Situacion europea en forjados de edificacion

STEEL FIBRE AS ONLY REINFORCING IN FREE SUSPENDED ONeway
AND TWO WAYS ELEVATED SLABS : DESIGN CONCLUSIONS BASED UPON FULL
SCALE TESTING RESULTS

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Introduction

Today, 75 buildings have been completed with SFRC suspended elevated flat slabs: References are in Belgium, the UK, ScanBaltics, Austria, Denmark and Spain. Herebelow photos of the Rocca Tower completed in 2008 in Tallinn, Estonia. 17 elevated floors have been completed using Steel Fibre only Reinforcing together with minimum continuity bottom rebars from columns to columns. The slabs had a span to depth ratio of 27.

Rocca Tower

SFRC with typical continuity rebars in the slab bottom
Full scale test in Holland and design following the FIB ModelCode 2010


Structure: 3 consecutive spans of a one way slab: 3x 5,40m x 180mm thickness
Tunnel form released at 16 hours after the installation.
Slabs and walls poured together and thus monolithical
3 separated sections completed so that 9 spans have been tested.
Mix design and requirements

In order to release the forms at 16 hours age, $f'_{c16h} > 14\text{N/mm}^2$

Mix design:
350kg/m³ CEMI/CEMIII with W/C<0,50, F4 fluidity
50kg/m³ dosage rate of HE90/60 (0,9mm diametre and 60mm length)
$f_y = 1100\text{N/mm}^2$. (wire tensile strength)

Laboratory testing results

The testing of the hardened concrete showed the following results at 28 days:

- $f'_c = 65\text{N/mm}^2$
- $f_{r1} = 6,1\text{N/mm}^2$ (EN14651)
- $f_{r4} = 5,3\text{N/mm}^2$ (EN14651)
- $f_{LOP} = 6,5\text{N/mm}^2$
- $f_{Max} = 8,0\text{N/mm}^2$

$\gamma_c = 2392\text{kg/m}^3$
Test set-up

3 sets of 3 continuous adjacent spans of 180mm thickness (A,B,C); (D,E,F) and (G,H,I) have been cast. The last set of spans are the 2400mm cantilever balconies (J,K,L) of 250mm thickness.

Overview of the locations of deflection apparatus (LVDT) and crack opening (CMOD) apparatus.
Full scale tests

The test load was applied by incremental steps and out of 16 blocks of concrete deposited on lumbers with 500mm spacing. The inner span has been tested first for all three frames and consecutively the outer spans (G,H,I) spans have been tested after one year.
## Overview of the collapse load and the deflection

The deflections recorded here are the last stable recordings before the collapse

<table>
<thead>
<tr>
<th>Identification of test span and date of test As in Fig.3</th>
<th>Maximum UDL intensity (kN/m²) (excluding self weight)</th>
<th>Deflection before collapse (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal span B: june 18, 2010</td>
<td>11,45</td>
<td>27</td>
</tr>
<tr>
<td>Internal span E: sept 23, 2010</td>
<td>14,20</td>
<td>33</td>
</tr>
<tr>
<td>Edge span A: sept 16, 2010</td>
<td>11,87</td>
<td>13</td>
</tr>
<tr>
<td>Edge span C: sept 14, 2010</td>
<td>11,35</td>
<td>14</td>
</tr>
<tr>
<td>Edge span F: sept 28, 2010</td>
<td>12,02</td>
<td>18</td>
</tr>
</tbody>
</table>
Summary of Observations during the testings:
Multiple cracking and moment redistribution

**Experimental observations:**

The initial cracking above the support occurred at 7-8kN/m² excluding self weight of slab.
The initial positive moment cracking occurred at 10-11kN/m² excluding self weight of slab.

**Statistical Verification:**

- Initially, the floor is a continuous beam on 4 supports and fixed in the walls.

**Cracking of negative moments occurs first at -25,3kNm/m or 4,68MPa flexion stress.**
- Then, the negative moment increases to 30,5kNm/m or 5,65MPa flexion stress followed by a decrease down to -25,3kNm/m.
- Simultaneously, the positive field moment increase from 12,5kNm/m to 30,5kNm/m

**Cracking load: 8 x (25,3 + 12,7)/5,32² = 10,8 kN/m² including the self weight of the slab**
- Net cracking load = 10,8kN/m² - self weight of slab = 6,5kN/m² responsible of the first negative moment cracking.
- Then the positive moment increases up to a loading intensity of 8 x (28 + 25,3)/5,32² = 15,2kN/m²
Subtracting the self weight of the slab, we obtain **10,9kN/m²**

A number of cracks were then visible in the bottom of the slab prior to the final collapse of the structure.
Positive moment cracking in the bottom prior to the collapse.
- Multiple cracking observed in a one-way slab.
- High residual capacity of the SFRC
Summary of the observations at the ultimate stage

- Maximum applied load onto the slab: 12,18kN/m² + 4,32kN/m² selfweight.

- Deviation of the maximum load: 1,16kN/m² or 9,5% thus much smaller than the 25 to 40% shown by standard prismatic specimen in flexion.

- Maximum average deflection: 20,83mm

- First crack average loading intensity: 9,25kN/m²
EN 14651 test results at 30 days and 129 days age

30 days:
\[
\begin{align*}
    f_{Lm} &= 6,01 \text{N/mm}^2, s = 0,52 \\
    f_{R1m} &= 5,60 \text{N/mm}^2, s = 2,10 \\
    f_{R3m} &= 5,00 \text{N/mm}^2, s = 1,96
\end{align*}
\]
\[
\begin{align*}
    f_{Lk} &= 5,08 \text{N/mm}^2 (1,77 \times 0,52 + 5,08 = 6,01) \\
    f_{R1k} &= 1,88 \text{N/mm}^2 (1,77 \times 2,1 + 1,88 = 5,6) \\
    f_{R3k} &= 1,53 \text{N/mm}^2 (1,77 \times 1,96 + 1,53 = 5,0)
\end{align*}
\]

129 days:
\[
\begin{align*}
    f_{Lm} &= 6,53 \text{N/mm}^2, s = 0,39 \\
    f_{R1m} &= 7,65 \text{N/mm}^2, s = 0,89 \\
    f_{R3m} &= 6,25 \text{N/mm}^2, s = 1,41
\end{align*}
\]
\[
\begin{align*}
    f_{Lk} &= 5,84 \text{N/mm}^2 (1,77 \times 0,39 + 5,84 = 6,53) \\
    f_{R1k} &= 6,07 \text{N/mm}^2 (1,77 \times 0,89 + 6,07 = 7,65) \\
    f_{R3k} &= 3,75 \text{N/mm}^2 (1,77 \times 1,41 + 3,75 = 6,25)
\end{align*}
\]

The 30 days characteristic strength are low and adversely affected by a very high deviation.
The notched beam EN14651 method is typical of an erratic and unpredictable deviation: due to casting, compacting and the notch itself
Calculations following FIB-MODELCODE 2010

FIRST REQUIREMENT

Minimum performance for structural reinforcement:

(5.6-2): \( \frac{f_{R1k}}{f_{Lk}} > 0,4 \) and (5.6-3): \( \frac{f_{R3k}}{f_{R1k}} > 0,5 \)

at 30 days: 0,37 < 0,4  0,81 > 0,5
It denotes a 5b class at 30 days.

Again here we see the 30 days test results that do not pass the condition unlike other tests and the ductility of the real scale slab tested as it has been demonstrated by the full scale tests that the SFRC material is fit for the application

at 129 days: 1,04 > 0,4  0,62 > 0,5
It denotes a 6a class (5.6.3) at 129 days
Calculations following FIB-MODELCODE 2010

SECOND REQUIREMENT

Ductility of the SFRC Structure.

\[ \delta_u > 20 \delta_{SLS} \quad (7.7-1) \quad \text{or} \quad \delta_{\text{PEAK}} > 5 \delta_{SLS} \quad (7.7-2) \]

(7.7-2) is verified as follows:

\[ \delta_{SLS} = \frac{q L^4}{384} = 0.89\text{mm} \quad \text{(slab with clamped ends)} \]

where \( q = 2.25\text{kN/m}^2 + 4.32\text{kN/m}^2 \) (max. live load + selfweight)

\[ q = 6.57\text{kN/m}^2 \]

Thus \( 20.83\text{mm} > 5 \times 0.89\text{mm} = 4.45\text{mm} \)

\( q = 2.25\text{kN/m}^2 \) should and could be increased to more than 3kN/m\(^2\) as a maximum allowable service loading intensity.

Thus \( 20.83\text{mm} > 5 \times 8.32/6.57 \times 0.89\text{mm} = 5.64\text{mm} \) (4kN/m\(^2\)
**Calculations following FIB-MODELCODE 2010**

Effects of redistribution in the structure: $K_{Rd}$ factor (7.7.2)

$P_{maxm} = 143\text{kN}$ with $s = 23\text{kN}$, thus $P_{maxk} = P_{maxm} - 1.96s = 97.92\text{kN}$

When, $f_m = 6.25\text{N/mm}^2$ and $f_{R3k} = 3.75\text{N/mm}^2$,

Hence,

$$K_{Rd} = \left(\frac{P_{maxk}}{P_{maxm}}\right) \times \left(\frac{f_m}{f_k}\right) = 1.141$$

$K_{Rd}$ is unpractical in reality: the design engineer does not know the real collapse load of the structure. The designed structure is never tested to rupture in reality!

$K_{Rd}$ is impossible to calculate in a real case of design
Calculations following FIB-MODELCODE 2010

The test slabs are calculated according to the yield line method:

\[
q_{pl} = \text{plastic load}
\]
\[
m_{pl} = \text{plastic moment} = W \cdot f_{eq}
\]
\[
q_{pl} = 16 \cdot W \cdot f_{eq}/l^2
\]
Calculations following FIB-MODELCODE 2010

The stress distribution model across the section is assumed to be of the linear elastic type as shown in **5.6.4 Constitutive Laws:**

\[ M_u = (0.5 f_{R3k} - 0.2 f_{R1k}) h^2/2 + (f_{FTs} - 0.5 f_{R3k} + 0.2 f_{R1k}) h^2/6 \]

\[ = (f_{R3k} + 0.05 f_{R1k}) h^2/6 = 4.05 \times 180^2/6 = 21.87 \text{ kNm/m} \]

Where \( f_{FTs} = 0.45 f_{R1k} = 2.73 \text{N/mm}^2 \)

\( f_{R3k} = 3.75 \text{ N/mm}^2 \)

\( f_{R1k} = 6.07 \text{N/mm}^2 \)

With a net span of \( L = 5.40 \text{m} \), \( w \) the slab own weight (180mm thickness), the ultimate loading is:

\[ q_u = 16 \frac{M_u}{L^2} = 12.00 \text{ kN/m} \]

\[ q_{test} = q_u \times K_{Rd} - w = (12.00 \times 1.14 - 4.32) \text{ kN/m}^2 = 9.36 \text{ kN/m}^2 \]

To be compared to the real test load (own weight excluded) - of 12 \text{kN/m}^2 or 22% more than predicted
Calculations following FIB-MODELCODE 2010

Comments: the unrealistic influence of the deviation of EN14651 tests on the design results.

In the example of this presentation, the standard deviation is around 10% for $f_{r1}$ which is at the low side. Should we have encountered a standard deviation of 40% in the beam test, then the predicted ultimate test value would have been only 2.5 kN/m² which corresponds only to 18% of the real test value!. While in reality the standard deviation on the rupture load on the structure is only 10%.

- The Model Code draft 2010 does not predict quite well and still underestimates somehow the behaviour of the structure in bending, and this only as long as the standard deviation in the characterisation tests is no more than 15%!
CONCLUSIONS

1)
- All five spans tested were ductile resulting from a significant moment redistribution and a multiple cracking pattern under both sagging and hogging moments.

- The deviations in reality is significantly smaller than the deviations in the characterisation test EN14651.
- EN14651 test method shows a deviation that is typical to the test method only.

- The first crack experimental loading intensity in all 5 spans occurred at more than 250 % of the standard housing service loading intensity in The Netherlands (2,25kN/m²).
- The ultimate loading experimental intensity in all 5 spans occurred at more than 450 % of the standard housing service loading intensity in The Netherlands (2,25kN/m²).

- The Model Code draft 2010 predicts well the ultimate behaviour of the structure in bending, as long as the standard deviation in the characterisation tests are in the range of 15%. Thus regardless of the deviation observed following EN 14651, use always 15% deviation like observed in the slab tests.
2) The FIB MC 2010 takes into account a material ductility but does not take enough into account the structural ductility due to the degree of indeterminacy of the structure.

As a result the possible service (unfactored) loading is limited to 2,25kN/m² to be compared 7,5kN/m² first negative moment crack loading intensity

We recommend to introduce a factor \( \eta \) like in the future SIS swedish standard where:
\( \eta = 1 \) for statically determinated beam (no structural ductility)
\( \eta = 1,4 \) in case of a structure showing multiple parallel yield lines
  (case of a one way slab of multiple spans or two ways under UDL)
\( \eta = 2,0 \) in case of a structure showing yield lines in x and y directions in top and bottom.
  (case of a two-ways slab of multiple spans subjected to point loadings, internal span)

\( \eta \) should then replace the \( K_{rd} \) factor

1,4 is also the maximum possible value of \( K_{rd} \) in the FIB Modelcode 2010
In the example we’ll now limit and use \( n =1,4 \) and 2,0
CONCLUSIONS

3) At 2.25 kN/m² x 1.4 = 3.15 kN/m², the flexion stress is = (3.15 + 4.32) x 5.4² x 6000/12 x 180² = 3.36 N/mm² thus at 60% of first crack.

4) **Practical guidelines for designing (safely):**
- The SFRC structure must remain uncracked in flexion under the most onerous service (unfactored) loading combination.
- Use no less than C30-37 and W/C < 0.50; careful curing of the slab.
- 0-16-20mm continuous agg. Grading with 500 kg total fines smaller than 200 microns F5 fluidity installed without poker vibrating; Min 350 kg CEMI/m³
  Minimum volumic weight of concrete: 23.80 kN/m³
- \( f_{c1} \) and \( f_{c3} > 5 \text{N/mm}² \)
- Span/depth < 25 to 30
- Use a minimum bottom continuity reinforcement \( A_{sb} \) all across the slab as follows:
  \[ A_{sb} = \frac{L \times (q + w)}{0.85 f_y} \]
  In the example here, it is: \( 5150 \times (3.50 + 4.32)/(370) = 108 \text{ mm}²/m \)
  thus 1 dia 16mm continuous in the bottom every 2.00m apart in the direction of the span.(4 kg/m³ rebars)
- 5) Fibre type: use 1 mm dia x 60 mm length (HE or undulated) with high strength steelwire of >1450 N/mm²
  - Thinner fibres tend to reduce the workability and show at the surface; don’t use them
Example of Calculations

One way slab calculation becomes:

\[(1,35w + 1,5p) L^2 < 16 \eta M_d \]
where \(w = 4,32\text{kN/m}^2; p = 3,50\text{kN/m}^2\) and \(\eta = 1,4\)

\[(6,48 + 4,73) 5,40^2 < 16*1,4*21,87 \text{ verified OK! (p = max SLS live loading intensity)}\]

In case of an edge span use 12 instead of 16

\[(1,35w + 1,5p) L^2 < 12 \eta M_d \]

\[
\begin{align*}
342 &< 367 \\
\end{align*}

Thus \(p \text{ (SLS)} < 12*1,4*21,88/(5.4^2*1.5) - 1,35*w/1,5 = 4,5\text{kN/m}^2 \text{ (Edge span = critical)}\)

Safety against first moment crack \(7,5/4,5 = 1,67\)

Safety against collapse \(12/3.5 = 3,43\)

\(P\) includes the live load, the partition walls loading, overlays and tiles

Two way slab (supported by columns) the same equation and result as the it has the same shortest pattern of yield lines in case of a uniformly distributed loading.

Minimum continuity bottom reinforcing rebars also needed and going from column to column;

\[
A_{sb} = L_x \times L_y \times (q + w) / 10^3 \quad f_y = 5400^2 \times 7,47 / 10^3 \times 435 = 501\text{mm}^2, \text{ thus}\]

Install 3 diam 16mm in the bottom in X and Y directions above each column footprint.
Tallinn full scale test (2007)

UDL = 5kN/m²
2mm deflection

Centre Point loading of 600kN
First crack at 120kN

600kN: 3mm crack opening. Bottom cracks only.

Cracking pattern: 0.3mm/250kN
Example of Calculations: Tallinn Test

3 x 5 m x 5m in 180mm thickness

**Two way slab** (supported by columns): case of point loading P.

\[2 \times 1.5 \times P + 1.35wL^2 < 16nM_d\]  \[\text{internal span}\]  \[P < \frac{(16nM_d - 1.35wL^2)}{3}\]

(P centre point loading)  \[n=2\]  \[P < 186\,\text{kN}\]

\[2 \times 1.5 \times P + 1.35wL^2 < 12nM_d\]  \[\text{edge span}\]  \[P < \frac{(12nM_d - 1.35wL^2)}{3}\]

(5400mm x 5400mm x 180mm flat slab)  \[\text{loading-edge}\]  \[n=2\]  \[P < 127\,\text{kN}\]

For the **corner span**, we use \[n=1.4\] as the structural ductility is more limited

(P centre point loading-corner)  \[P < 75\,\text{kN}\]

**Minimum continuity bottom reinforcing rebars** also needed and going from column to column;

\[A_{sb} = \frac{L_x \times L_y \times (q + w)}{10^3} \times 0.85f_y = \frac{5000^2 \times 7.47}{10^3} \times 435 = 503\,\text{mm}^2\]

Install 3 diam 16mm in the bottom in X and Y directions above each column footprint.

These results of the two ways slab are comparable to the Tallinn full scale test (2007)
Example of Calculations: Shear/Punching-out of Tallinn Test

3 x 5 m x 5m in 180mm thickness (flat plate)
The FIB MC 2010 is silent about shear or punching-out of slab saying it not critical.
Around the columns support, we’ll use a swedish SIS standard draft official in 2014.

\[
f_{cfl} := 6.01 \frac{N}{mm^2} \quad f_{cflR1} := 6.07 \frac{N}{mm^2}
\]

\[
C_1 := \frac{f_{cflR1}}{f_{cfl}} = 1.01
\]

\[
f_{cflR3} := 3.75 \frac{N}{mm^2}
\]

\[
C_3 := \frac{f_{cflR3}}{f_{cfl}} = 0.624
\]

\[
\gamma_{ft} := 1.50
\]
Example of Calculations: Shear/Punching-out of Tallinn Test

3 x 5 m x 5m in 180mm thickness (flat plate)

The FIB MC 2010 is silent about shear or punching-out of slab saying it not critical. Around the columns support, we’ll use a Swedish SIS standard draft official in 2014.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{ctR1}$</td>
<td>$0.45f_{cflR1}$</td>
<td>$0.45$ N/mm$^2$</td>
</tr>
<tr>
<td>$f_{ctR3}$</td>
<td>$0.37f_{cflR3}$</td>
<td>$0.37$ N/mm$^2$</td>
</tr>
<tr>
<td>$\eta_{det}$</td>
<td>$2.0$</td>
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</tr>
<tr>
<td>$\eta_f$</td>
<td>$1$</td>
<td></td>
</tr>
<tr>
<td>$C$</td>
<td>$0.45$</td>
<td>a general coefficient</td>
</tr>
<tr>
<td>$d$</td>
<td>$0.18m$</td>
<td>a thickness factor</td>
</tr>
<tr>
<td>$R$</td>
<td>$180mm$</td>
<td>a reference length</td>
</tr>
<tr>
<td>$k$</td>
<td>$1 + \sqrt{\frac{R}{d}} = 2$</td>
<td></td>
</tr>
<tr>
<td>$V_{Rdf}$</td>
<td>$\frac{k}{2} \cdot C \cdot f_{cflR3} = 1.688$ N/mm$^2$</td>
<td>For 1 m run of slab</td>
</tr>
<tr>
<td>$b$</td>
<td>$1m$</td>
<td></td>
</tr>
<tr>
<td>$T_d$</td>
<td>$V_{Rdf} \cdot b \cdot d = 3.038 \times 10^5$ N</td>
<td></td>
</tr>
</tbody>
</table>
Example of Calculations: Shear/Punching-out of Tallinn Test

\[ C := 0.45 \]
\[ d := 0.18 \text{m} \]
\[ R := 180 \text{mm} \]
\[ k := 1 + \sqrt{\frac{R}{d}} = 2 \]
\[ V_{Rdf} := \frac{k}{2} \cdot C \cdot f_{cflR3} = 1.688 \frac{N}{\text{mm}^2} \]
\[ b := 1 \text{m} \]
\[ t_d := V_{Rdf} \cdot d = 3.038 \times 10^5 \frac{N}{\text{m}} \]
\[ w := 4.32 \frac{kN}{\text{m}^2} \]
\[ p := 3.5 \frac{kN}{\text{m}^2} \]
\[ a := 0.20 \text{m} \]
\[ b := 0.20 \text{m} \]
\[ \Sigma := (a + b + 2 \cdot d) \cdot 2 = 1.52 \text{m} \]
\[ T_d := \Sigma \cdot t_d = 461.7 \text{kN} \]
\[ T_{SLS} := \frac{T_d}{1.5} = 307.8 \text{kN} \]

\[ \frac{T_{SLS}}{(w + p) \cdot L_x \cdot L_y} = 1.574 \quad \text{OK!} \]
Tallinn Full scale test: the corner span

Corner span: 300kN Ultimate
70mm deflection
Tallinn Full scale test: the central span

Tallinn test 08/2007; center point loading

Central span: 600kN Ultimate
More than 70mm deflection