Steel fibre only reinforced concrete in free suspended elevated slabs: Case studies, design assisted by testing route, comparison to the latest SFRC standard documents

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ABSTRACT: The total replacement of traditional rebars is now completely routine for 15 year in applications such as industrial and commercial suspended slabs resting on pile grids, which can span from 3 m to 5 m each way, with span to depth ratios from 15 to 20. Seven millions of square meter have been completed so far. More recently, the structural use of steel fibre-only reinforcement at high dosage rate has been developed as the sole method of reinforcement for fully elevated suspended slabs spanning from 5 m to 8 m each way, with a span to depth ratio of 30. More than forty buildings are now completed. The SFR concrete mix is also fully pumpable and doesn’t need any poker vibrating. Significant time and cost savings are then achieved. Design methods are derived from round slabs flexion testing and from full scale testing results of real elevated suspended slabs. Three full scale testing slabs have been built in order to monitor deflections, punching and cracking when they are loaded up to final rupture. This article summarizes the design methods and compares them to the provisions of the most recently available steel fibre reinforced concrete standards.

1 INTRODUCTION
This article explains how and why, standard design material characteristics of SFRC derived from small standard prismatic beam specimen should not be relevant as such in the design procedure of SFRC elevated suspended slabs. If only test results from prismatic specimen are available then it is strongly recommended to use an application-factor on the derived material characteristics as described in this article.

2 FULL SCALE TESTS
2.1 Full scale test procedures
Four full scale testing procedures of SFRC elevated free suspended slabs (at Ternat, Townsville, Bissen and Tallinn) have been organised in order to investigate the real structural behaviour of the structural SFRC in slab applications. The different test slab patterns are described in table 1. In the case of the 160 mm thick slab (Ternat and Townsville cases) with 45 kg/m³ TABIX/Twincone Steel fibers the slab rested on a pile grid of 3100 mm × 3100 mm which is typical for a pile suspended industrial ground slab. Each column had a section of 210 mm × 210 mm.

In the case of the 200 mm and 180 mm thick slab with 100 kg/m³ TABIX 1,3/50 steel fibers the slab rested on a column grid of 6 m × 6 m, respectively 5 m × 5 m, a typical grid spacing for elevated, free suspended flat plates in residential or commercial buildings.

Firstly the slabs were loaded with water barrels up to the service loads. In this state no cracks due to external loads could be observed. In the uls-tests the slabs were loaded under a test rig with a single load in the middle of several fields (see figure 1).

2.2 Results of Full scale tests procedures
For testslab 1 and 2 a comparison of the calculated allowable loadings and experimental loads is shown in table 2 for 2 different fields. Based on design recommendations of ArcelorMittal the maximum loading intensity (SLS) is shown in line A in comparison to experimental loadings. In line B the maximum loading intensities derived from typical prismatic beam-tests can be compared with the realistic test results.
Table 1. Overview of the realized full scale tests.

<table>
<thead>
<tr>
<th>Year and Location</th>
<th>N° fields x + y/ N° columns</th>
<th>span length</th>
<th>thickness</th>
<th>column size</th>
<th>dosage rate of steel fibers</th>
<th>span/depth-ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1: 1994 Ternat, Belgium</td>
<td>3/16</td>
<td>3100</td>
<td>160</td>
<td>210 × 210</td>
<td>45</td>
<td>19</td>
</tr>
<tr>
<td>2: 2000 Townsville, Australia</td>
<td>3/16</td>
<td>3100</td>
<td>160</td>
<td>210 × 210</td>
<td>45</td>
<td>19</td>
</tr>
<tr>
<td>3: 2004 Bissen, Luxembourg</td>
<td>3/16</td>
<td>6000</td>
<td>200</td>
<td>300 × 300</td>
<td>100</td>
<td>30</td>
</tr>
<tr>
<td>4: 2007 Tallinn, Estonia</td>
<td>3/16</td>
<td>5000</td>
<td>180</td>
<td>300 × 300</td>
<td>100</td>
<td>28</td>
</tr>
</tbody>
</table>

Figure 1. Full scale test in Tallinn with test rig and deflection gauges to measure the deformation of the slab under the loads in the uls.

Table 2. Test results of the full scale tests case 1 and 2 in Ternat and Townsville.

<table>
<thead>
<tr>
<th>Test</th>
<th>Max loads (SLS) AM design</th>
<th>Experim. loads at first crack</th>
<th>ULS loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Bissen kN/m²</td>
<td>Tallinn kN</td>
<td>Bis. kN</td>
</tr>
<tr>
<td>1: mid span</td>
<td>7/110</td>
<td>9/100</td>
<td>200</td>
</tr>
<tr>
<td>2: corner sp.</td>
<td>4/60</td>
<td>6/60</td>
<td>140</td>
</tr>
</tbody>
</table>

Max loads (SLS) derived from standard beam tests

2.3 Comparison of the results

In D), Case(2), Bissen test: these calculated loadings are ridiculously low when compared to reloading the corner span up to 260 mm(!) deflection when the residual loading intensity is still 150 kN(!) compared to 17 kN maximum loading intensity possible according to standard prismatic specimen based calculations.

In A), the first column to the left shows the maximum load intensity recommended in service condition when calculated using the yield line theory where yield moments are derived from round indeterminate slab testing, while the two last columns show experimental loading intensities during full scale tests.

In B), the same case when calculated according to small standards prismatic specimen testing results.

3 TESTS WITH CIRCULAR PLATES

3.1 Circular plate test procedures

A plastic design method according to the yield lines theory of Johanssen has been adopted based on the analysis of results of these four full scale tests together with statically indeterminate circular slabs (1.50 m...
and 2.00 m net span/150 mm and 200 mm depth) laboratory testing.

The centre deflection on circular plates of 1.50 m net span and 0.15 m thickness, subjected to centre point loading is monitored as shown in figure 2. The resisting yield moment, needed in the design method, is then derived based upon yield lines of Johanssen.

The plate is simply supported along its perimeter. During the flexion testing, a loading-deflexion diagram is recorded where the first cracks are visible (∼0.1 mm opening) under the point loading, followed by a pseudo-elastic multiple cracking linear stage ending up in the formation of yield lines of a fully plastic flexion stage according to the theory of Johanssen. The higher the dosage rate of steel fibres above \( V_f = 0.50\% \), the finer is the observed multiple cracking pattern as well as the larger number of yield lines and the higher ultimate loading intensity.

The undeterminate and structural nature of a such slab testing is what is needed to analyse SFRC suspeded slab applications as it enables to take into account the two way action, the multiple cracking and the plasticity of the section that increases the ultimate loading intensity vs. first crack by a factor between 3 and 6!.

Round determined slabs resting (ASTM C 1550) on a three point support do not show the same multiple cracking behaviour as three predetermined cracks are formed so that it is quite similar in results to the small prismatic beam specimen with only less deviation due to their larger size.

3.2 Results and evaluation of circular plate tests

The final rupture pattern is the typical “FAN” pattern where the expression of the equilibrium of rotation of one circular sector gives:

\[
P_{ult} = 2\pi \cdot M_R \tag{1}
\]

\( M_R \) = yield line Moment of Johanssen

The testing method seems to be very accurate with almost no dispersion of results, as different batches of the same mix design don’t deliver different flexion diagrams (see figure 3). It is indeed a structural type of testing to demonstrate formation of yield-lines in a slab.

4 DESIGN OF SFRC FLAT SLABS

This method of design of ArcelorMittal SFRC suspeded slabs is based on the analysis of the shortest pattern of yield-lines where almost all deformations are concentrated due to plastic rotation.

The “reinforcement percentage” or fibre dosage rate must be sufficient and high enough to ensure yielding of the section. The rupture mechanism to be considered is the least favourable for the proposed load and support, giving the minimum ultimate loading intensity \( Q_{ult} \).

4.1 Design based on ArcelorMittal-method

In case of udl a simple mechanism using the perimeter and median is used as this gives the shortest total length of yield lines.

\[
Q_{ult} = 16 \cdot M_R \quad \text{(center span, case 1)} \tag{2}
\]
In case of point loads, other types of mechanisms can be used, depending on the case.

The design conditions become:

In case 1 (central panel/restrained edge, fig. 04a):

$$\gamma_p \cdot P \cdot \frac{L_a}{8 \cdot b} + (\gamma_g \cdot G + \gamma_p \cdot Q) \cdot \frac{L_a}{16} \leq \frac{M_R}{\gamma_M}$$

(4)

In case 2 (edge panel, fig. 04b):

$$\gamma_p \cdot P \cdot \frac{L_a}{6 \cdot b} + (\gamma_g \cdot G + \gamma_p \cdot Q) \cdot \frac{L_a}{12} \leq \frac{M_R}{\gamma_M}$$

(5)

With $G =$ ownweight of the slab, $Q =$ udl and $P =$ pointloads. The safety factors on materials and loads are $\gamma_M = 1.5$, $\gamma_g = 1.35$ and $\gamma_p = 1.5$.

A typical realized example of these calculations is the full scale test slab of Ternat and Townsville (see table 1 and table 2). It shows how simple it is to use the equations above in cases 1 and 2.

$$G = 0.16m \cdot 24 \frac{KN}{m^2} = 3.84 \frac{KN}{m}$$

(6)

Case 1 (central span):

$$Q = 0 \frac{KN}{m}, \ P = 85kN; \ L_a = 3.10 - 0.21 - 0.16 = 2.73m$$

$$f_{ck} = 35 \frac{N}{mm^2} \ and \ f_{cru} = 22.2 \frac{N}{mm^2},$$

derived from round slab test (see above)

$$M_R = 0.45 \cdot f_{cru} \cdot h^2 = 25.58 \frac{KNm}{m}$$

(7)

So we need to verify:

$$1.5 \cdot 85 \frac{2.73}{8.31} + 1.35 \cdot 3.84 \cdot \frac{2.73^3}{16} \leq \frac{25.58}{1.5}$$

(4)

16.44 < 17.05 so the condition is verified

The calculation shows that a 85 kN point load imposed at mid span of a central span under service condition is the maximum acceptable load. The experimental full scale flat plate test showed a first crack at 110 kN intensity and the ultimate at 450 kN under the same conditions!

The global safety factor in reality is then of 450/85 = 5.30!

The 110 kN first cracking intensity is confirmed as well when using a Finite Element software calculation together with a 5 N/mm² flexural strength of the fibre reinforced concrete.

The same verification for case 2 (edge span):

$$Q = 0 \frac{KN}{m}, \ P = 58kN; \ L_a = 3.10 - 0.20 - 0.08 = 2.82m$$

$$1.5 \cdot 58 \frac{2.82}{6.31} + 1.35 \cdot 3.84 \cdot \frac{2.82^2}{12} \leq \frac{25.58}{1.5}$$

(5)

16.62 < 17.05 so the condition is verified

In this case the experimental full scale flat plate test showed a first crack at 80 kN intensity and the ultimate at 180 kN under the same conditions! The global safety factor in reality is then of 180/58 = 3.10 even if the slab was already predamaged due to the test in the mid span which was done before!

4.2 Design based on standard SFRC beam tests

The prismatic specimens in flexion, notched or unnotched, are unable to let the multiple cracking build up so that it is a single hinge mechanism, impossible to relate to a slab structure where SFRC multiple cracking and yielding develops.

Using an SFRC showing $Re,3 = 0.80$ with 5 N/mm² M.O.R in flexion and taking into account the prismatic specimen deviation (30%), then it results in

$$f_{cru,k} = 0.7 \cdot 0.8 \cdot \frac{5}{2.7} = 1.04 \frac{N}{mm^2}$$

(8)

and hence a $M_R = 11.98 \frac{KNm}{m}$

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In case(1) the maximum load condition comes up with 30 kN point load instead of 80 kN! The global safety factors of 5.3 and 3.1 are increased up to 14.80 and 8.68!

4.3 Verification of the two design methods

At 30 kN point loading, the flexural stress of sfrc is of $f_{ck} = 5 \cdot 30/110 = 1.36$ N/mm². To limit the use of SFRC to 1.3 N/mm² flexural stresses under service conditions in elevated flat slabs is a nonsense that is going to eliminate the use of SFRC from a
very successful and longer term experienced application very soon.

This is a basic explanation as to why a design procedure based upon SFRC standard prismatic specimen in flexion testing is not realistic and refrain the SFRC slab from being competitive in cost against the traditional design with meshes and rebars.

Not only prismatic specimens in flexion do not only supply a useful information for slab or flat plates design, but even more the range of curvature of the cracked specimens subjected to 3,00 mm deflexion or 3,5 mm crack opening, fail totally to develop in a real suspended slab where the curvatures are a lot smaller including at the very ultimate stage!

To depict clearly the differences between the testing methods, small beam specimens-, round undetermined slabs-, full scale slab test, it is informative to compare typical values of the curvature at different stages of deflection see table 4 $\kappa(10^{-5} \text{ l/mm})$

A 500 mm prismatic specimen at 3.5 mm deflection.
B 1500 mm round slab at 10 mm deflection. (span/150)
C 500 mm prismatic specimen at 1 mm deflection
D 1500 mm round slab at 5 mm deflection, corresponding to $P_{\text{ult}}$ of which the yield moment of Johanssen is derived. $P_{\text{ult}}$ is generally 200 to 350% of $P_{1st\text{crack}}$. $P_{1st\text{crack}}/P_{\text{ult}}$ ranges are respectively 40 – 60 kN/120 – 190 kN depending on concrete strength and type or dosage rate of steel fibres
E full scale slab at ultimate load (Bissen test: 470 kN/ Tallinn test: 595 kN)
F 1500 mm round slab at first crack or 1 mm deflection and both the Ternat and Townsville full scale slabs at 110 kN point load
G full scale slab test at maximum UDL service loads.
(What the customer pays for)
H full scale slabs of Bissen (3 adjacent 6 m x 6 m bays/ 200 mm) and Tallinn (3 adjacent 5 m x 5 m bays/180 mm) at first crack (Bissen test: 160 kN/Tallinn test: 120 kN).

It's quite relevant to note that E and H curvatures are respectively 10 and 100 times smaller than in A. Moreover the high scatter of the prismatic tests results are not taken into account in the above numbers so that we should increase even more the curvature at any given loading intensity by about 30%.

4.4 Conclusion

It points out the complete lack of practicality of the small beam specimen in flexion analyzing SFRC slabs at a stage deformation and curvature that are completely irrelevant to free suspended slabs.

As the prismatic beam test is defined as the standard test method to derive the material behavior of sfrc, the results of these tests must be multiplied by an application factor of up to 3 and even higher to design steel fibre only reinforced fully elevated flat slabs. Alternatively large scale round slab specimen can be used to seriously derive the material properties.

Under these conditions steel fibre reinforced elevated flat slabs are an economic and competitive solution with a high level of safety. More than 40 realized projects show the time and cost efficiency of this construction method.

5 SOME REALIZED PROJECTS

5.1 Shopping mall “Ditton Nams”

Daugavpils/Latvia

For the expansion of an existing shopping mall in the east of Latvia several flat slabs were realized with steel fibre reinforced concrete as the sole method of reinforcement (see figure 5 and 6).

5.2 Triangle office building Tallinn/Estonia

The building had a triangle shape with no rectangle inside. To replace time and const intensive reinforcement it was decided to use sfrc only (see figures 7 and 8).

5.3 Juhkentali apartment building Tallinn/Estonia

The contractor of the triangle office building decided to use the same technology for his next projects after the triangle office building was realized successfully. One of it was the Juhkentali apartment house near the city center of Tallinn (see figures 9–11).
Many more projects in the Baltic countries, Germany, Austria, BeNeLux, Spain and the UK have been realized and show, that steel fibre reinforced only flat slabs can be used and provide an economic and safe solution for modern elevated flat slabs. For more

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