Load capacity of inner lining systems due to impact from falling rocks

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1. Introduction

Inner lining systems in tunnels is a method to prevent water ingress in the traffic area. The method is commonly used in countries with cold climate and is often combined with a conventional near-rock strengthening. Although many established inner lining systems exist, challenges still remain to find the optimal solution considering function, maintenance and investment cost. In e.g. Norway, some 20 different systems have been developed during the last 50 years, but have since the 1990ies focused on systems with a combination of prefabricated concrete elements and shotcrete (Broch et al., 2002). A comparison between the Norwegian and the Swiss system is presented in (Ramoni & Matter, 2013). In the Norwegian system, the inner lining is suspended in rock bolts with a gap to the rock surface, whereas in the Swiss system the inner lining is applied in direct contact with the near-rock strengthening. It is concluded that the latter has more advantages considering reliability, accessibility, maintenance, service life, safety and cost. A recent example of the Swiss system was used in the Gotthard Base tunnel, consisting of double layers of shotcrete separated by a sealing membrane to prevent ingress of water and to decrease the ground water pressure on the outer layer (Stadelmann et al., 2007).

The Norwegian system was first used in Sweden for the Northern Link project in Stockholm. It consists of a sealing membrane that is suspended in densely spaced rock bolts, see Fig. 1. The sealing membrane was used since it was believed that rock grouting alone would not be sufficient to prevent water ingress in the traffic area. The membrane was mounted at least 100 mm from the rock profile and was covered with a 100 mm thick layer of shotcrete towards the traffic area, to withstand the loads from e.g. pressure and suction (NVF, 2008).

![Fig. 1 Inner lining system for the Northern Link project, a) photo by T. Dalmalm, b) sketch of the inner lining system.](image-url)
One of the concerns with the system is how to perform inspections and conditional assessments of the primary rock strengthening. One concept of inspection method is based on identifying cracks in the inner lining due to potential movements of the primary rock strengthening. Calculations presented by (Fredriksson, 2006) shows the relation between the displacement of a rock bolt and the risk of cracking of the inner lining system. The extent of cracking depends on the thickness of the primary strengthening, the thickness of the inner lining and the spacing and length of the rock bolts.

A similar inner lining system was used in parts of the recent Stockholm City Line project. Similar calculations regarding the relation between displacements of individual rock bolts and cracking of the inner lining is presented by (Rosengren, 2011a). For slender rock bolts, buckling may occur before cracking of the inner lining system. Assuming 120 mm of shotcrete and 16 mm diameter of the rock bolts, the maximum length of 0.5 m for the rock bolts was obtained to cause cracking of the inner lining. In (Rosengren, 2011b), the load carrying capacity of the primary rock strengthening is studied, for an assumed pyramid shaped rock between the rock bolts. It is concluded that the composite action with the rock stiffness is necessary to withstand this block under the assumed conditions.

The function, methods of inspection and potential load scenarios for the inner lining system is further discussed by (Taube & Olsson, 2013). Cracking of the inner lining as an indirect method for inspection of the primary strengthening is discussed, e.g. due to displacements of the rock bolts or from falling blocks of rock.

A similar inner lining system is planned to be used for the Stockholm Bypass project, an 18 km long road tunnel. One of the questions is what loads the inner lining system should be designed for, especially considering falling rocks. According to the Swedish technical requirements for tunnels (TRVK Tunnel 11) it is stated that a near rock strengthening or an inner lining system shall be designed for a single falling rock of 6 kN acting on a surface of 0.2×0.2 m if the inner lining is not located near the rock surface. It is further stated that if the inner lining prevents inspection of the primary rock strengthening, the inner lining shall be designed for a block load of 60 kN, acting on a surface of 1×1 m. The first load is interpreted as an impact load and the second as an equivalent static load. For the case of the Northern Link tunnel project, it was reasoned that a single falling block was not a relevant load case, since it would be prevented by the primary rock strengthening (Vägverket, 2006). It can be argued that the condition of the primary rock strengthening is not assessable by inspections.

With the aim of understanding the behaviour of inner lining systems due to impact loads, an experimental scheme was conducted within a research project. Part of the results are presented in this paper, the complete study is presented in (Andersson, 2014). The methodology involves experimental testing of a series of concrete slabs, validation of numerical models and simulation of the response of the complete inner lining system.

2. Manufacturing of the concrete slabs

A total of 18 slabs were manufactured according to the specifications in Fig. 2. A rebar mesh was used in 13 of the slabs and an outer layer of 30 mm of steel fibre reinforced concrete (SRFC) was used for three of these. The main reason for the SRFC-layer was to compare the amount of spalling of concrete underneath the slabs during impact loading. Four slabs were sprayed with only SRFC and no rebar mesh, these were only tested for static failure. The last slab was sprayed with only plain shotcrete, from which beam- and core samples were taken for reference testing. The slab supports consisted of four M20 threaded bars spaced 1.2 m, corresponding to the rock bolts of the inner lining system. The bars were intentionally made stronger than the in-situ case to assure failure of the slab before the bolts.
The slabs were sprayed in vertical position, see Fig. 3. The rebar mesh was positioned centric in the cross section and fixed to the M20 bars by rebar crosses. Due to the pressure during spraying, the rebar mesh was however pushed towards the formwork and measurements revealed a concrete cover ranging from 35 – 40 mm, measured from the surface of the formwork. Since this corresponds to the compressive zone during testing, a lower load capacity would be expected.

The slabs were subjected to 7 days of water curing and kept in the formwork for 28 days. The experimental tests started 84 days after casting.

![Fig. 2 Specifications for the concrete slabs.](image1)

![Fig. 3 Spraying of the concrete slabs, photo by A. Andersson.](image2)

### 3. Static load tests

Three of the slabs with rebar mesh were tested to static failure. The slabs were mounted upside down and the load was applied centric using a hydraulic pump, see Fig. 4. The support bars were fastened to rigid points of a laboratory floor. The slabs were instrumented with a total of 17 displacement gauges (LVDT) and one load cell in between the slab and the hydraulic pump. Sensor 1-4 measured vertical displacement of the support points, to account for the elasticity of the bars. Sensor 5-8 measured the vertical displacement at the slab quarter points and sensor 9 at the slab midpoint. The crack widths were measured with sensor 10-17. For slab S2, sensor 11 and 15 was repositioned after the crack was detected at an adjacent location.
The applied load $F$ versus the vertical midpoint displacement $d$ is presented in Fig. 5. The experimental crack load ranged from $47 - 59 \text{kN}$ occurring at a vertical displacement of $0.6 - 1.1 \text{mm}$, after subtracting the support displacements. The ultimate load ranged from $58 - 82 \text{kN}$ at a displacement of $70 - 80 \text{mm}$. The variation in ultimate load mainly depends on the position of the rebar mesh. Since the reinforcement is located near the compressive zone, the difference between cracking load and ultimate load is small. All three slabs showed a two-way flexural failure.

The response of the slabs were also simulated using an FE-model that accounts for the non-linear behaviour of concrete and large displacements. The model is further described in (Andersson, 2014). The results from the FE-model in Fig. 5 illustrate the influence of the concrete cover $t_b$, measured from the compressive side of the slab. Using $t_b = 30 \text{mm}$ results in good agreement with the experiments, but with $t_b = 60 \text{mm}$ the ultimate load is increased by a factor 2. The crack load mainly depends on the tensile strength of the concrete and is not influenced significantly by the concrete cover.

![Fig. 5](image-url)  
**Fig. 5** Load-displacement response during static load tests, comparison between experiments and FE-models. The concrete cover $t_b$ is measured from the compressive side of the slab.
The relation between the applied load $F$, the crack width $w$ and the midpoint displacement $d$ is illustrated in Fig. 6. The experiments show that it takes a vertical displacement of about 2 mm to obtain a crack width of 0.2 mm. The first crack occurred almost simultaneously along sensor 10-14 and 12-16. The second crack along the same direction, see Fig. 4, formed at larger displacements. A crack also formed in the perpendicular direction, registered by sensor 13. The parallel crack measured by sensor 17 developed at larger displacements. The corresponding cracks at the opposite side of the slab initially propagated adjacent to sensor 11 and 15 which were re-installed after the cracks were detected. Good agreement is found between the experiments and the FE-model, both regarding displacements, crack widths and load. Further results are found in (Andersson, 2014).

**Fig. 6** Relation between the load $F$, crack width $w$ and vertical midpoint displacement $d$, comparison between experimental results from slab S2 (solid lines) and the FE-model (dashed lines).

### 4. Impact load tests

Impact load tests were performed on ten of the slabs, whereof three with an outer layer of SFRC. The slabs were suspended in a steel frame, partially seen in Fig. 7a. The columns were anchored to the same floor as used for the static load tests and post-tensioned using dywidag-bars. This prevented any potential uplift due to inertia forces during impact. Load cells were installed at the top end of each hanger and connected to the above steel beams by a hinge. The free length of the hangers were 1.15 m, using similar M20 bars as for the static load tests. The load consisted of a 600 kg steel weight, lifted to the prescribed height using an overhead crane and released with a tailor-made release lock device.

**Fig. 7** After impact of 600 kg from 1.5 m, a) the mass and the slab mounted on the frame, after impact (photo by A. Andersson), b) cracking and zones of spalling at the bottom of the slab.
The result of an impact test from 1.5 m is shown in Fig. 7. The slab showed a primary one-way flexural failure, but with radial secondary cracks. For the particular case, a total of 16 kg of concrete fell out from underneath the slab. The remaining slabs were tested at heights from 1 – 2 m, with various amount of concrete spalling. The reason for the large amount of spalling was likely due to the large concrete cover. Neither of the three slabs with an outer layer of SRFC showed any significant spalling, despite a free fall height up to 2 m. Hence, the SRFC was shown to efficiently prevent spalling due to impact loading, even in the case of large concrete cover.

The same FE-model as used for the static load tests were modified to simulate the impact load tests. The mass was lumped to a single node point and connected to the slab using a non-linear truss element with no tensile stiffness, to enable loss of contact between the mass and the slab. The time history of the total hanger reaction force is shown in Fig. 8, for the case of 1 m free fall height. The peak load is about 200 kN, more than three times the static load carrying capacity. The peak vertical displacement of the slab was about 50 mm and was calculated based on time integration of an accelerometer mounted on the slab near the point of impact.

Results at different free fall heights are shown in Fig. 9. No clear correlation between the height and the peak load could be found, neither from the experiments nor the FE-model, despite a significant increase in momentum of the impact mass. The reason is likely due to increased damage of the slabs, resulting in a reduced stiffness and increased displacements, as seen in Fig. 9b. This also resulted in a longer duration of the impact force. The impulse load, defined as the integral of force over time during impact, was calculated to 3.9 – 4.1 kNs for 1 m free fall height and 5.0 – 5.4 kNs for 2 m free fall height.

Fig. 8 Response of the slab during impact load of 600 kg from 1.0 m, a) total reaction force in the hangers, b) vertical displacement of the slab midpoint. Comparison between three experimental tests and the FE-model.

Fig. 9 Comparison between the FE-model and experiments from impact loading, a) peak reaction force in all hangers, b) peak vertical displacement of the slab.
The influence of damage to the slab was also studied by performing multiple impact tests on a single slab, illustrated with $\times$ in Fig. 9. The height was increased from an initial 0.08 m to 0.54 m, the first crack occurred after the second test at a height of 0.17 m. The last two impacts showed a lower peak reaction force in the hangers, due to the reduced stiffness of the slab. It is also concluded that the FE-model is sufficiently accurate in predicting both the peak load and the corresponding displacement.

5. Simulation of the inner lining system

The FE-model that was used for the slab tests was extended to describe the whole inner lining system, illustrated in Fig. 10. The concrete arch is modelled with 4-noded shell elements, where the rebar mesh in modelled as an orthotropic layer with elasto-plastic material properties. A triaxial plasticity based concrete model is employed, using a smeared crack approach with orthogonal fixed crack formulation and strain softening in both tension and compression. The concrete is assigned properties corresponding to the average values of concrete class C40. The thickness of the arch is assumed to 120 mm and the rebar mesh d16#100 is located with a concrete cover $t_b = 40$ mm, measured from the top surface.

The rock bolts d16#1.2m, $L=1.5$ m are modelled with beam elements that accounts for yielding and buckling. The bolts are assumed clamped at the upper node corresponding to the rock. The abutments of the arch is assumed pinned along the whole length of 30 m. The studied load case is a single load at the midpoint of the arch crown. The element mesh size at this location is 0.2 m.

![Fig. 10 The FE-model of the inner lining system.](image)

The simulations show a static load capacity of 500 kN, resulting in a flexural failure. The tensile capacity of one rock bolt is 100 kN and the results show that the four bolts adjacent to the load point is near yielding. The remaining 100 kN is distributed to nearby rock bolts and partially down to the abutment of the arch. It should be mentioned that the present FE-model is not able to accurately describe potential punching failure and that the results may therefore be non-conservative. It should also be stresses that the results are based on the average material strength parameters, without any safety factors.

The tensile principal strains and vertical displacements at the vicinity of the loaded are shown in Fig. 11. The results from the static simulation is shown in Fig. 11a,b,e. The resolution of the model is not sufficient to distinguish between individual cracks, but clearly shows localised strain under the load and within an area of about 2×2 m. Also the deformation of the arch is rather localised.
The same model is used to simulate the impact of a 600 kg mass falling from 1 m. The corresponding principal tensile strain and vertical displacements are shown in Fig 11c,d,f at the point of maximum displacement. The amount of cracking is more extensive than the corresponding static test. The vertical peak displacement is 12 mm, 3 – 5 times smaller than observed from the experiments of the slabs. The reason is that the load distribution is larger and that the boundaries of the studied segment is constrained due to the continuity of the arch.

The time history of the reaction force in the model is presented in Fig. 12. The total reaction force at impact is about 500 kN, similar to the static load capacity of the model. 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 the 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 after that moving along a parabola. The second impact of the mass occurs after about 0.3 s, not shown in the figure. 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.

**Fig. 11 Segment of the FE-model in the region of loading, a) principal tensile strain at the top surface at static failure load, b) similar at the bottom surface, c) principal tensile strain at the top surface at peak impact load of 600 kg from 1.0 m, d) similar at the bottom surface, e) displacement during static failure load, f) peak displacement during impact load.**

**Fig. 12 Response of the inner lining system during impact of a 600 kg mass from 1.0 m, a) total reaction force in the system, b) vertical displacement of the arch at the position of impact. Results from the FE-model.**
6. Conclusions

Increased knowledge of static and dynamic behaviour of inner lining systems have been obtained by a combination of downscaled experiments and numerical simulations. The FE-models showed good agreement compared to the experiments and were therefore used in simulating the response of the whole inner lining system.

The experimental results from the static slab tests showed a cracking load of 5 – 6 tonnes with a vertical deflection of 0.6 – 1.1 mm. A deflection of about 2 mm was required to obtain a crack width of 0.2 mm. The failure load ranged from 6 – 8 tonnes with a vertical displacement of 70 – 80 mm, hence proving a significant ductility. The reinforcement was located near the compressive side, resulting in a moderate increase in ultimate load capacity compared to the cracking load. The simulations indicate that if the reinforcement would have been placed centric along the thickness, the ultimate load capacity would increase by a factor 2.

A total of ten slabs were tested for impact load using a 600 kg steel mass that were dropped from height varying from 1 – 2 m. All slabs were dominated by a one-way flexural failure, with secondary radial cracks. Significant spalling occurred, at most corresponding to a concrete mass of 16 kg. A contributing factor may have been the large concrete cover. For three of the slabs an outer layer of 30 mm SRFC was applied, facing downwards during the impact tests. None of these slabs showed any significant spalling, proving the SRFC to be efficient in confining the concrete. No clear correlation between peak reaction force during impact and the free fall height was found, neither from the experiments nor the simulations. The reason is likely due to increased damage during increased momentum of the mass, resulting in decreased stiffness and increased displacements. Multiple impact tests on a single slab showed lower peak reaction force at greater initial damage of the slab. A better measure is the impact force, estimated to 3.9 – 4.1 kNs for a free fall height of 1 m and 5.0 – 5.4 kNs at 2 m.

Simulations of the complete inner lining system indicate a significant load capacity. The static flexural capacity was estimated to 50 tonnes, whereof 40 tonnes were transmitted to the four nearby hangers. The corresponding vertical displacement of the arch was about 10 mm. The impact of a 600 kg weight from 1 m resulted in a peak reaction force of about 100 tonnes. At the first downward movement yielding occurred in the four nearby hangers; at the following upward movement the hangers buckled and the reaction force was redistributed to nearby hangers. The model showed a similar flexural failure as from the slab tests, but with a slightly larger load distribution. The vertical displacement of the inner lining system was significantly lower compared to the slab tests, due to the arch continuity acting as restraints.

It should be stressed that the presented FE-model does not account for punching failure or spalling, and may be non-conservative in estimating the real load carrying capacity of the inner lining system. Furthermore, the average material strength parameters obtained from experimental tests were used, without any safety factors. In further studies, the model should be extended to include punching failure and spalling, as well as the influence of safety factors for accidental loading. Such a model should be verified against relevant experimental testing to assure its validity.
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8. References


