3): Eq. 6.65: $\tan \theta_{fat}$
Fatigue of a Rectangular Reinforced Concrete Cross-section and reinforcement distributions for the fatigue stress check. 

**AQB** uses instead a fixed value of $4/7$ for the tangents. The user may overwrite this value however with any desired value.

$$\tan \theta = 4/7 \Rightarrow \cot \theta = 7/4 = 1.75$$

$$\tan \theta_{fat} = \sqrt{4/7} = 0.756$$

$$\cot \theta_{fat} = \sqrt{7/4} = 1.3229$$

$$\tau_{II} = (427.770 \cdot 10^{-3} + 0.065 \cdot 10^{-3}) \cdot (1.0 + 1.75^2)$$

$$\tau_{II} = 1740.048 \cdot 10^{-3} \text{ MPa}$$

$$\sigma_{II} = \frac{1740.048 \cdot 10^{-3}}{1.75 + 0.0} = 994.313 \cdot 10^{-3} \text{ MPa}$$

$\sigma_s$: steel stresses

$$\sigma_{sl} = \frac{f_Q \cdot \tau_Q}{(\cot \theta_{fat} + \cot \alpha) \cdot \sin \alpha} + \frac{f_T \cdot \tau_T}{\cot \theta_{fat}}$$

$$A_{sl/cut} = A_{sl} / 2 = 4.11 \text{ cm}^2/m$$

$$f_T = \frac{B_0 \cdot f_r}{A_{sl/cut}} \quad \text{and} \quad f_Q = \frac{B_b \cdot f_r}{A_{sl/cut}}$$

where $f_T$ and $f_Q$ are factors expressing the shear links reinforcement ratios. They are depending on $f_r$, a factor for total reinforcement, $B_0$, the width of the cut and $B_b$, the total width of the cut. Since in this case it is a box cross-section and taking into account the position of the cut, we get that $B_0 = B_b = 0.4$ m.

The factor $f_r$ has only two possible values $f_r = 1.0$ or $f_r = 2.0$. It depends on the cross-section and the shear cut. If $B_{\text{max}} < B_0$ then $f_r = 2.0$.

$$f_T = \frac{0.4 \cdot 1.0}{4.11 \cdot 10^{-4}} = 973.532 \quad \text{and} \quad f_Q = \frac{0.4 \cdot 1.0}{4.11 \cdot 10^{-4}} = 973.532$$

$$\sigma_{sl} = \frac{973.532 \cdot 427.770 \cdot 10^{-3}}{1.3229 + 0.0} \cdot 1.0 + \frac{973.532 \cdot 0.065 \cdot 10^{-3}}{1.3229}$$

$$\sigma_{sl} = 314.882 \text{ MPa}$$

*Figure 4: Cross-section Overview*
Figure 5: Factor of total reinforcement, $f_r = 1.0$ (left), $f_r = 2.0$ (right)

Accordingly, we calculate the stresses for the rest of the loadcases. For each loadcase the stresses are calculated for two cases, for $\tau_T$ and for $-\tau_T$, in order to determine the most unfavorable case. The results are presented in Table 4

<table>
<thead>
<tr>
<th>LC</th>
<th>$Q_z$</th>
<th>$\tau_T \cdot 10^{-3}$</th>
<th>$\tau$</th>
<th>$\tau_{II}$</th>
<th>$\sigma_{II}$</th>
<th>$\sigma_{sl}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[kN]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
</tr>
<tr>
<td>911</td>
<td>523.14</td>
<td>0.428</td>
<td>0.065</td>
<td>0.427</td>
<td>1.740</td>
<td>0.993</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-0.065</td>
<td>0.427</td>
<td>1.739</td>
<td>0.993</td>
</tr>
<tr>
<td>912</td>
<td>566.03</td>
<td>0.463</td>
<td>17.31</td>
<td>0.480</td>
<td>1.952</td>
<td>1.115</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-17.31</td>
<td>0.4455</td>
<td>1.8120</td>
<td>1.0346</td>
</tr>
<tr>
<td>913</td>
<td>608.911</td>
<td>0.498</td>
<td>-34.41</td>
<td>0.4635</td>
<td>1.8851</td>
<td>1.0764</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>34.41</td>
<td>0.5323</td>
<td>2.1649</td>
<td>1.2362</td>
</tr>
</tbody>
</table>

From the table, the minimum and maximum value of the steel stress is determined:

- Max. $\sigma_{sl} = 391.77$ MPa
- Min. $\sigma_{sl} = 314.79$ MPa

As the exact fatigue stress check is not available, the simplified methods according to DIN EN 1992-1-1/NA (Sect. 6.8, Fatigue) are selected via the coefficients $\lambda_z$, $\lambda_T$, $\lambda_{II}$, $\lambda_{sl}$.

The admissible sways of the damage equivalent stress range for the shear links are obtained, as follows:

$$\Delta \sigma_{sl,eq}(N^*) = \lambda_{II} \cdot (\sigma_{sl,max} - \sigma_{sl,min}) = 1.0 \cdot (391.77 - 314.79)$$

$$\Delta \sigma_{sl,eq}(N^*) = 76.98 \text{ MPa}$$

For reinforcing steel adequate fatigue resistance should be assumed, if the following is satisfied:

$$\gamma_{f,\text{fat}} \cdot \Delta \sigma_{sl,eq}(N^*) \leq \frac{\Delta \sigma_{\text{RISK}}(N^*)}{\gamma_{s,\text{fat}}}$$

$$1.0 \cdot 76.98 \leq \frac{175}{1.15}$$

$\lambda_z$: Coeff. equiv. stress range shear links, here input as 1.0

6.8.2 (2): Eq. 6.64: $\eta$ factor for effect of different bond behaviour

$$\eta = \frac{A_s + A_p}{A_s + A_p + \frac{1}{\gamma_{s,\text{fat}}}}$$

(no prestress) $\Rightarrow \eta = 1.0$, thus no increase of calculated stress range in the reinforcing steel

6.8.5 (3): Eq. 6.71: Verification using damage equivalent stress range

(NDP) 6.8.4 (6): Table 6.3DE: Parameters for fatigue strength curves for reinforcing steel $\Delta \sigma_{\text{RISK}}(N^*) = 175$ for straight/bent bars and $N^* = 10^6$ cycles

(NDP) 2.4.2.3 (1): Partial factor for fatigue loads $\gamma_{f,\text{fat}} = 1.0$

(NDP) 2.4.2.4 (1): Partial factors for materials $\gamma_{s,\text{fat}} = 1.15$
\[ \Delta \sigma_{equ}(N^*) = 76.98 \leq 152.2 \text{ MPa} \]

If a coefficient \( \lambda = 2.0 \) is input for the shear links, resulting in a stress range of \( \Delta \sigma_{equ}(N^*) = 2 \cdot 76.98 = 153.97 \text{ MPa} \), a star (*) will be printed in the output next to the shear link stress range, denoting that the limit value of 152.2 MPa has been exceeded.

The design fatigue strength of concrete is determined by:

\[
f_{cd,\text{fat}} = k_1 \cdot \beta_{cc}(t_0) \cdot f_{cd} \cdot \left( 1 - \frac{f_{ck}}{250} \right)
\]

\[
f_{cd,\text{fat}} = 1.0 \cdot 1.0 \cdot 19.83 \cdot \left( 1 - \frac{35}{250} \right) = 17.0567 \text{ MPa}
\]

In the case of the compression struts of members subjected to shear, the concrete strength \( f_{cd,\text{red}} \) should be reduced by the strength reduction factor \( \nu_1 \) according to 6.2.3(3).

\[
\nu_2 = 1 \text{ for } \leq C50/60
\]

\[
\nu_1 = 0.75 \cdot \nu_2 = 0.75
\]

\[ \Rightarrow f_{cd,\text{fat,red}} = 0.75 \cdot 17.0567 = 12.7925 \text{ MPa} \]

A satisfactory fatigue resistance may be assumed, if the following condition is fulfilled:

\[
E_{cd,\text{max,equ}} + 0.43 \cdot \sqrt{1 - R_{\text{equ}}} \leq 1
\]

\[
\frac{\sigma_{cd,\text{max,equ}}}{f_{cd,\text{fat,red}}} + 0.43 \cdot \sqrt{1 - \frac{\sigma_{cd,\text{min,equ}}}{\sigma_{cd,\text{max,equ}}}} \leq 1
\]

\[
\sigma_{cd,\text{max,equ}} \leq f_{cd,\text{fat,red}} \cdot \left( 1.0 - 0.43 \cdot \sqrt{1 - \frac{\sigma_{cd,\text{min,equ}}}{\sigma_{cd,\text{max,equ}}}} \right)
\]

\[
\sigma_{cd,\text{max,equ}} \leq 12.7925 \cdot \left( 1.0 - 0.43 \cdot \sqrt{1 - \frac{0.9933}{1.2362}} \right)
\]

\[
\sigma_{cd,\text{max,equ}} = 1.2362 \leq 10.35 \text{ MPa}
\]

Accordingly the above verification is done for the minimum and maximum nonlinear stresses of concrete, as calculated from 

\[ \sigma_{cd,\text{max,equ}} \leq f_{cd,\text{fat}} \cdot \left( 1.0 - 0.43 \cdot \sqrt{1 - \frac{\sigma_{cd,\text{min,equ}}}{\sigma_{cd,\text{max,equ}}}} \right) \]

\[
\sigma_{cd,\text{max,equ}} \leq 17.0567 \cdot \left( 1.0 - 0.43 \cdot \sqrt{1 - \frac{5.69}{6.60}} \right)
\]

\[
\sigma_{cd,\text{max,equ}} = 6.60 \leq 14.33 \text{ MPa}
\]

**Stress limitation**

\[ \sigma_{\text{max,t}} = k_3 \cdot f_{yk} \]
Figure 6: Min/Max. Nonlinear Stresses of Concrete at "TOP" Point (BEM)

\[ \sigma_{max,t} = 0.80 \cdot 500 \text{ MPa} \]
\[ \sigma_{max,t} = 400 \text{ MPa} \]
5 Conclusion

This example shows the verification of a reinforced concrete beam to fatigue. It has been shown that AQB follows the fatigue verification procedure, as proposed in DIN EN 1992-1-1/NA [1] [2]. The insignificant deviation arises from the fact that the benchmark (reference) results have been calculated by using the BEM analysis. By introducing the FEM analysis, AQUA calculates now the $1/W_{Vz}$, $1/W_{Vy}$ and $1/W_T$ values more accurate.

6 Literature
