Impact loading on concrete slabs
Experimental tests and numerical simulations

ANDREAS ANDERSSON
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Report
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Abstract

In this report, the load capacity of concrete slabs subjected to concentrated loads is studied, considering both the static load capacity and the response due to impact of a falling mass. The purpose of the study is to gain more knowledge on the static and dynamic behaviour of the slabs and to use that knowledge in the assessment of the load capacity of inner lining systems in tunnels. The methodology involves experimental testing of a series of slabs, validation of numerical models and simulating the response of the inner lining system.

A total of 18 slabs were manufactured, consisting of shotcrete and reinforcement mesh. Some of the slabs also included steel fibre reinforced concrete (SRFC). The size of the slabs were 1.75×1.75×0.12 m, suspended in four hangers #1.2 m and loaded centric on an area of 0.2×0.2 m. In addition, a series of core samples and beams were cut from two of the slabs for material testing and verification of numerical models.

From the static load tests of the slabs, the load at cracking was obtained at 50 – 60 kN with a vertical midpoint displacement of 0.6 – 1.0 mm. The ultimate load ranged from 60 – 80 kN. The slabs showed significant ductility with a peak displacement of about 70 – 80 mm at post-failure. All slabs showed a two-way flexural failure. The concrete cover was in average 30 mm, measured from the compressive side, resulting in little difference between the crack load and the ultimate load. A vertical displacement of about 1 – 2 mm was required to obtain a crack width of 0.2 mm. Three slabs with only SRFC were tested until static failure, the ultimate load ranged from 85 – 90 kN but with less ductility compared to the slabs with rebar mesh.

Impact load tests were performed using a steel mass of 600 kg. The free fall height was varied from 1 – 2 m. The peak impact load varied from 200 – 250 kN, without any clear correlation with the height. The corresponding impulse load varied from 4.0 – 5.5 kNs with a clear correlation to the height. All slabs subjected to impact load showed a one-way flexural failure, the residual strength after impact was sufficient to carry the static load of the steel weight. Several of the slabs showed significant fallout of concrete during impact, in one extreme case a total of 16 kg. Three slabs were tested with an outer layer of 30 mm of SRFC, none of these slabs showed any significant fallout.

The static and dynamic response of the slabs have been simulated using nonlinear FE-models. The models generally show good agreement, both for static load, crack widths and response during impact. Similar models were used to simulate the response of the inner lining system. The results indicate a significant load capacity, both due to static and impact loading. The models are however not able to account for potential punching failure.

Keywords: Shotcrete, concrete slab, impact load, inner lining system, FEM.
Sammanfattning

I föreliggande rapport studeras lastkapaciteten hos betongplattor under såväl statisk belastning och stötbelastning från en fallande massa. Syftet med studien är att erhålla mer kunskap kring verkningsättet vid statisk belastning och stötbelastning samt att använda denna kännedom vid utvärdering av verkningsättet hos inklädnadssystem i tunnelar. Metodiken som används baseras på experimentell provning av en uppsättning betongplattor, verifiering av numeriska modeller och simulering av responsen hos inklädnadssystem.

Totalt 18 plattor har tillverkats, bestående av sprutbetong och armeringsnät. Några av plattorna utfördes även med stålfiberarmering. Plattornas storlek var 1.75×1.75×0.12 m, upphängda i fyra hängtag #1.2 m och belastade centriskt på en yta av 0.2×0.2 m. Dessutom utfördes provning på en serie kärnor och balkar, utskurna från likadana plattor, vilka användes för verifiering av de numeriska modellerna.

Från de statiska belastningarna uppmättes en spricklast på ca. 50 – 60 kN vid en vertikal nedböjning av 0.6 – 1.0 mm. Brottlasten varierade från 60 – 80 kN. Plattorna uppvisade betydande deformationskapacitet med en nedböjning efter brott på ca. 70 – 80 mm. Samtliga plattor uppvisade börjbrott i två riktningar. Täckande betongskikt var i genomsnitt 30 mm räknat från den tryckta zonen, vilket resulterade i liten skillnad mellan spricklast och brottlast. En vertikal nedböjning på ca. 1 – 2 mm krävdes för att uppnå en sprickvidd på 0.2 mm. Tre plattor med endast stålfiberarmering provades till statiskt brott, brottlasten var 85 – 90 kN men uppvisade mindre deformationskapacitet jämfört med plattorna med armeringsnät.

Stötbelastningar utfördes med en stålvikt på 600 kg. Fallhöjden varierades från 1 – 2 m. Största momentana reaktionskraft varierade mellan 200 – 250 kN, utan tydlig korrelation med fallhöjden. Motsvarande impulsmängd varierade från 4.0 – 5.5 kNs med en tydlig korrelation mot fallhöjden. Samtliga plattor som utsattes för stötbelastning uppvisade börjbrott i en riktning med tillräcklig restbärförmåga att bära den statiska lasten av fallvikten. Flertalet av plattorna visade betydande utstötning av betong på plattans undersida, i extremfallet totalt 16 kg. Tre av plattorna som provades försågs med ett yttre lager av 30 mm stålfiberarmering, ingen av dessa uppvisade någon betydande utstötning.


Nyckelord: Sprutbetong, betongplatta, stötbelastning, inklädnadssystem, FEM.
Preface

This report was prepared at the Division of Structural Engineering and Bridges, KTH, and consists of the work according to contract TRV2014/9989, commissioned by Trafikverket and funded by EU (TEN-T).

The project was initiated by Mr. Mattias Roslin at Trafikverket, his feedback and interest in the project is greatly acknowledged. Thanks are also due to Mr. Arvid Taube, Trafikverket, for long and fruitful discussions.

The manufacturing of the concrete slabs was performed by Byggs Sprutbetong AB.

Mr. Claes Kullberg, Division of Concrete Structures, KTH, and Mr. Stefan Trillkott, Division of Structural Engineering and Bridges, KTH, performed the experimental testing and instrumentation together with the author. Mr. Claes Kullberg also designed the frame for the dynamic tests. The author which to express his sincere gratitude to both Mr. Claes Kullberg and Mr. Stefan Trillkott for their expertise and long experience in experimental testing.

The facilities at the Cement and Concrete Institute (CBI) were used during testing of concrete core samples and simple beams.

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Andreas Andersson

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The author is responsible for the content of this publication.
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Chapter 1

Introduction

1.1 Background

In this report, impact on concrete slabs aims at the application of inner lining systems in tunnels, subjected to impact from falling rocks. The inner lining concept is a method to prevent water leakage and the risk of icing inside tunnels and is frequently used in countries with cold climate. Blocks of falling ice may result in a severe safety risk in both road and railway tunnels. Although the inner lining is not part of the main load bearing system, it is designed to withstand several different load scenarios. In this report, the case of impact load from falling rocks is studied.

The Swedish requirements for tunnel design, TRVK Tunnel 11 (Trafikverket, 2011), stipulates the following design loads related to rock impact on inner lining systems:

- a 600 kg block landing on a square surface of 0.2x0.2 m (D.3.2.15),
- an extreme block load of 6000 kg acting on a square surface of 1x1 m (D.4.7).

The free fall height i.e. the distance to the rock surface is typically less than 0.5 m but may span up to 1.5 m in some cases. A too conservative design may result in an unnecessary thick structure and lack of knowledge of the impact phenomena may result in an unsafe design.

1.2 Aim, scope and limitations

The overall questions to be studied are:

- what is the load capacity of inner lining systems subjected to impact loading,
- what kind of model can describe the behaviour sufficiently accurate.

The aim of the research presented in this report is to determine the static and dynamic load capacity of concrete slabs by experimental testing. The properties and manufacturing of the slabs aim at being representative for inner lining systems in tunnels. The width and boundary conditions of the slabs are however scaled to fit the experimental setup. The results may therefore not be directly translated to the global load capacity of the tunnel lining. Instead, the experimental results serve as validation
of numerical models that in turn are used in predicting the capacity of the inner lining system.

The following possible failure scenarios are identified:

- global flexural failure of the inner lining,
- local punching failure of the inner lining,
- failure of the rock bolts, tensile, buckling or pull-out,
- spalling at the intrados of the inner lining system.

During a series of static slab tests, the displacement and crack widths were recorded until failure. The results may have application in evaluating the consequences of cracks observed during tunnel inspections.

1.3 Manufacturing of the slabs

A total of 18 slabs were manufactured on April 1st, 2014. The dimensions of the slabs are 1750x1750x120 mm, Figure 1.2. Rock bolt anchorage with reinforcement cross d8 was installed #1.2 m prior to casting of the slabs. A rebar mesh d8#100 located centric in the cross-section was used in 13 of the slabs, Figure 1.4. Out of these, 3 slabs were made with an outer layer of steel fibre reinforced shotcrete (SFRC), Figure 1.5. Four slabs were casted with only SFRC and no rebar mesh. One slab was casted with plain shotcrete and no rebar mesh, for further core sample tests and simple beam tests. The slabs during shotcrete blasting is shown in Figure 1.1. The configuration of the slabs and type of tests performed are summarised in Table 1.1.

![Figure 1.1: Manufacturing of the concrete slabs, a) formwork with rock bolt anchors and reinforcement mesh, b) during shotcrete blasting.](image)
1.3. MANUFACTURING OF THE SLABS

Figure 1.2: Drawing of the slab [mm].

Figure 1.3: Photo of the slab before casting.
CHAPTER 1. INTRODUCTION

Figure 1.4: Cross-section illustrating the anchorage and rebar mesh.

Figure 1.5: Cross-section illustrating the anchorage and rebar mesh, case with an outer layer of SFRC.

Figure 1.6: Cross-section illustrating the anchorage, case without rebar mesh and only SFRC.
After casting, the slabs were subjected to 7 days of water curing in outdoor environment.

Table 1.1: Summary of the slab configurations and performed tests.

<table>
<thead>
<tr>
<th>Lit.</th>
<th>rebar</th>
<th>date tested</th>
<th>type of test</th>
<th>Lit.</th>
<th>rebar</th>
<th>date tested</th>
<th>type of test</th>
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<td>2014-09-01</td>
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<td>steel fibre</td>
<td>2014-09-01</td>
<td>static</td>
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Chapter 2

Beams in flexural failure

2.1 Test setup

A series of standard 4-point beam bending tests were performed to evaluate the flexural capacity of the concrete. 6 beams were cut from each of slab S11 and S16. The target dimension was 500x125x75 mm. The tests were performed according to EN 12390-5 and EN 14488-3. The setup is shown in Figure 2.1 and Figure 2.2.

![Figure 2.1: Schematic of the 4-point beam bending tests.](image1)

![Figure 2.2: Beam B1 during testing.](image2)
2.2 Experimental results, plain concrete

The beams cut from slab S11 consists of plain concrete without any reinforcement and showed a very sudden brittle failure. For beam B1 a load rate of 0.004 mm/s and a sampling frequency of 3 Hz. For beam B2 to B6, the load rate was decreased to 0.002 mm/s and the sample frequency increased to 10 Hz. The failure of the beams is illustrated in Figure 2.3.

The load-displacement paths are presented in Figure 2.4. Due to the brittle rupture, the softening part from cracking to failure is often described by a single point and should hence be treated with care.

Figure 2.3: Beam B1 to B6 with plain concrete, after failure.
2.2. EXPERIMENTAL RESULTS, PLAIN CONCRETE

Figure 2.4: Measured load-displacement, beam B1 to B6.

The results from the beam with plain concrete are summarised in Table 2.1. The tensile strength is calculated according to Equation (2.1), where \( L = 3d \) and \( F_{\text{max}} \) the total applied peak load. The Young’s modulus is estimated by using Equation (2.2), valid in the elastic range. A linear regression is performed from 0.1\( F_{\text{max}} \) to 0.6\( F_{\text{max}} \).

\[
f_{\text{ct}} = \frac{M}{W} = \frac{(F_{\text{max}}/2)L/3}{wh^2/6} = \frac{F_{\text{max}}L}{wh^2}
\]

\[
\delta_{\text{mid}} = \frac{23}{108} \frac{(F/2)L^3}{6EI} \rightarrow E = \frac{23L^3}{108wh^3} \frac{F}{\delta_{\text{mid}}}
\]

Table 2.1: Summary of results, beam B1 to B6 with plain concrete.

<table>
<thead>
<tr>
<th></th>
<th>Lit.</th>
<th>age (days)</th>
<th>3d (mm)</th>
<th>w (mm)</th>
<th>h (mm)</th>
<th>a (mm)</th>
<th>( \delta_{\text{cr}} ) (mm)</th>
<th>( F_{\text{max}} ) (kN)</th>
<th>( f_{\text{ct}} ) (MPa)</th>
<th>( E_c ) (GPa)</th>
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<td>132</td>
<td>455</td>
<td>129</td>
<td>77</td>
<td>166</td>
<td>0.12</td>
<td>8.3</td>
<td>5.0</td>
<td>30.9</td>
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<tr>
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<td>132</td>
<td>455</td>
<td>120</td>
<td>78</td>
<td>148</td>
<td>0.13</td>
<td>11.0</td>
<td>6.8</td>
<td>34.6</td>
<td></td>
</tr>
<tr>
<td>B3</td>
<td>132</td>
<td>455</td>
<td>122</td>
<td>78</td>
<td>194</td>
<td>0.13</td>
<td>10.6</td>
<td>6.5</td>
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<tr>
<td>B4</td>
<td>132</td>
<td>455</td>
<td>122</td>
<td>75</td>
<td>175</td>
<td>0.13</td>
<td>10.6</td>
<td>7.0</td>
<td>35.4</td>
<td></td>
</tr>
<tr>
<td>B5</td>
<td>132</td>
<td>455</td>
<td>125</td>
<td>75</td>
<td>168</td>
<td>0.15</td>
<td>10.4</td>
<td>6.8</td>
<td>34.0</td>
<td></td>
</tr>
<tr>
<td>B6</td>
<td>132</td>
<td>455</td>
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<td>78</td>
<td>187</td>
<td>0.12</td>
<td>10.4</td>
<td>6.3</td>
<td>33.0</td>
<td></td>
</tr>
</tbody>
</table>

**average:**

|   |   |   |   |   |   |   | 10.2 | 6.4 | 33.7 |

**std:**

|   |   |   |   |   |   |   | 0.9 | 0.7 | 1.6 |

**covar:**

|   |   |   |   |   |   |   | 0.09 | 0.12 | 0.05 |
2.3 Experimental results, with SFRC

The beams cut from slab S16 consists of 35 mm steel fibres (approximately 55 kg/m³). As expected, the beams showed a more ductile behaviour compared to the beams from slab S11. Beam FB6 was tested first and with a load rate of 0.004 mm/s and a sample frequency of 10 Hz. The remaining beams FB1 to FB5 were tested with a load rate of 0.006 mm/s.

![Figure 2.5: Beam FB1 to FB6 with SRFC, after failure.](image)

The load-displacement paths are presented in Figure 2.6 and Figure 2.7 and the results are summarised in Table 2.2. The behaviour until peak load is comparable with the beams from slab S11. The residual strength is about 10% of the peak load at 5 mm mid-span deflection.
2.3. EXPERIMENTAL RESULTS, WITH SFRC

Figure 2.6: Measured load-displacement, beam FB1 to FB6.

Figure 2.7: Measured load-displacement, beam FB1 to FB6, at cracking.

Table 2.2: Summary of results, beam FB1 to FB6 with SFRC.

<table>
<thead>
<tr>
<th>Lit.</th>
<th>age (days)</th>
<th>3d (mm)</th>
<th>w (mm)</th>
<th>h (mm)</th>
<th>a (mm)</th>
<th>$\delta_{cr}$ (mm)</th>
<th>$F_{max}$ (kN)</th>
<th>$f_{ct}$ (MPa)</th>
<th>$E_{c}$ (GPa)</th>
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<tr>
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<td>133</td>
<td>455</td>
<td>124</td>
<td>76</td>
<td>175</td>
<td>0.15</td>
<td>9.9</td>
<td>6.3</td>
<td>28.1</td>
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<tr>
<td>FB2</td>
<td>133</td>
<td>455</td>
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<td>77</td>
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<td>455</td>
<td>120</td>
<td>75</td>
<td>136</td>
<td>0.15</td>
<td>9.0</td>
<td>6.1</td>
<td>26.1</td>
</tr>
<tr>
<td>FB4</td>
<td>133</td>
<td>455</td>
<td>124</td>
<td>75</td>
<td>226</td>
<td>0.15</td>
<td>9.7</td>
<td>6.4</td>
<td>30.5</td>
</tr>
<tr>
<td>FB5</td>
<td>132</td>
<td>455</td>
<td>127</td>
<td>77</td>
<td>229</td>
<td>0.13</td>
<td>10.1</td>
<td>6.3</td>
<td>35.8</td>
</tr>
<tr>
<td>FB6</td>
<td>132</td>
<td>455</td>
<td>124</td>
<td>77</td>
<td>218</td>
<td>0.12</td>
<td>9.6</td>
<td>5.8</td>
<td>33.0</td>
</tr>
</tbody>
</table>

average: 9.9  6.2  31.1
std: 0.7  0.3  3.6
covar: 0.07  0.04  0.12
2.4 Core samples

A series of concrete core samples were taken from slab S11 and S16. According to SS 13 72 07, cores with diameter $d = 100$ mm and height $h = 100$ mm can be used to determine the object-specific compressive strength. To determine the cylinder strength class, the dimensions should instead be $d = 150$ mm and $h = 300$ mm ($d = 160$ mm and $h = 320$ mm in EN 12390-3). The cylinder strength class may be estimated from core samples of different size, age and curing temperatures using Equation (2.3) and Equation (2.4), from SS 13 72 07. For the current series, $\beta_1 = 1.05$ is a factor that depends on the diameter, $\beta_2$ depends on the age and $\beta_3 = 1.20$ depends on the slenderness $h/d$. The factor $j_{20}$ is the equivalent curing time in days at a temperature different from 20 °C. As result, $j_{20}$ is used to calculate the factor $\beta_2$ that accounts for the curing time. Based on temperature data, $j_{20} = 31$ days and $\beta_2 \approx 1.0$. Finally, $f_2 \approx 0.8f_{obs,2}$. The compressive and splitting strength are determined using Equation (2.5).

\[
f_2 = \frac{f_{obs,2}}{\beta_1 \beta_2 \beta_3}
\]  
\[
j_{20} = \sum \beta_5 j_T,
\]
\[
\beta_5 = \begin{cases} 
0 & \text{for } T < 4°C \\
\frac{T - 1}{4} & \text{for } 4 \leq T \leq 20°C
\end{cases}
\]

\[
f_{cc} = \frac{F_{max}}{A}, f_{ct} = \frac{2F_{max}}{\pi dh}
\]

The results are summarised in Table 2.3 to Table 2.6. The steel fibres does not influence the compressive strength but increase the splitting strength by about 30%.

Table 2.3: Summary of results, compression tests of cores without steel fibres.

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<th>$h$ (mm)</th>
<th>$\rho$ (kg/m$^3$)</th>
<th>$F_{max}$ (kN)</th>
<th>$f_{cc}$ (MPa)</th>
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<td>94.5</td>
<td>102.5</td>
<td>2224.3</td>
<td>512.0</td>
<td>73.0</td>
</tr>
<tr>
<td>C2</td>
<td>84</td>
<td>94.8</td>
<td>104.0</td>
<td>2225.3</td>
<td>520.1</td>
<td>73.7</td>
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average: 73.0  
std: 1.4  
covar: 0.02

\(^1\) Temperature data taken from http://slb.nu/lvf
Table 2.4: Summary of results, splitting tests of cores without steel fibres.

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<th>$f_{\text{ct}}$ (MPa)</th>
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Table 2.5: Summary of results, compression tests of cores with steel fibres.

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<th>h (mm)</th>
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<th>$F_{\text{max}}$ (kN)</th>
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Table 2.6: Summary of results, splitting tests of cores with steel fibres.

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<th>d (mm)</th>
<th>h (mm)</th>
<th>ρ (kg/m³)</th>
<th>$F_{\text{max}}$ (kN)</th>
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</tbody>
</table>
2.5 FE-models

2.5.1 Geometry

The response from the beam tests have been simulated using the FE-model depicted in Figure 2.8. The model consists of 4-noded plane stress elements and is analysed in the commercial software SOLVIA03. The same model is used for both the plain concrete beams and the SFRC-beams, but with different material properties. Material nonlinearity and large displacements is accounted for and the load is applied using displacement control.

![Figure 2.8: 2D plane stress model (dL=5 mm), at failure.](image)

2.5.2 Material models for concrete

Two different material models are used to describe the nonlinear behaviour of concrete; a uni-axial multi-linear model and a tri-axial plasticity-based model. The uni-axial model is a rotating crack model and follows the same stress-strain path for loading and unloading. It is hence primarily intended for monotonic loading. The main advantage of the model is its simplicity and ability to describe arbitrary stress-strain paths. The tri-axial model is based on plasticity using a smeared crack approach and strain softening. Once a crack occurs, it is oriented orthogonal to the current principal tensile strain. The crack direction is then locked in that direction regardless of the subsequent principal tensile strain field. This is known as an orthogonal fixed crack model. By default, relations in bi-axial and tri-axial stress states are predefined and only the uniaxial stress-strain relations are required, if no further information of the multi-axial stress state is known. The model may be used for cyclic loading. The tri-axial model is further described in (Bathe & Ramaswamy, 1979) and (Bathe et al., 1989).

When following tensile failure, the stress normal to a tensile failure plane is decreased linearly to $k\varepsilon_t$ at zero tensile strength according to Figure 2.9a. The tri-axial failure envelope in tension never exceeds the uniaxial tensile strength $\sigma_t$. At tensile failure, the shear stress is controlled by a shear stiffness reduction factor, addressed by the command \texttt{SHEFAC}. The shear stress in an active crack plane is limited by \texttt{SHEFAC}$\cdot\sigma_t$. By default, \texttt{SHEFAC} = 0.5. Increasing the shear stiffness reduction factor may produce a more stable analysis during multi-axial stress states. The compressive softening is
described by a polynomial function (Bathe & Ramaswamy, 1979) but may be fitted to the softening curves in e.g. Model Code 2010.

![Figure 2.9: Failure envelopes for concrete, a) stress-strain relations for both uni-axial loading \((\varepsilon, \sigma)\) and tri-axial loading \((\varepsilon', \sigma')\) using \(\sigma_2 = \sigma_3 = 0.2\sigma_1\), b) bi-axial failure envelope, reproduced from (SOLVIA, 2007).](image)

The flexural strength of plain concrete is governed by the tensile strength \(f_{ct}\) and the displacement depends on the Young’s modulus \(E_0\). The post-failure and cracking is mainly governed by the fracture energy \(G_f\), described in Equation (2.6). Different models for the strain softening are shown in Figure 2.10. Using a stress-strain formulation, the fracture softening becomes mesh-dependent.

\[
G_f = \int_0^w \sigma_i dw
\]  

(2.6)

![Figure 2.10: Different models for tension softening in concrete, a) plain concrete, b) SFRC.](image)

In the case of linear tension softening, the crack width \(w_k\) at zero tensile stress is given by Equation (2.7a), which expressed in strain gives \(\varkappa\) according to Equation (2.7b). For plain concrete, the crack distance \(s_{rm}\) is set to the element length. According to
Model Code 2010, the bi-linear tension softening is described by Eq. (2.8). An exponential tension softening was proposed by (Cornelissen et al., 1986), based on curve fitting of a large set of experimental results, Equation (2.9). For plain concrete, $c_1 = 3$ and $c_2 = 7$ was used, taken from (Cornelissen et al., 1986). In the following analysis, the crack width $w_c$ is adjusted to comply with a given fracture energy. For SFRC, the tensile stress at which the fibres are activated are often set to $\alpha = 0.45$, and the strain from $f_{ct}$ to $\alpha f_{ct}$ is set to 0.1‰, e.g. (Vandevalle, 2000).

$$G_t = \frac{f_{ct} w_k}{2} \rightarrow w_k = \frac{2G_t}{f_{ct}}$$  \hspace{2cm} (2.7a)

$$\kappa \varepsilon_i = \frac{w_k}{s_{cm}}, \varepsilon_i = \frac{f_{ct}}{E_0} \rightarrow \kappa = \frac{2G_t E_0}{s_{cm} f_{ct}^2}$$  \hspace{2cm} (2.7b)

$$w_i = \frac{G_f}{f_{ct}}, w_c = \frac{5G_f}{f_{ct}}$$  \hspace{2cm} (2.8)

$$\sigma_i = f_{ct} \left(1 + \left(c_1 \frac{w}{w_c}\right)^3 \right) \exp\left(-c_2 \frac{w}{w_c}\right)$$  \hspace{2cm} (2.9)

### 2.5.3 Concrete quality classes

In all simulations, the average values of the concrete strength parameters are used, without any partial coefficients of safety factors. For concrete class C30 to C60, the relevant parameters are presented in Table 2.7. The fracture energy is described according to Eq. (2.10).

| Concrete properties for class C30 – C60 (Model Code 2010). |
|-----------------|-----|-----|-----|-----|
| $E_{ci}$ (GPa)  | 33.6| 36.3| 38.6| 40.7|
| $\varepsilon_{c1}$ (%‰)| -2.3| -2.4| -2.6| -2.7|
| $\varepsilon_{cu}$ (%‰)| -3.5| -3.5| -3.4| -3.3|
| $f_{cm}$ (MPa)  | 38  | 48  | 58  | 68  |
| $f_{ctm}$ (MPa) | 2.9 | 3.5 | 4.1 | 4.4 |
| $G_t$ (N/m)     | 140 | 147 | 152 | 156 |

$$G_t = 73 \left(f_{cm}/10^6\right)^{0.18}$$  \hspace{2cm} (2.10)
2.5.4 Plain concrete beams

A mesh convergence test is performed, using the model presented in Figure 2.8. The results are presented in Figure 2.11 and show that \( dL = 5 \text{ mm} \) is needed to obtain a converged tension softening path. For the flexural capacity however, a significantly more coarse mesh may be used. The uni-axial concrete model is used, with the bi-linear tension softening curve.

![Figure 2.11: Concrete C50, mesh convergence test, (plane stress model).](image)

Using the same geometry, a model with 8-noded solid elements and a model with 4-noded shell elements is analysed, Figure 2.12. The plane stress and solid model show similar results, but the shell model severely overestimates the flexural capacity. The reason is likely that the fracture energy of the shell model depends on all integration points along the thickness direction and is influenced by bending, whereas for the plane stress and solid elements, the strain is constant over the element length.

![Figure 2.12: Concrete C50, influence of the element type.](image)

Based on the reference plane stress model, the influence of the concrete class is shown in Figure 2.13. The increased flexural strength is proportional to the increase in tensile strength, but similar tension softening paths are obtained.
The influence of the tensile strength is further studied in Figure 2.14, showing similar results but also an influence on the tension softening. In Figure 2.15, the fracture energy is varied, influencing both the flexural capacity and the tension softening.

Figure 2.13: Variation of the concrete quality class.

Figure 2.14: Concrete class C50, variation of the tensile strength.

Figure 2.15: Concrete class C50, variation of the tensile fracture energy.
The influence of the tension softening curve is illustrated in Figure 2.16. Similar results are obtained with the bi-linear and the exponential curve, whereas the linear curve results is some higher flexural strength and a more sudden softening. The tri-axial model is based on a linear tension softening, but does not produce the exact same result as the corresponding uni-axial model. The reason for this may be due to bi-axial stress state.

**Figure 2.16:** Concrete C50, influence of different tension softening curves.

### 2.5.5 Beams with SFRC

Similar studies are performed for the SFRC beams as for the plain concrete beams. The tension softening follows Figure 2.10b, similar to the bi-linear curve but with a higher fracture energy and post tension cracking. The fracture energy was not measured directly from the experiments, but have been estimated by calibrating the FE-model. The results are illustrated in Figure 2.17 and further analyses are performed with $8G_f$, where the value of $G_f$ is referred to Table 2.7.

**Figure 2.17:** Concrete class C50, variation of the fracture energy.
Results for different concrete classes are shown in Figure 2.18. Similar to the plain concrete beams, the increased flexural strength is mainly due to the tensile strength, but since the fracture energy is significantly larger, the tension softening is also influenced.

The tensile strength at which the steel fibres are activated primary influence the tension softening path, Figure 2.19. The value $\alpha=0.45$ shows a good fit with the experiments.

![Figure 2.18: Variation of the concrete quality class, model with 8Gf.](image)

![Figure 2.19: Concrete class C50, variation of post-tensile strength $\alpha$.](image)

The crack width at which the steel fibres are activated was not measured during the experiments, but is studied in Figure 2.20. Larger values of $w_f$ means that more fracture energy is taken by the concrete in early cracking, which results in a larger flexural capacity. Best fit with the experiments is obtained for $w_f = 0.03$ mm.
Figure 2.20: Concrete class C50, variation of the steel fibre crack width $w_f$.

Similar to the case of plain concrete, the shell model overestimates the flexural strength for SFRC, Figure 2.21.

Figure 2.21: Concrete class C50, compare plane stress and shell model.
Chapter 3

Slabs subjected to static load

3.1 Test setup

A series of static tests were performed to determine the static load capacity of the slabs. The rock bolts (threaded M20 bars) were fastened to the existing mounts in the laboratory floor, Figure 3.1. The existing mounts in the floor are spaced 1.0 m, auxiliary beams were used to fasten the bars with 1.2 m spacing. The load is centric on the slab via a 200×200×50 mm steel plate and a hydraulic jack. A load cell is placed between the steel plate and the hydraulic jack. The setup is shown in Figure 3.2.

Figure 3.1: Fastening of the bars, auxiliary beams and the hydraulic jack used during the static load tests.

The slab was instrumented with LVDT\(^2\) transducers to measure displacements and crack widths. Sensor p\(_1\) – p\(_4\) measured the vertical deflection at the support points, mainly to account for any slip during initial loading, but also the elasticity of the M20 bars. Sensor p\(_5\) – p\(_8\) measured the vertical displacement at the quarter-points and p\(_9\) at

\(^2\) Linear Variable Differential Transformer, measures displacement.
the midpoint. The remaining sensors $p_{10}$ – $p_{17}$ measured crack width at the slab top surface. The vertical sensors were mounted to auxiliary beams shown in Figure 3.3.

Figure 3.2: Layout and instrumentation during the static load tests.
3.2 Results, slabs with rebar mesh

Slab S1 – S3 consisting of rebar mesh were tested until static failure. The load–vertical displacement response is presented in Figure 3.5 until failure and in Figure 3.4 for the cracking load. The average support displacement from sensor p1 to p4 is subtracted from the midpoint displacement at sensor p9. The results are summarised in Table 3.1. The results from each slab is analysed individually below.

Table 3.1: Summary of load and displacement at cracking and ultimate load.

<table>
<thead>
<tr>
<th>Lit.</th>
<th>( P_{crack} ) (kN)</th>
<th>( d_{crack} ) (mm)</th>
<th>( P_{ul} ) (kN)</th>
<th>( d_{ul} ) (mm)</th>
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<td>57.2</td>
<td>0.69</td>
<td>58.3</td>
<td>70.0</td>
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</table>

Figure 3.3: Instrumentation of slab S2, auxiliary beams to support the LVDT transducers.
Figure 3.4: Load – vertical displacement response for slab S1 – S3, at cracking.

Figure 3.5: Load – vertical displacement response for slab S1 – S3, until failure.
3.2. RESULTS, SLABS WITH REBAR MESH

Slab S1

Sensors $p_{10} - p_{17}$ for crack width measurements were installed successively after the formation of cracks in the two directions. The first crack appeared in the $x$-direction at a load of 30 kN. The loading was stopped and sensor $p_{11}$, $p_{15}$, $p_{17}$ and $p_{13}$ were installed. The crack width at this point was measured manually to 0.4 mm at all four sensor locations. At the peak load of 59 kN, a crack formed in the $y$-direction. The load was again stopped and sensors $p_{12}$, $p_{16}$, $p_{14}$ and $p_{10}$ were installed. The crack widths were measured manually, $w_{12} = 0.5$ mm, $w_{16} = 0.7$ mm, $w_{14} = 0.8$ mm and $w_{10} = 0.8$ mm. The loading was then continued until failure at 65 kN. The position of the sensors and the crack pattern at failure is shown in Figure 3.6.

![Slab S1 after failure](image)

Figure 3.6: Slab S1 after failure.

The deflection of the slab is shown in Figure 3.7. The slab shows a significant deformation capacity, even after the peak load is reached.
The crack width $w$ versus the load $F$ is shown in Figure 3.8. All sensors were installed after initial cracking, and the crack width at installation was measured manually. Figure 3.9 presented the crack width versus midpoint displacement (with the support displacement subtracted). At position $p_{11} - p_{15}$ and $p_{13} - p_{17}$ the vertical displacement was 2.5 mm at the initial crack width of 0.4 mm. For the second crack, the displacement at $p_{12} - p_{16}$ and $p_{10} - p_{14}$ was 6.5 mm at the initial crack width of 0.5 mm and 0.8 mm respectively.

Figure 3.10 shows the crack width directly after installation of sensor $p_{12}$. The cracks along sensor $p_{10} - p_{14}$ and $p_{12} - p_{16}$ are shown in Figure 3.11 and along $p_{13} - p_{17}$ and $p_{11} - p_{15}$ in Figure 3.12.
3.2. RESULTS, SLABS WITH REBAR MESH

Figure 3.8: Crack width opening of slab S1, initial crack width based on manual measurement using a crack width ruler.

Figure 3.9: Midpoint displacement vs. crack widths for slab S1.
CHAPTER 3. SLABS SUBJECTED TO STATIC LOAD

Figure 3.10: Crack at sensor $p_{12}$.  

Figure 3.11: Cracking along sensor $p_{10}$, $p_{14}$ and $p_{12}$, $p_{16}$.  

Figure 3.12: Cracking along sensor \( p_{13}, p_{17} \) and \( p_{11}, p_{15} \).
Slab S2

All of sensor $p_{10} - p_{17}$ was installed prior to loading of slab S2 with the intention to measure the full crack – displacement response. To increase the probability of measuring the full crack propagation, sensor $p_{10} - p_{13}$ were installed with an overlap to sensors $p_{14} - p_{17}$. The first crack appeared along sensor $p_{10}$, $p_{14}$ and $p_{12}$, $p_{16}$. Two cracks formed over $p_{14}$ and $p_{12}$, but since only one crack appeared over adjacent $p_{10}$ and $p_{16}$, the second crack can be estimated as the difference between the gauges. At the load 25 kN a crack appeared beside $p_{11}$. The loading was stopped and sensor $p_{11}$ was reinstalled over the crack. The initial crack width was measured to 0.4 mm before the loading was continued. At a load of 40 kN, a crack appeared beside $p_{15}$. Similar procedure as for $p_{11}$ was performed and the crack width was measured to 0.4 mm. At a load of 48 kN, a crack appeared over $p_{13}$ and a second crack over $p_{17}$ at 73 kN. Slab S2 after failure is shown in Figure 3.13.

Figure 3.13: Slab S2 after failure.

The crack width vs. load is shown in Figure 3.14. At sensor $p_{12}$, $p_{16}$, $p_{10}$, $p_{14}$ and $p_{13}$, $p_{17}$, the initial cracking was recorded. At peak crack load, the crack width is less than 0.1 mm.

Figure 3.15 show the crack width vs. vertical displacement. At sensor $p_{12}$, $p_{16}$ and $p_{10}$, $p_{14}$, the vertical displacement is about 1.2 – 1.5 mm at a crack width of 0.2 mm. At sensor $p_{13}$, the vertical displacement is 2.7 mm at the crack width of 0.2 mm.
3.2. Results, Slabs with Rebar Mesh

Figure 3.14: Crack width of slab S2, $w_{11}$ and $w_{15}$ installed after initial crack.

Figure 3.15: Midpoint displacement vs. crack widths for slab S2, sensor $p_{11}$ and $p_{15}$ were installed after initial crack.
Slab S3

The instrumentation of slab S3 was similar to the procedure of slab S2. The first crack appeared beside $p_{12}$ and $p_{16}$. Sensor $p_{12}$ was reinstalled but $p_{16}$ was left in its original position, $w_{12} = 0.1$ mm was measured. The second crack appeared beside $p_{10}$ and $p_{14}$. Sensor $p_{10}$ was reinstalled but $p_{14}$ was left in its original position, $w_{10} = 0.3$ mm was measured. Additional cracks formed at $p_{13}$ and later at $p_{15}$. Slab S3 after failure is shown in Figure 3.16.

![Slab S3 after failure](image)

Figure 3.16: Slab S3 after failure.

The crack width vs. load is shown in Figure 3.17. Similar to slab S2, the crack widths at peak crack load is less than 0.1 mm.

The crack width vs. vertical displacement is shown in Figure 3.18. Sensor $p_{10}$ and $p_{12}$ was installed after initial cracking. Sensor $p_{11}$, $p_{14}$, $p_{16}$ and $p_{17}$ was not repositioned after cracking. Sensor $p_{17}$ lost contact when the parallel crack at $p_{13}$ formed. For sensor $p_{13}$, the crack width was less than 0.1 mm at a vertical displacement of 2.5 mm, rapidly increasing to 0.5 mm crack width at a vertical displacement of 2.7 mm. Similar behaviour was found at sensor $p_{17}$, but starting at a vertical displacement of 3.2 mm.
3.2. RESULTS, SLABS WITH REBAR MESH

Figure 3.17: Crack width of slab S3.

Figure 3.18: Midpoint displacement vs. crack widths for slab S3.
3.3 Results, slabs with only SFRC

Slab S15, S17 and S18 consisted of only steel-fibre reinforced concrete (SFRC), without any reinforcement mesh. The ultimate load capacity was in average 20% higher than the slabs with rebar mesh, but the steel fibre reinforced slabs are less ductile. All of the steel fibre reinforced slabs failed in one-way bending and split in half after cracking. Table 3.2 presented the ultimate loads and appertaining vertical midpoint displacement. The thickness of the slabs were measured at seven points along each fracture surface, the average value is presented in Table 3.2. All three slabs were thicker than the target value of 120 mm. The largest thickness was measured for slab S17, also showing the largest ultimate load capacity.

Table 3.2: Summary of load and displacement at cracking and ultimate load.

<table>
<thead>
<tr>
<th>Lit.</th>
<th>$P_{uls}$ (kN)</th>
<th>$d_{uls}$ (mm)</th>
<th>$t_{mean}$ (mm)</th>
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<tbody>
<tr>
<td>S15</td>
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<td>1.3</td>
<td>140</td>
</tr>
<tr>
<td>S17</td>
<td>108.5</td>
<td>2.3</td>
<td>150</td>
</tr>
<tr>
<td>S18</td>
<td>85.7</td>
<td>2.9</td>
<td>135</td>
</tr>
</tbody>
</table>

The load – displacement for the SFRC slabs are presented in Figure 3.19 and Figure 3.20. The SFRC slabs are more ductile compared to the slabs with rebar mesh for vertical displacements up to 3 mm. However, for the SFRC slabs the ultimate load and the peak crack load is the same, leading to a global softening for increased displacements. This means that for increased loading beyond the ultimate load, the slabs would experience a brittle failure. The load was applied with displacement control, after about 20 mm vertical displacement, all three slabs failed in a one-way brittle flexural failure.

---

**Figure 3.19:** Load – vertical displacement response for slab S15, S17 and S18 at peak load.
3.3. RESULTS, SLABS WITH ONLY SFRC

![Graph showing load-vertical displacement response for slab S15, S17 and S18 until failure.](image)

Figure 3.20: Load – vertical displacement response for slab S15, S17 and S18 until failure.

The instrumentation and crack pattern is presented for the three slabs in Figure 3.21, Figure 3.23 and Figure 3.25. The crack width vs. load and crack width vs. midpoint displacement is shown in Figure 3.22, Figure 3.24 and Figure 3.26.

All three slabs showed similar results. The crack width at peak load is about 0.2 mm but is increased to about 0.6 mm before any significant loss of load capacity. At the crack width of 0.2 mm, the vertical midpoint displacement is 1.3 mm, 1.8 mm and 2.0 mm for slab S15, S17 and S18 respectively. The crack width then increased near linear with the vertical displacement, showing about 10 mm crack width at a vertical displacement of about 20 mm, before failure.
Slab S15

For slab S15, the crack formed through sensor $p_{12}$ and $p_{16}$. The loading was then stopped and sensor $p_{14}$ reinstalled over the crack. The crack-width $w_{14} = 2.5$ mm was measured.

Figure 3.21: Slab S15 after failure.

Figure 3.22: Crack-width, load and displacement for sensor $p_{12}$ and $p_{16}$ of slab S15.
Slab S17

For slab S17 the crack formed through sensor $p_{14}$ and $p_{16}$.

Figure 3.23: Slab S17 after failure.

Figure 3.24: Crack-width, load and displacement for sensor $p_{14}$ and $p_{16}$ of slab S17.
Slab S18

For slab S18 the crack formed through sensor $p_{12}$, $p_{14}$ and $p_{16}$.

![Slab S18 after failure.](image1)

![Crack-width, load and displacement for sensor $p_{12}$, $p_{14}$ and $p_{16}$ of slab S18.](image2)
3.4 FE-models

3.4.1 Geometry and models

Similar as for the beam tests, a set of FE-models have been analysed to simulate the results from the static load tests of the slabs. The primary model consists of 4-noded shell elements, two different mesh sizes are shown in Figure 3.27. The rebar mesh is included as an orthotropic layer, having elasto-plastic material with \( f_y = 500 \) MPa and \( E_s = 200 \) GPa. The rebar cross and M20 bars are modelled with beam elements and are assigned the same material properties as the rebar mesh. The load is applied using displacement control in a single point, but is distributed to a surface of about 0.2×0.2 m, depending on the mesh size.

The position of the rebar mesh is referred to the concrete cover \( h_c \), measured from the bottom surface of the slabs. The target concrete cover was 60 mm, but measurements of slab S4 – S10 revealed that the cover was in average 30 mm (min 28 mm, max 40 mm). In the reference FE-model, \( h_c = 30 \) mm is therefore used.

![Figure 3.27: The 3D FE-model, a) reference model with dL=70 mm, b) coarse mesh, dL = 200 mm. Red dots indicate result points.](attachment:image.png)
3.4.2 Results, slabs with rebar mesh

The results at cracking using the reference model is shown in Figure 3.28, the gray lines corresponds to the experimental results from slab S1-S3. The complete response until failure is shown in Figure 3.29. Similar to the experiments, the ultimate load is in the same range as the cracking load. The reason is that the rebar mesh is located near the compressive zone and the load capacity is therefore mainly governed by the tensile strength.

When calibrating the FE-models, the tensile strength was reduced to 40% of the average values from the corresponding concrete class, in order to fit the experiments. The same factor was also obtained when using an FE-model with solid elements.

![Figure 3.28](load-displacement.png)  
Figure 3.28: Load-displacement predicted by the reference FE-model, at cracking.

![Figure 3.29](results.png)  
Figure 3.29: Results from the 3D FE-model with rebar mesh, influence of the concrete class.
The principal tensile strain in the FE-model is shown in Figure 3.30. Similar to the experiments, the first and the second crack does not appear at the same load level. This is obtained in the FE-model by introducing a random variation in the thickness for each element.

Figure 3.30: Principal tensile strain at the top surface at a) 1.5 mm deflection, b) 2.5 mm deflection, the reference model with concrete class C50.

The influence of the concrete cover is illustrated in Figure 3.31 and Figure 3.32. The load at cracking is mainly governed by the tensile strength and is not influence significantly by the reinforcement.

Figure 3.31: Results at cracking, reference model with concrete class C50.
The ultimate load is however significantly increased for higher concrete covers. For $t_b = 60$ mm, the ultimate load is about 120 kN, compared to 60 kN for $t_b = 30$ mm.

![Graph showing load vs. depth](image)

Figure 3.32: Results until failure, reference model with concrete class C50.

A comparison of the mesh size is shown in Figure 3.33. The coarse mesh results in slightly lower crack load but about the same ultimate load.

![Graph showing mesh size comparison](image)

Figure 3.33: Comparison between the medium and coarse element mesh model, reference model with concrete C50 and $t_b = 30$ mm.

The slab is also modelled with solid elements, mainly to obtain a better prediction of the crack widths. The tensile strain of the solid model is shown in Figure 3.34. To obtain irregular crack pattern, a random scatter of the top surface nodes is introduced to the model. The crack pattern from the solid model is compared to the obtained cracks from slab S1 in Figure 3.35.
Figure 3.34: Principal tensile strain at cracking, the FE-model with solid elements.

Figure 3.35: Principal tensile strain from the solid model and obtained cracks from slab S1.

The response from the solid model is compared to the shell model and the experiments in Figure 3.36 during cracking. The shell model and solid model gives near identical results and are in good agreement with the experiments.

The crack width vs. load and crack width vs. vertical midpoint displacement is presented in Figure 3.37. The crack widths from the shell model is referred to the slab midsurface which will underestimate the results. The corresponding results from the solid model is referred to the top surface. For crack widths less than 1 mm, the shell model still gives results in the same range as the experiments, at increased displacements, the crack width is however underestimated. The solid model is able to accurately predict the position of increased cracking, occurring at about 2.5 mm vertical displacement.
Figure 3.36: Load – displacement during cracking, compare experiments (gray) with reference shell model (blue) and solid model (red).

Figure 3.37: Crack opening $w$, vertical displacement $d$ and load $F$, compare experiments (gray) with reference shell model (blue) and solid model (red).

3.4.3 Results, slabs with only SRFC

The FE-models are modified for analysis of the slabs with only SRFC. The orthotropic layer for the rebar mesh is removed, the tension softening is modified and the fracture energy is increased. The results for a model with different fracture energy is presented in Figure 3.38. Similar to the beam tests, best fit with the experiments is obtained using $8G_f$. The model is able to accurately describe the ultimate load, but slightly underestimates the ductility at moderate displacements. At larger displacements, the ductility is instead slightly overestimated. Further calibration may result in a better fit.
The influence of the concrete quality class is shown in Figure 3.39. Similar to the beam tests, this primary influence the ultimate load and not the ductility at large displacements.

The results from a shell model and a solid model is shown in Figure 3.40. Similar to the slabs with rebar mesh, both models give near identical results in load-displacement response. The crack width is better estimated with the solid model, showing good agreement with experiments, Figure 3.41. Better results may be obtained with the shell model if the crack width from the mid surface to the top surface is accounted for.
Figure 3.40: Load – displacement during cracking, compare experiments (gray) with reference shell model (blue) and solid model (red).

Figure 3.41: Crack opening \( w \), vertical displacement \( d \) and load \( F \), compare experiments (gray) with reference shell model (blue) and solid model (red).
Chapter 4

Slabs subjected to impact load

4.1 Test setup

Dynamic impact loading was performed on slab S4 – S10 and S12 – S14. The three last slabs were manufactured with an outer layer of steel fibre reinforced concrete, the remaining were manufactured similar to slab S1 – S3.

The impact loading consisted of a 600 kg steel mass, dropped from different heights \( h \), Figure 4.2. The mass was stabilised with vertical guide rails, shown in Figure 4.1. The same steel slab as from the static load tests was used in distributing the load from the mass onto the slab. The slab was suspended in the hangers that were connected to a frame by a one-way hinge. A complete view of the frame is shown in Figure 4.4. Each hanger was instrumented with a load cell. In addition, an accelerometer \( a_1 \) was installed on the top of the mass and \( a_2 \) on the top of the slab, near the steel plate, shown in Figure 4.3 During the impact load tests, data was sampled with 19.6 kHz using an HBM QuantumX DAQ-system.

Figure 4.1: View of the setup, after impact loading.
CHAPTER 4. SLABS SUBJECTED TO IMPACT LOAD

Figure 4.2: Elevation of the slab, suspended in the hangers.

Figure 4.3: Detail of the mass, a) lower end showing the steel plate and the accelerometer on the slab, b) the upper surface of the mass, showing the accelerometer on the mass.
Figure 4.4: View of the complete frame, before load test.
4.2 Initial tests, multiple impacts

Slab S7 was tested with multiple impact tests of increasing height \( h \), starting from 0.08 m to 0.53 m. Table 4.1 presents the total peak load \( F_{\text{max}} \) in all hangers, the peak vertical displacement \( d_{\text{max}} \) of the slab and the impulse load \( I \). The vertical displacement is obtained by time integration of the accelerometer signals using Eq. (4.1), which for \( \gamma = 0.5 \) and \( \beta = 0.25 \) results in the trapezoidal rule (Clough & Penzien, 2003).

The impulse load is obtained by integrating the force over time while the mass is in contact with the slab, 0-\( t_1 \), Eq. (4.2).

\[
\dot{x}_i = \dot{x}_{i-1} + (1 - \gamma) \Delta t \ddot{x}_{i-1} + \gamma \Delta t \dddot{x}_i \\
x_i = x_{i-1} + \Delta t \dot{x}_{i-1} + (0.5 - \beta) \Delta t^2 \ddot{x}_{i-1} + \beta \Delta t^2 \dddot{x}_i \\
I = \int_0^h F \, dt
\]

(4.1)

(4.2)

Table 4.1: Summary of peak load, peak displacement and impulse load in slab S7.

<table>
<thead>
<tr>
<th>( h ) (m)</th>
<th>( F_{\text{max}} ) (kN)</th>
<th>( d_{\text{max}} ) (mm)</th>
<th>( I ) (kNs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.08</td>
<td>97</td>
<td>20</td>
<td>1.4</td>
</tr>
<tr>
<td>0.17</td>
<td>130</td>
<td>27</td>
<td>2.1</td>
</tr>
<tr>
<td>0.25</td>
<td>150</td>
<td>39</td>
<td>2.5</td>
</tr>
<tr>
<td>0.34</td>
<td>155</td>
<td>42</td>
<td>2.9</td>
</tr>
<tr>
<td>0.43</td>
<td>145</td>
<td>-</td>
<td>3.1</td>
</tr>
<tr>
<td>0.53</td>
<td>136</td>
<td>37</td>
<td>3.2</td>
</tr>
</tbody>
</table>

The cracking of slab S7 after the final impact test is shown in Figure 4.5, showing similarities with the static load tests S1 – S3. The deflection of the slab along the side of hanger F2 – F3 is shown in Figure 4.6. The propagation of the same crack is shown in Figure 4.7 for the fall heights 0.17 – 0.53 m. After the first impact from 0.08 m, the slab was uncracked. The response in the hangers during the different tests are shown in Figure 4.8.
4.2. INITIAL TESTS, MULTIPLE IMPACTS

Figure 4.5: Slab S7 after impact loading.

Figure 4.6: View of slab S7 after the impact tests according to Table 4.1.
Figure 4.7: Main crack at the edge of slab 7, along the side of hanger F1 – F4, after impact, a) to e) corresponds to fall height 0.17 m to 0.53 m.
Figure 4.8: total load in hanger F1-F4 during impact of the 600 kg steel mass.
4.3 Results, single impact

The results from the single impact tests are shown in Figure 4.9 to Figure 4.26.

Figure 4.9: Slab S9 after impact loading, h = 1.2 m.

Figure 4.10: Total load in the hangers, slab S9 with h = 1.2 m.
4.3. RESULTS, SINGLE IMPACT

Figure 4.11: Slab S4 after impact loading, h = 1.0 m.

Figure 4.12: Total load in the hangers, slab S4 with h = 1.0 m.
Figure 4.13: Slab S8 after impact loading, $h = 1.0$ m.

Figure 4.14: Total load in the hangers, slab S8 with $h = 1.0$ m.
Figure 4.15: Slab S12 after impact loading, $h = 1.0$ m.

Figure 4.16: Total load in the hangers, slab S12 with $h = 1.0$ m.
CHAPTER 4. SLABS SUBJECTED TO IMPACT LOAD

Figure 4.17: Slab S5 after impact loading, h = 1.5 m. 16 kg of fallout.

Figure 4.18: Total load in the hangers, slab S5 with h = 1.5 m.
Figure 4.19: Slab S6 after impact loading, h = 1.5 m. 0.9 kg of fallout.

Figure 4.20: Total load in the hangers, slab S6 with h = 1.5 m.
Figure 4.21: Slab S14 after impact loading, $h = 1.5$ m. 0.3 kg of fallout.

Figure 4.22: Total load in the hangers, slab S14 with $h = 1.5$ m.
4.3. RESULTS, SINGLE IMPACT

Figure 4.23: Slab S10 after impact loading, h = 2.0 m. 6.6 kg of fallout.

Figure 4.24: Total load in the hangers, slab S10 with h = 2.0 m.
Figure 4.25: Slab S13 after impact loading, \( h = 2.0 \) m. 0.3 kg of fallout.

Figure 4.26: Total load in the hangers, slab S13 with \( h = 2.0 \) m.
The hanger loads from the different impact tests are summarised in Figure 4.27 and Table 4.2.

Figure 4.27: Total load in the hangers, comparison of different fall heights. Slab S12, S13 and S14 had a lower layer of SFRC.

Table 4.2: Summary of peak load, peak displacement and impulse load for the impact tests.

<table>
<thead>
<tr>
<th>Slab no.</th>
<th>h (m)</th>
<th>$F_{\text{max}}$ (kN)</th>
<th>$d_{\text{max}}$ (mm)</th>
<th>$I$ (kNs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>1.2</td>
<td>226</td>
<td>61</td>
<td>4.5</td>
</tr>
<tr>
<td>4</td>
<td>1.0</td>
<td>203</td>
<td>46</td>
<td>3.9</td>
</tr>
<tr>
<td>8</td>
<td>1.0</td>
<td>248</td>
<td>60</td>
<td>4.1</td>
</tr>
<tr>
<td>12</td>
<td>1.0</td>
<td>210</td>
<td>55</td>
<td>4.0</td>
</tr>
<tr>
<td>5</td>
<td>1.5</td>
<td>204</td>
<td>63</td>
<td>4.6</td>
</tr>
<tr>
<td>6</td>
<td>1.5</td>
<td>227</td>
<td>50</td>
<td>4.4</td>
</tr>
<tr>
<td>14</td>
<td>1.5</td>
<td>187</td>
<td>51</td>
<td>4.7</td>
</tr>
<tr>
<td>10</td>
<td>2.0</td>
<td>253</td>
<td>77</td>
<td>5.4</td>
</tr>
<tr>
<td>13</td>
<td>2.0</td>
<td>230</td>
<td>80</td>
<td>5.0</td>
</tr>
</tbody>
</table>
Figure 4.28 and Figure 4.29 shows the damage to slab S5 after impact, with a total fallout of 16 kg.

Figure 4.28: Photo of slab 5 after impact loading.

Figure 4.29: The concrete block that fell out from slab 5.
The tests showed that the peak hanger force was not significantly influenced by the free fall height, as would be expected for elastic conditions. The load is also much smaller than the ultimate load of the hangers. The reason is likely that increased height resulted in greater damage of the slab, resulting in less stiffness and larger displacements. Figure 4.30 shows a clear correlation between the height and impulse load but not between the height and peak load.

![Figure 4.30: Correlation between the free fall height $h$, impulse load $I$ and peak load $F_{\text{max}}$.](image)

For slab S5, S6, S9 and S10, the permanent deformation after impact (without the mass) was measured in a grid of 3×3 positions using a total station. The midpoint displacement and largest corner uplift is presented in Table 4.3. The displacement of slab S10 is larger than presented in Table 4.2, possibly owing to uncertainties of the time integration of the accelerometers.

<table>
<thead>
<tr>
<th>slab no.</th>
<th>$h$ (m)</th>
<th>$d_{\text{mid}}$ (mm)</th>
<th>$d_{\text{corn}}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>1.2</td>
<td>38</td>
<td>-18</td>
</tr>
<tr>
<td>5</td>
<td>1.5</td>
<td>62</td>
<td>-39</td>
</tr>
<tr>
<td>6</td>
<td>1.5</td>
<td>79</td>
<td>-34</td>
</tr>
<tr>
<td>10</td>
<td>2.0</td>
<td>127</td>
<td>-60</td>
</tr>
</tbody>
</table>
4.4 FE-models

4.4.1 Geometry and models

The FE-models used for the static simulations have been modified to simulate the impact load. The reference model is shown in Figure 4.31. The frame is included in the model since the response of the slab during impact is governed by the total flexibility.

The mass is lumped in a single node and is therefore assumed rigid. The node is connected to a non-linear truss element that behaved ideal elasto-plastic in compression but has no stiffness in tension. The lower end of the truss element is connected to the slab, that is distributed to the load area of about 0.2×0.2 m via vertical constraints, Figure 4.32. To avoid instability of the system, the node of the mass is only allowed to move in the vertical direction. The permanent load is applied in a static step before impact, and the impact load is applied as an initial velocity $v$.

The tri-axial concrete model is used, to account for the progressive damage during cyclic loading. The analysis is performed using an explicit time integration scheme.

Figure 4.31: FE-model of the slab and the frame during impact loading.

Figure 4.32: Concept of applying the load from the mass to the slab, using a non-linear truss element and a lumped nodal mass.
4.4.2 Results

The peak load vs. free fall height is presented in Figure 4.33. Similar to the experiments, the FE-model predicts similar peak load in the hangers within the height 1 – 2 m. The FE-model is also in good agreement with the initial tests of slab S7, although each point from the FE-model corresponds to a single impact. The experiments of slab S7 shows that the peak load decreases as the slab suffers more cracking, likely due to the reduced global stiffness.

The peak vertical midpoint displacement of the slab is presented in Figure 4.34. The FE-model shows reasonably good agreement compared to the single impact tests. Larger displacements are generally found for slab S7, probably due to the multi-impact tests where the slab suffered increased damage and decreased stiffness. It should also be noted that the experimental displacements were time integrated from the accelerometers and are afflicted with some uncertainties.
An example of the time history response is presented in Figure 4.35, for the case of 
$h = 1.0$ m. The FE-model shows good agreement with the experimental tests, not only 
in peak reaction force and peak displacement, but also during the following response.

**Figure 4.35:** Response in the slab due to the mass of 600 kg from 1.0 m free fall 
height, a) total reaction in the hangers, b) vertical displacement of the slab.
Chapter 5

Simulation of inner lining systems

5.1 The system

The inner lining system in tunnels is a method to prevent water ingress and forming of ice in the traffic area. A solution that is common in Norway is based on stretching a sealing membrane between rock anchorages that forms a gap to the primary rock strengthening. The membrane is in turn protected by a layer of shotcrete towards the traffic area. The rock reinforcement is designed to resist all loads from the rock mass independent of the inner lining system.

For the Norra länken tunnel project, a similar inner lining system was adopted, Figure 5.1. The membrane was mounted a minimum of 100 mm from the rock surface and was provided with a 100 mm thick shotcrete layer that formed the inside of the traffic area. The same concept has also been used in parts of Citybanan and is planned to be used for the Stockholm Bypass project.

Figure 5.1: Inner lining system in Norra Länken, a) photo by T. Dalmalm, b) sketch of the inner lining system.
5.2 FE-model

An FE-model of the inner lining system is shown in Figure 5.2. The arch is modelled with 4-noded shell elements, the span $B = 14$ m and the height $h = 2.3$ m, according to preliminary drawings of the Stockholm Bypass project. A section $L = 30$ m of the tunnel is studied, whereof a section of $6$ m is modelled with a denser mesh. The thickness is 120 mm and a rebar mesh d16#100 is included as an orthotropic layer in the shell elements, a concrete cover $t_b = 40$ mm is used, measured from the top surface. The abutments of the arch is assumed pinned.

Rock bolts with d16#1200 and a length of 1.5 m are modelled with beam elements that accounts for yielding and buckling. The bolts are assumed clamped at the connection with the rock.

The shotcrete is modelled with the tri-axial material model. The material strength corresponds to the average values of concrete class C40. The rebar mesh and the rock bolts are assigned a yield strength $f_y = 500$ MPa.

![Figure 5.2: View of the global 3D FE-model of the inner lining system.](image_url)
5.3 Static load capacity

The static flexural capacity of the arch is studied for a load acting on a surface of 0.2×0.2 m, similar to the slab tests. The load-displacement response is presented in Figure 5.3. The ultimate tensile load of a single rock bolt is 100 kN, at the peak load of 500 kN, the four rock bolts closest to the load are near yielding. The remaining load is taken by the adjacent bolts and a small portion is transmitted down to the support of the arch.

The principal tensile strain in the arch near the loading is shown in Figure 5.4. The resolution of the model is not sufficiently high to distinguish separate cracks, but the model clearly shows localised strain under the load and within an area of about 2×2 m. The deformation of the arch is also rather localised. It should be mentioned that the model is not able to account for punching failure, and the results may be on the unsafe side. Further, average value of the concrete and steel strength have been used.

Figure 5.3: Load-displacement due to a local load at the crown.

Figure 5.4: Segment of the arch at static ultimate load, a) principal tensile strains at the top surface, b) principal tensile strains at the bottom surface, c) vertical displacement.
### 5.4 Impact loading

The impact load is modelled similar to the slab tests, assuming the mass in a single load, transferred to the arch by a non-linear truss element. The response from a mass of 600 kg from a height of 1.0 m is shown in Figure 5.5.

The total reaction force at impact is about 500 kN, similar to the static load capacity in Figure 5.3. During the first downward movement of the arch, yielding occurs in the rock bolts closest to the point of impact. During the following upward movement, buckling of rock bolts occur, resulting in redistribution of the load to the adjacent bolts. The peak displacement is 12 mm, occurring after 7 ms. The mass loses contact with the arch after 16 ms and is then moving upwards. The second impact of the mass occurs after about 0.3 s. The largest reaction force in the system is about 1000 kN and is obtained when the mass has lost contact with the arch. Based on a duration of impact of 16 ms, an impulse load of 6.0 kNs is calculated.

**Figure 5.5:** Response of the inner lining system due to a mass of 600 kg with a free fall height of 1 m, a) total reaction force in the hangers and the arch supports, b) vertical displacement of the concrete arch and the mass at the point of impact.

The principal tensile strains are shown in Figure 5.6, at peak vertical displacement of the arch, e.g. 7 ms after impact. The results show similarities with the corresponding static response in Figure 5.4, with the difference that the cracking of the top surface is more extensive and that the vertical displacement is slightly larger. It should again be stressed that the model does not account for punching failure and may show results on the unsafe side.
Figure 5.6: Segment of the inner lining system during impact (at max vertical displacement), a) principal tensile strain at the top surface, b) principal tensile strain at the bottom surface, c) vertical displacement.
Chapter 6

Discussion and conclusion

6.1 Experimental results

Based on core samples and four point flexural beam tests, the shotcrete corresponded to the average values of concrete class C40 – C50. The beam tests with plain concrete showed a brittle failure, whereas the fracture energy of the beams with SRFC was significantly larger.

The concrete cover of the slabs were in average 30 mm, measured from the compressive side, instead of the intended 60 mm. This resulted in a small difference between the crack load and the ultimate load in the static load tests. This may also have been an contributing factor to the large amount of fallout during the impact tests. The position of the reinforcement did however provide a significant ductility and no case of reinforcement failure was observed. This may also have been beneficial during the impact load tests, both in avoiding reinforcement failure but also in extended cracking and energy absorption. No tests with other concrete covers were however performed, so no definite conclusions can be drawn. The tests with an outer layer of SRFC was shown to efficiently prevent fallouts. The results from the slab tests can not directly be translated to the inner lining system, since the load distribution and boundary conditions are different.

6.2 Numerical models

The nonlinear FE-models were shown to be in good agreement with the experimental testing, both for the beams and the slabs. Some modifications to the tensile strength were however required to obtain a best fit with the concrete slabs, possibly due to shortcomings of the model. The FE-model was also able to predict the failure mode and crack widths with a fair accuracy, at least when comparing with the static slab tests. The crack widths were underestimated using the shell model, since the results referred to the midsurface of the slab.

The simulation of impact loads showed good agreement in both peak load and peak vertical displacement of the slab. Both the models and the experiments showed that there were no clear correlation between the free fall height and the peak reaction force.
in the hangers. The reason is likely that a larger impact results in a greater reduction of stiffness and larger displacements. The impulse load was however shown to correlate with the free fall height.

Simulations of the inner lining system showed a significantly larger load capacity than the concrete slabs, even though the load distribution was not significantly larger. It should however be stressed that the models does not account for punching failure. Further, all results presented in this report referrers to the average material strength, safety factors need to be applied even for accidental loads.

6.3 Further research

The experimental part of this report is limited to scaled down slab tests. Furthermore, the reinforcement was located near the compressive zone, resulting in a low utilisation. Slabs with the reinforcement closer to the surface of cracking should show a significantly larger load capacity, but may result in a more brittle failure, due to reinforcement rupture and increased stiffness, especially during impact loading.

Full-scale tests of the inner lining system would give valuable knowledge of the whole structure and could be used to verify the simulations.

The FE-models can be further developed, mainly to include punching failure. This should be combined with experimental tests where punching failure is likely to occur.
Bibliography


