STEEL FIBRE AS ONLY REINFORCING IN FREE SUSPENDED ONE WAY ELEVATED SLABS: DESIGN CONCLUSIONS OF A TUNNEL FORMED SLAB AND WALLS BASED UPON FULL SCALE TESTING RESULTS

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Summary: Full scale tests on five one-way slabs with steel fibre concrete have been tested at the Technical University of Eindhoven. The slabs were produced using a tunnel forming technique, which is a well known technique for residential applications. The walls and slabs have been executed with only steel fibre concrete. All five spans tested showed a quite ductile behaviour resulting from a significant moment redistribution of moments together with a multiple cracking pattern under sagging moments. The deviations on the load bearing capacity between the five spans tested are significantly smaller than the deviations in the characterisation test EN14651. EN14651 test method shows a deviation that is typical to the test method only and not to the real behaviour of the structure as tested. 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 behaviour of the structure in bending, as long as the standard deviations in the characterisation tests are in the range 10-20%.

1 INTRODUCTION

In September 2008, a technical delegation of the Dutch "Cement&BetonCentrum" visited a jobsite in Talinn (Estonia). The jobsite concerned two-way suspended elevated slabs on columns, of a 17 stories high residential tower. The two-ways spanning slabs with a span to depth ratio of 27, consist of steel fibre reinforced concrete only in the spans, and include structural integrity reinforcement running from column to column in the bottom of the slab.

Based upon the conclusions of the technical delegation, the Dutch "Cement&BetonCentrum" decided to install and test a real full scale slab in Eindhoven at the Technical University, The Netherlands.

It was however decided to test in this case 3 one-way spanning slabs of three consecutive continuous spans with steel fibre reinforced concrete only and without any structural integrity reinforcement (Figure 1, Figure 2). The walls and the slabs are reinforced with steel fibre reinforced concrete only.

The slab was indeed part of a tunnel form installation where the slab and the supporting walls are
cast together and the forms being released the next day in order to re-use them at the next location. The technique of tunnel forming is indeed frequently in use in The Netherlands for residential applications (Figure 3).

Figure 1: View of the tunnel formed test slabs of Eindhoven

Figure 2: Casting of the tunnel formed slabs and walls at TU Eindhoven

Figure 3: Typical scheme of installation of tunnel formed slabs in The Netherlands (source: www.joostdevree.nl)
2 TEST SET-UP

Three tunnel-formed slabs of three consecutive spans have been installed: slabs of 180 mm thickness, 2,30 m width and 5,40 m span while the supporting walls are of 250 mm thickness.

Each form is installed with a camber at mid-span of 18 mm as in the traditional method. The camber generally compensates for the sagging due to the own weight of the slab. After formwork removal the deformation was only 1 mm and in the next weeks until the test day another deformation of 5 mm was observed.

The tunnel formed slabs are separated from each other by a polystyrene foam filling between the moulds so that it was possible to test load 3 different frames. Each frame consists of 3 spans.

The mix design was of 350 kg CEMI and CEMIII with W/C <0,50 and included 50kg/m³ of type I hooked ends steel fibres of 0,9 mm diameter, 60 mm length and 1100 MPa tensile strength.

The mix was of a F4 fluidity so that only a light mechanical vibration was needed.

Walls and slabs have been poured together, with the same steel fibre reinforced concrete, no additional reinforcement was used.

As required for such a construction method in The Netherlands, the form release started 16 hours after the concrete installation. This requirement is mandatory in order to enable the general contractor to re-use the tunnel form as early as possible to form the next floor.

Therefore the mix design was such that the compressive strength needs to be at least of 14N/mm² at 16 hours.

The 28 day compressive strength at maturity is of 65N/mm² recorded with 150 mm cubes.

At 28 days f₁₁ and f₁₄ flexion strengths according to EN14651 were of respectively 6,1N/mm² and 5,3N/mm²

The loading consisted of a distributed loading intensity installed in incremental steps of a number of concrete blocks of different sizes to achieve a final surface load of up to 12kN/m² (Figure 4).

The blocks were unloaded by the crane on lumber at 500 mm spacing between them.

Figure 4: Loading test of an internal span

The three bays have a span of 5,4 m as shown in Figure 5 and Figure 6. Three sets of 3 such spans (corresponding to the letters A to I in Figure 5) have been installed in 2,3m width. The last set
of a three-span slab was of 4.5m width with cantilever balconies (corresponds to the letters J, K, L in Figure 5). The story height is 3.0 m. The floors of the first three parts are 180 mm thick, the part to which the balconies are attached and the actual balconies are 250 mm thick. The walls are 250 mm thick (Figure 3). The shell formwork is poured in situ. The basic requirement for the concrete composition was that the flexural tensile strength after 14 hours when stripping the formwork would be 1.4 N/mm² with a steel fibre content of 50 kg/m³. The 50 kg/m³ steel fibres were chosen to obtain a sufficiently ductile behaviour of the structure, so that after initial cracking redistribution thanks to support line rotation capacity would be possible, such that the collapse moment in the field could be obtained.
During the tests the sagging moment deflection measurements were recorded together with the resulting crack opening that was measured by LVDT’s: an overview of the measuring points is given in Figure 7. The LVDT’s 1 until 6 register the crack width, the LVDT’s 5 until 11 register the deformation. The objective of LVDT’s 1 till 4 was to identify a possible deformation of the walls but this was not the case.

Immediately prior to the test, the corresponding test beams, cubes and prisms were tested to characterise the composite material:
- compressive strength;
- flexural tensile strength $f_1$, $f_2$, $f_3$ and $f_4$ for steel fibre reinforced concrete according to EN14651;
- Young’s modulus.
3 FULL SCALE TESTS – LOAD APPLICATION

The test load was applied by placing concrete blocks on lumber as shown in Figure 8. Each loading intensity step increase was applied after stabilization of the former. The weight of the blocks was 1500 kg, 750 kg and 300 kg. The whole test had a duration of +/- 1 hour.

![Figure 8: Plan view: position of the load blocks](image)

The loading sequence: for each frame, the inner span has been tested first and consecutively the outer spans.

The spans G, H, I have been tested after 1 year and are not taken into account in this investigation and paper.

An overview of the collapse load when stabilization of the loading intensity was impossible and the deflection just before collapse are shown in Table 1.

It should be noted that test D is not representative as the supports were damaged by the prior demolition of the next middle field and hence there was no resisting hogging moment. Test D has not been taken into account in the further investigation.

Table 1: summary of single spans full scale tests performed. [1]

<table>
<thead>
<tr>
<th>Identification of test span and date of test As in Fig.3</th>
<th>Maximum UDL intensity (kN/m²) excl. self weight</th>
<th>Deflection before collapse (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal span B; June18, 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 D; Oct. 05, 2010</td>
<td>6,96</td>
<td>20</td>
</tr>
<tr>
<td>Edge span F; Sept 28, 2010</td>
<td>12,02</td>
<td>18</td>
</tr>
</tbody>
</table>
A deflection diagram recorded during loading application is shown in Figure 9.

Ultimate loading intensity and first crack loading intensities of 160kN and 120kN have been achieved and these figures are to be compared to the Dutch national housing service limit of 30kN.

![Real scale test deflection diagram](image)

Figure 9: Real scale test deflection diagram

The curves in the graph are the vertical deformations of 1 slab.

The average deflection due to the applied load just before collapse at the last stable observed step of loading, was 20.8 mm at a load of 12.0 kN/m². This agrees rather well with the predictions explained in the following paragraph. The 20.8 mm was the total deflection from demoulding until just before collapsing, thus the deflection caused only the superimposed loading intensity.

The floor initially behaves like a continuous beam on four support points, fixed in the wall. The moment at the support points is determining and cracking will first occur there. Cracking occurs at approximately 25.3 kNm/m (M = σW with σ = 6,5*0,72 N/mm²; the 6,5 N/mm² is the average first crack strength LOP from beam tests, the factor 0,72 is the factor converting the average value into a characteristic value). Next, the moment capacity at the support point will first increase to approx. 30.5 kNm/m (M = σW with σ = 7,7*0,72; the 7,7 N/mm² is fR1m from beam test) and then decrease to approx. 25.3 kNm/m, whilst the field moment will simultaneously increase from approx. 12.7 kNm/m to 30.5 kNm/m. Cracking is therefore expected to occur at 8 x (25.3 + 12.7) / 5,32² = 10.8 kN/m². After subtracting the dead load, this yields a load of 6.5 kN/m² loading intensity at negative moment first crack.

In reality, the initial cracking above the support point was found to occur at 7 to 8 kN/m².
After passing 30.5 kNm/m the support point moment decreases and the field moment increases. Based on the same data, cracking in the field should occur at a load of approx. 8x(28 + 25.3) / 5,32² = 15.2 kN/m². After subtracting the self-weight of 4.3 kN/m², this yields to a loading intensity of 10.9 kN/m².

Indeed, the initial positive moment crack was found to occur at approx. 10 to 11 kN/m².

Subsequently a number of other cracks as shown in Figure 10, occurred at the bottom of the field before the structure finally collapsed.

![Figure 10: View of the positive moment multiple cracking observed](image)

It can be concluded that the SFRC shell after cracking above the support points has a high residual capacity and that collapse is preceded by multiple cracking pattern in the field.

The load obtained, arithmetically calculated at 15.9 kN/m², versus the usual load of 4.3 + 0.5 + 1.75 = 6.55 kN/m² for residential buildings or 4.3 + 1.0 + 2.5 = 7.8 kN/m² for offices, opens up perspectives for the future. There is hardly any difference in capacity between an end field and a middle field. The fixing capacity could be generated by the concrete wall.

We can summarize the observations at failure as follows:

- Maximum load applied (average) of 12.18 kN/m² intensity with a deviation of s = 1.16 kN/m² or 9.5% coefficient of variation and thus much smaller figure than the typical 25 to 30% shown by standard prismatic specimen in flexion.
- Maximum average deflection observed of 20.83 mm at failure.
- Self weight of floor is of 24 kN/m³*0.18 m = 4.32 kN/m².
- Total loading intensity at failure of 16.5 kN/m²
- First crack loading intensity (average value) of 9.25 kN/m² (120 kN)


4.1. Material test results according to EN14651 [4]

There is material testing available and data tested are at 30 days and 129 days age.
At 30 days, the EN14651 flexion strengths are as follows:

1. \( f_{L,m} = 6.0 \text{ N/mm}^2, s = 0.52 \rightarrow f_{L,k} = 5.08 \text{ N/mm}^2 \)
2. \( f_{R,1,m} = 5.60 \text{ N/mm}^2, s = 2.1 \rightarrow f_{R,1,k} = 1.88 \text{ N/mm}^2 \)
3. \( f_{R,3,m} = 5.0 \text{ N/mm}^2, s = 1.96 \rightarrow f_{R,3,k} = 1.53 \text{ N/mm}^2 \)

While at 129 days they are as follows:

1. \( f_{L,m} = 6.53 \text{ N/mm}^2, s = 0.39 \rightarrow f_{L,k} = 5.84 \text{ N/mm}^2 \)
2. \( f_{R,1,m} = 7.65 \text{ N/mm}^2, s = 0.89 \rightarrow f_{R,1,k} = 6.07 \text{ N/mm}^2 \)
3. \( f_{R,3,m} = 6.25 \text{ N/mm}^2, s = 1.41 \rightarrow f_{R,3,k} = 3.75 \text{ N/mm}^2 \)

4.2. Minimum performance requirement for steel fibre concrete, to be considered as structural reinforcement

In order to function as structural reinforcement at ultimate state, the two minimum conditions to meet are: \( f_{R,1,k}/f_{L,k} > 0.4 \) (5.6-2) and \( f_{R,3,k}/f_{R,1,k} > 0.5 \) (5.6-3)

At 30 days age:
- \( f_{R,1,k}/f_{L,k} \) is 1.88/5.05 = 0.37 < 0.4 thus (5.6-2) almost fulfilled but adversely affected by the high deviation observed only on the small standard specimen at 30 days age.
- \( f_{R,3,k}/f_{R,1,k} \) is 1.53/1.88 = 0.81 > 0.5 thus (5.6-3) fulfilled and denotes the Class: 5b

At 129 days age:
- \( f_{R,1,k}/f_{L,k} \) is 7.65/5.84 = 1.29 > 0.4 thus (5.6-2) fulfilled
- \( f_{R,3,k}/f_{R,1,k} \) is 3.75/6.07 = 0.62 > 0.5 thus (5.6-3) fulfilled and denotes Class: 6a
4.3. Second requirement: structural ductility of the structure in SFRC

According to 7.7.2. Design Principals, of the Model Code, we observe that the necessary condition 7.7.2 δpeak ≥ 5δSLS is fulfilled.

The maximum service loading intensity (SLS) loading is the sum of the live load of (1,75 + 0,5) or 2,25 kN/m² and the self-weight of the slab or 4,32kN/m² and hence the total SLS loading intensity is of 6,57 kN/m².

As the slab was clamped at both ends the maximum deflection is: δSLS = ql4/384 = 0,89 mm and hence the condition 7.7.2 , δpeak ≥ 5δSLS is indeed verified 20,83 ≥ 5*0,89 = 4,45.

4.4. Bending moment capacity in Ultimate Limit State

κrd factor from sub-clause 4.6.2.2 takes into account favourable effects thanks to redistribution in the structure.

\[
P_{\text{maxk}} = P_{\text{maxm}} - 1.96s = 97.92 \text{kN}
\]

\[
f_m = 6.25 \frac{N}{\text{mm}}
\]

\[
f_k = 3.75 \frac{N}{\text{mm}}
\]

\[
\kappa_{\text{rd}} = \left( \frac{P_{\text{maxk}}}{P_{\text{maxm}}} \right) \left( \frac{f_m}{f_k} \right) = 1.141
\]

Plastic design:

The test slabs are here calculated according to yield line method.

The FIB Model Code draft version 2010 defines two possible stress distribution laws across the section of the slab: the linear model as shown in Figure 12 and the rigid plastic model (not shown). For the further analysis in this paper, the linear model has been selected.
The bending moment capacity of the section is calculated on the basis of average residual strength values, as well as characteristic residual strength values, and both with the values derived at 30 days and 129 days. A back calculation of the uniformly distributed loads that can be supported by the structure, was made consecutively, based on the bending moment capacities. These back calculated loads are compared to the real loads executed on the structure. The whole back calculated exercise does not take into account load nor material safety factors.

Let us fully calculate one case: based upon characteristic values after 129 days. The other cases follow the same rational and can be found back in Table 2.

Based upon the EN14651 results at 129 days age of flexion samples, we obtain:

\[
\begin{align*}
f_{r1k} &= 6.07 \text{ kN/m}^2 \\ f_{r3k} &= 3.75 \text{ kN/mm}^2 \\ f_{FTs} &= 0.45 f_{r1k} = 2.732 \text{ kN/mm}^2 \\ M_u &= \left(0.5 f_{r3k} - 0.2 f_{r1k}\right) \frac{h^2}{2} + \left(f_{FTs} - 0.5 f_{r3k} + 0.2 f_{r1k}\right) \frac{h^2}{6} = 21.889 \text{kN-m} \\
l &= 5.25 \text{ m} \\
\gamma_c &= 24 \text{ kN/m}^3
\end{align*}
\]
The real tested average maximum load (excl. own weight) is 12.0 kN/m² (c), which is 18 % higher than the predicted load, based on characteristic values on 129 days.

Table 2: Overview of prediction versus test

<table>
<thead>
<tr>
<th>@ 129 days</th>
<th>Mu (kNm/m)</th>
<th>Q_prediction_test (kN/m²)</th>
<th>q_test (kN/m²)</th>
<th>q_test/q_prediction_test (%) (c/a)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CHAR</td>
<td>21.9 (a)</td>
<td>10.2 (b)</td>
<td>12.0 (c)</td>
<td>118</td>
</tr>
</tbody>
</table>

It can be observed from the table that the test load can be back calculated using the Draft Model Code, version 2010, using the performance, measured in the beam test. One should notice, however, that the beam tests, used for the characterisation of steel fibre concrete, usually show high variation coefficients ranging from 10 to 40%. In the example explained in the paper the standard deviation is around 10% for fr1 which is at the low side. If a standard deviation of 40% in the beam test was encountered, then the predicted 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 predicts well the behaviour of the structure in bending, as long as the standard deviations in the characterisation tests are in the range 10-20%.

5 CONCLUSIONS

- All five spans tested showed a quite ductile behaviour resulting from a significant moment redistribution of moments together with a multiple cracking pattern under sagging moments.
- The deviations on the load bearing capacity between the five spans tested are significantly smaller than the deviations in the characterisation test EN14651.
- EN14651 test method shows a deviation that is typical to the test method only and not to the real behaviour of the structure as tested.
- 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 behaviour of the structure in bending, as long as the standard deviations in the characterisation tests are in the range 10-20%.
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