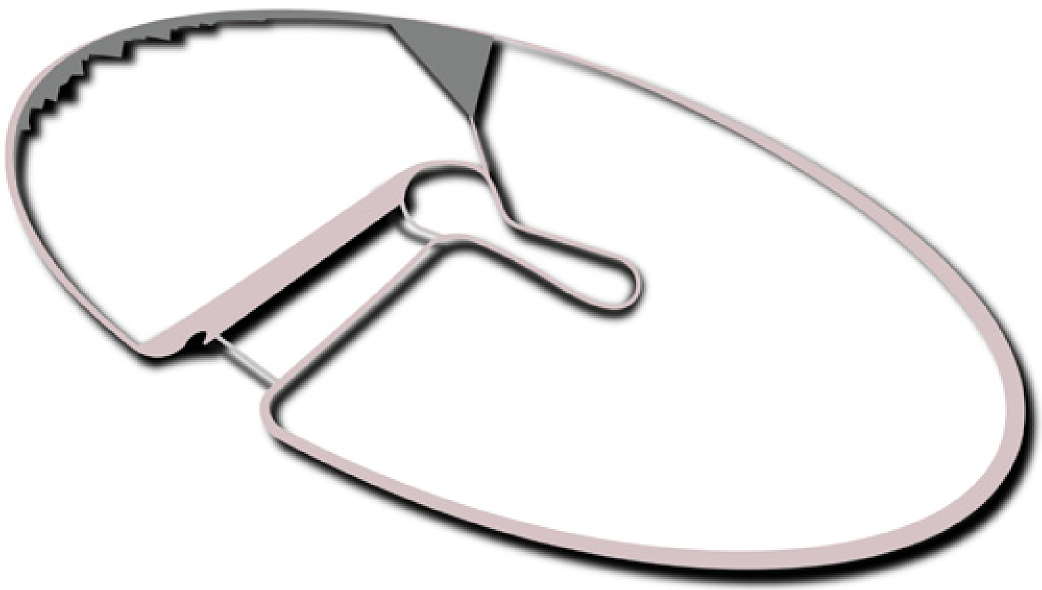
PROCEEDINGS

# Ninth International Symposium on SPRAYED CONCRETE

## - Modern Use of Wet Mix Sprayed Concrete for Underground Support



Sandefjord, Norway 17.-20. June 2024

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norwegian concrete association

9TH INTERNATIONAL SYMPOSIUM ON

# SPRAYED CONCRETE

## MODERN USE OF WET MIX SPRAYED CONCRETE FOR UNDERGROUND SUPPORT

**SANDEFJORD, NORWAY 17. – 20. June 2024**

**Edited by**
**Øyvind Bjøntegaard**
**Thomas Beck**
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**9th International Symposium on - SPRAYED CONCRETE**
**- Modern Use of Wet Mix Sprayed Concrete for Underground Support**
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# CONSIDERATIONS TO SPRAYED CONCRETE, A $CO_2$-INTENSE BUT EXTREMELY VERSATILE TUNNEL SUPPORT

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**Abstract**

Cement used in concrete production accounts for around 8% of global $CO_2$ output as expressed in terms of Global Warming Potential (GWP). Studies have shown that most of the $CO_2$ emissions in new-build construction are related to the consumption of cement in concrete. This can be interpreted as a clear message that our current approach to the construction of new tunnels must be reappraised.

This reappraisal should begin by making the $CO_2$ emissions from the predominant building material, concrete, the decisive factor at the earliest possible stage of the design process. Innovative design submissions supporting the reduction of $CO_2$ should now be carefully considered. At present the classification of concrete takes strength and exposure into account but not the Global Warming Potential. Transparency can and will help us to make the right decisions and this paper intends to open our eyes to how to reduce the GWP of underground construction.

The key decisions in tunnel design are taken by owners and designers, such as support requirements and related structural solutions. Safety and structural integrity are important facets of tunnel construction, but design expectations still need to be considered. Using sprayed concrete as the main excavation support is unavoidable, but on the other hand, it is the principal contributor to $CO_2$ emissions. Using less concrete to reduce GWP may seem the logical solution, but structural integrity still needs to be maintained over the agreed service life of the tunnel.

The human factor namely the choice of spraying equipment and cost pressures heavily influence the quality of the placed sprayed concrete. These practical aspects of construction need to become the focus of activity to ensure the expected structural longevity whilst consuming as little material as possible. It is of utmost importance, therefore, that all the elements that determine the result are optimized to achieve the desired quality and durability. These will include professionally trained nozzle operators, choice of admixture to optimize the concrete mix design as well as suitable machinery with low variability and pulsation. Experience and training are often the keys to mastering the prevailing conditions and satisfying increasingly demanding technical specifications.

## INTRODUCTION

Sprayed concrete (as shotcrete) was established as a structural support solution in underground construction around 70 years ago. It is extremely versatile and can be placed at any thickness for any given excavation contour. It is concrete as we know it with a few adjustments and due to the needs of the applications, it requires a higher cement (clinker) content. The challenge we are facing is the high $CO_2$ emissions that are inherent in cement from the clinker sintering process, which contributes about 8% to annual global $CO_2$ emissions [1].

If we look at tunnel construction sites, we must acknowledge that a huge portion of the $CO_2$ emitted during the construction of a new-build tunnel is bound as gray carbon in sprayed and cast concrete [2, 3]. We must then conclude that the solution must also lie in minimizing the use of concrete at tunnel sites. This is a design and application task, which will be outlined below.

## BASIC CONSIDERATIONS

Long-term structural stability is needed when underground structures are planned. With an expected service life of 100 years or more, any placed support measure must ensure structural integrity for this period or longer. To achieve this, safety factors are often subconsciously exceeded, and in many cases, sprayed concrete is regarded as only a temporary support measure, mostly in soft ground tunnel designs. Additionally, the quality of the placed sprayed concrete is often called into question, which results in unnecessary additional thickness. Consequently, more concrete than is needed is placed per linear tunnel meter.

Over-compensating for the safety factor conflicts with building tunnels by emitting as little $CO_2$ as possible. How can this be solved? (I) Owners and designers must consider the total emitted $CO_2$ per tunnel meter as a design criterion, in addition to the well-established factors such as cost. (II) Applicators must ensure that the placed sprayed concrete guarantees longevity through quality and durability to reassure stakeholders.

## TUNNEL DESIGN

The designers and owners are key to minimizing the emission of $CO_2$ in any project. By establishing $CO_2$ as a key decision criterion, incentivizing smart reduction proposals, and ensuring quality, all involved parties can and will be able to contribute to the common reduction goal.

Strength and exposure classes are established as quality criteria for all concrete types. To gain momentum for $CO_2$ reduction, GWP classes should also be established [4]. This then functions as a benchmark; any reduction will then benefit the innovating party. To achieve this, transparency in terms of $CO_2$ is needed, which EPDs (Environmental Product Declaration) provide.

Establishing specific EPDs (an EPD for a specific product with specific packaging produced at a specific site) is a lot of work and is not always needed. At the very early design stage of a tunnel project, accuracy can be regarded as the enemy and pragmatism your friend. Time is of the essence because carbon-saving decisions made early on tend to be more cost-effective and successful. Take decisions as early as possible, even if the input data for exact carbon calculations is not yet known. Therefore, at the very early project and design stages, the designers should focus on the big "$CO_2$ contributors", which in most cases will be the sprayed and cast concrete. Since $CO_2$ emissions are dominated by the cement content of concrete, and the cement is in turn dominated by its clinker content, cement transparency in terms of $CO_2$ per ton is a must. Cement EPDs for each cement type are needed because a generic cement EPD will not help in the calculation of projects. After all, the $CO_2$ emissions vary substantially between cement types.

Cement transportation, concrete mixing at the plant, concrete transportation to the site and finally spraying do not increase $CO_2$ by much and can be “ignored” during the early decision-making process, provided that extreme transportation distances are not expected. Some reference emission data is shown below in Figure 1.

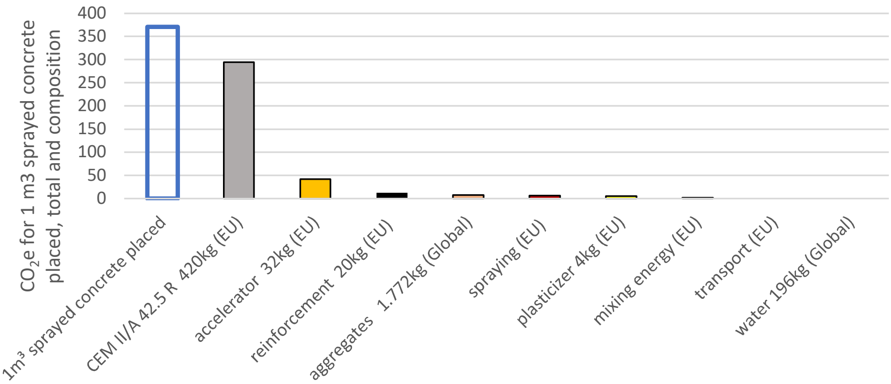


Figure 1: $CO_2$e for 1m³ applied sprayed concrete – relevance of $CO_2$e contributors [5]

The biggest lever to $CO_2$ savings is when sprayed concrete is accepted as permanent support, which is often not the case. Typical double-shell linings of sprayed concrete as a primary and often temporary lining and cast concrete for long-term load bearing are applied at equal thicknesses. Changing this black-and-white approach to something more pragmatic would be beneficial. For example if the primary support – the sprayed concrete – would at least contribute to a certain extent to the long-term tunnel stability. Our goal must be to apply the primary sprayed concrete lining to a level of quality, that passes all the requirements for long-term support and gains the confidence of stakeholders.

In cases where the sprayed concrete forms the final support, as is the case in many hard rock tunnels, the philosophy is to apply the sprayed concrete at such a high and reliable quality standard, that the long-term properties are never called into question. Additional safety factors such as increased thicknesses can then be omitted and higher strength classes can be defined based on real conditions.

A further design dimension to $CO_2$ savings per linear tunnel meter is spray-applied membranes or specialty watertight layers, such as those investigated in the Supercon research project [6].

Using fibers to replace mesh and bars is another method used to reduce the total concrete volume per linear tunnel meter. The sprayed support can easily follow the excavation contours at the required thickness and over-spraying of the exposed mesh is no longer required. However, the owner must accept the undulation of the final support, which may not always meet his aesthetic expectations and some operational requirements.

All the above considerations should be carefully thought through by the designer and put in the contract. They can be summarized as follows:

- GWP must be established as an additional design criterium
- Pragmatism is required at an early design stage, the focus should be on the big $CO_2$ contributors. Accurate $CO_2$ documentation can be carried out as the project progresses
- Sprayed concrete should be used as permanent support where possible

- Spray-applied waterproofing membranes and/or watertight sprayed concrete should also be considered
- Fiber reinforcement should be used where possible

More detailed elaborations on design considerations can be found in [7].

## APPLICATION OF SPRAYED CONCRETE

Sprayed concrete is concrete that comprises more cement than standard cast concrete but is placed differently. This placing may negatively impact the final quality if not carried out properly.

The sprayed concrete application can be broken down into 3 major contributing factors:

- The concrete mix-design
- The spraying equipment
- The skill of the applicator

Mix design can be exactly evaluated and set up in a very controlled way. But what should the focus and main goal be for permanent sprayed concrete? It can be safely assumed that sprayed concrete is barely loaded when used in a hard rock environment or as a final spray-applied lining on a primary support layer, both of that is not comparable to the loaded primary layer in soft ground support applications. Minimizing the expected concrete shrinkage of our unloaded or only locally loaded lining is required, which largely acts as a safety factor for potential local geotechnical challenges. Additionally, the concrete should be compact for longevity and should not demonstrate porosity from imperfect application methods. How can this be achieved and what can negatively influence these features?

Spray equipment with an interrupted concrete stream at the nozzle, also called pulsation will ultimately cause layering of the placed product with the probability to negatively impact the final properties of the concrete. Additionally, spraying concrete at the wrong distance and/or the wrong angle onto the receiving substrate will almost always negatively influence the quality of the hardened concrete.

### Spray Equipment - machine considerations

The pulsation of the concrete stream from the nozzle significantly impacts density and homogeneity and should therefore remain low. Strong pulsation will lead to layering (see Figure 2) and an uneven distribution of the accelerator which is caused by the mixing of a constant flow of the accelerator with the interrupted flow of concrete from the alternation (switchover) of the pump cylinders. Efforts have been made over the years in Austrian research projects to quantify this pulsation [8].

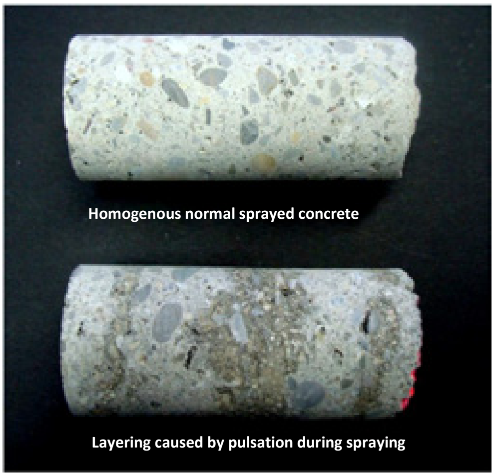


Figure 2: An example of layering compared to competent application

This has resulted in the introduction of a pulsation rate which will also be considered in the revision of the Austrian Sprayed Concrete Guideline [9]. It will define the pulsation rate of the spraying machine as the proportion of time between the concrete flow interruption and the total stroke. The interruption time (ti) begins at the start of the switchover (end of stroke) when the pressure drops and until it again reaches a pressure level of 90%. The stroke time (ts) begins at the start of the switchover (end of stroke) until the next switchover, see Figure 3 below.

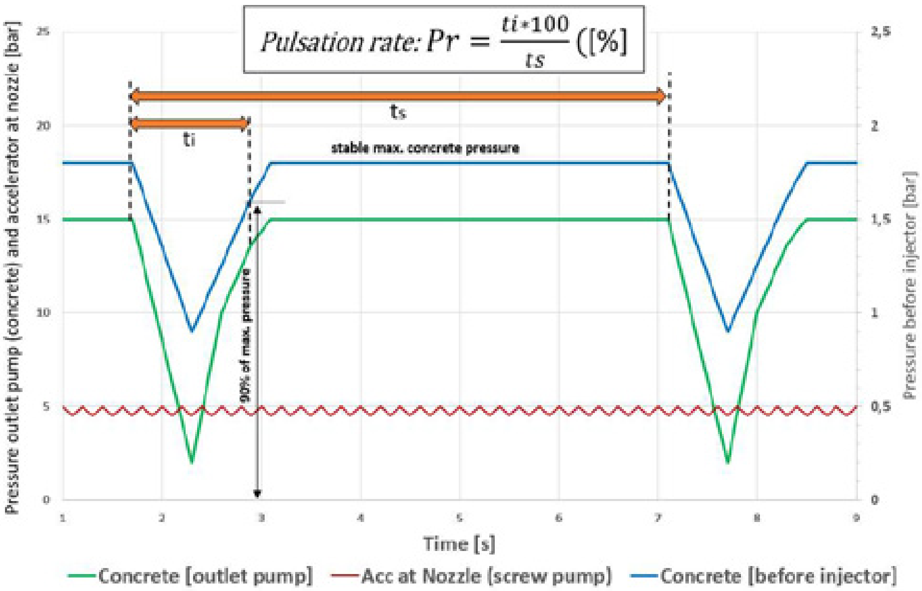


Figure [3]: Typical pressure development during the operation of a sprayed concrete pump (graphic-based on [8])

The pulsation rate depends on the technical parameters of the machine, concrete output, and also partly on the rheology of the concrete. The recommended pulsation rate for high-quality durable sprayed concrete linings is expected to be below 10% according to the new Austrian Guideline.

Pulsation increases as the concrete output increases. This is partly caused by the decreasing filling ratio of the concrete pump cylinders because faster-moving pistons leave less time for the concrete to fill. Cylinder geometry (diameter and stroke) and the rheology of the concrete influence the degree of filling as well as the details of the switchover device. Typical filling ratios are in the range of 70 – 85%.

It is assumed that today's accelerator dosing units accurately deliver the required volumes and are automatically synchronized with the actual concrete throughput. Choosing accelerator pumps with a constant delivery rate and calibrating them time over time will add to the quality of the final sprayed concrete.

**Mix-design**

The key to a good application is the uninterrupted concrete flow at the nozzle, which means little or no (visible) pulsation.

Theoretical machine output measurements are carried out with water as a reference medium instead of concrete because it easily fills the pistons even at high speed due to its low viscosity. In contrast, if a sprayed concrete mix, especially when it has lost its workability is forced through the concrete pump and nozzle, it cannot completely fill the cylinders, causing a high pulsation and a low final concrete quality. Therefore, the viscosity of fresh concrete mixes matters.

The rheological properties of a sprayed concrete mix can be described technically by yield stress and plastic viscosity. Simple devices such as the V funnel [10] can be used to assess the rheology, where only the through flow time is measured. The faster the flow through, the better the flow into the pump cylinders will be and the higher the filling rate.

Concrete rheology is influenced by many parameters, such as aggregate grading, amount of water, the amount of cement/binder and type as well as plasticizer technology. Low concrete viscosity is required during pumping, which can be shortly after mixing or hours later depending on the project conditions. Therefore, a combination of specialty plasticizers and hydration control admixtures is needed to ensure that the required workability over a longer time is maintained. Some suppliers claim low concrete viscosity over long periods but it is good practice to use the V funnel test at different concrete ages, best supplemented by the well-known test on the DIN spread table for reassurance.

To reduce the shrinkage potential of the concrete mix, the use of anti-shrinkage admixtures is advisable. Additionally, polymers can be used to soften the concrete matrix when thin-layer spraying is expected. Fibers aid the distribution of the partly unavoidable shrinkage cracks and limit their width so that they remain below critical levels to ensure concrete durability [6].

**Applicator**

The human factor in the quality of placed concrete is often underestimated. Many factors influence application such as nozzle distance and angle, ability to assess the quality of concrete upon arrival, accelerator dosage, and finally the machine throughput. All the factors above lie in the hands of the operator and must therefore be trained. Some training schemes are already established [11, 12] which are being increasingly demanded by owners, which regarding quality is a good sign.

The accelerator type and dosage are named here because this can easily be checked at all job sites by simple observation. Many operators aim for higher-setting characteristics (in practice strength after 3-10 minutes), to allow the build-up of thicker layers. However, this does not always have a positive influence on final quality. When the sprayed concrete stream hits the substrate, it should be of a “yogurt-type” consistency, slightly soft to accommodate compaction, but not too soft, to avoid sagging. It should become stiff in seconds. This behavior is influenced by the accelerator dosage and type and can be assessed in tests before critical applications.

### Curing

Curing is probably the most underestimated contributor to improved quality. As mentioned earlier, many sprayed concrete layers are only lightly loaded if at all, hence they are going to experience shrinkage. Shrinkage is caused by water loss, therefore water loss should be reduced and/or delayed.

Redirecting the ventilation from freshly applied sprayed concrete layers will help. The use of a water hose at a very early age (hours and not days after application) is easy to organize. Internal curing admixtures or a spray-applied film acting as evaporation protection are further approaches to consider.

Some more information on durable sprayed concrete lining can be found in [12, 13].

## CONCLUSION

Building tunnels with less $CO_2$ is a team effort, it must begin early, and pragmatism is required. Key to it is awareness, that the rules are set by the owner by establishing GWP as additional decision criteria, followed by the designer carefully scrutinizing the main $CO_2$ contributors already at a very early project stage. When the construction starts, build quality is the main goal to ensure the planned longevity of the tunnel lining without heavy maintenance needs. We all must contribute with all our abilities and start now, excuses can no longer be accepted.

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NONLINEAR NUMERICAL MODELLING OF SPRAYED CONCRETE

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**Abstract:**

The material behaviour of sprayed concrete is well understood. The ageing of mechanical properties, creep and the nonlinear stress strain behaviour have been investigated in experimental and theoretical studies. Despite this, the last phenomenon is rarely incorporated in numerical models for design analyses. This paper presents the validation of a strain-hardening plasticity and a nonlinear elastic constitutive model, using experimental data from both uniaxial and triaxial compressive tests. The models use Itasca CG's FLAC3D finite difference software.

This subject raises some interesting questions such as code compliance and the use of shell elements vs zones (solid elements) which will be explored in the paper. Many reinforced concrete codes permit the nonlinear analysis of structures. Where the codes indicate a stress strain behaviour this is normally formulated as in one dimensional stress space. Most commercial software for geotechnical numerical modelling offers wider variety of constitutive models for zones than shell elements. The later are often only linear elastic in stark contrast to the real behaviour of sprayed concrete. Using zones for a thin tunnel lining is problematic for several reasons. Shell elements are more appropriate and easier to use.

The work presented in this paper was later successfully extended to the full modelling of sprayed concrete lined tunnels. The nonlinear models predicted significantly lower loads that the usual design approaches. This would have enabled a cost and carbon footprint saving of 25%.

**Keywords:**

Nonlinear elasticity, plasticity, constitutive modelling, design calculations.

## INTRODUCTION

Despite the thorough understanding of the mechanical behaviour of concrete (and sprayed concrete), there remains a gap between the simulation of this behaviour in design analysis and a more complete representation of this. Specifically, the state-of-the-art in commercial design include phenonema such as the ageing of mechanical properties (like strength and stiffness) and creep (usually through a reduced, "effective" modulus of elasticity. The behaviour is modelled as linear elastic in both compression and tension. The nonlinear stress strain relationship – both in compression and tension – is typically ignored. Often this is justified on the grounds that most tunnel linings are designed so that they are not loaded to a high degree of utilization – i.e. to close to the strength of the concrete. In fact, above about 30 to 40% of the compressive strength, concrete exhibits a nonlinear stress strain response [1]. Numerous authors have demonstrated that sprayed concrete tunnel linings can experience utilizations above 30%, especially at the face when the sprayed concrete is young and has a low strength (e.g. [2]). This raises the question of whether or not it is important to include the nonlinear behaviour in design calculations for sprayed concrete lined (SCL) tunnels, such as numerical models. Findings from previous studies have suggested that including this nonlinearity can lead to lower predictions of loads in SCL tunnels [3].

This paper will explore two theoretical options for simulating nonlinear stress strain behaviour. These are applied in shell elements and validated against experimental data. Where either option is available for shell elements in a numerical modelling software, this shows that the lining can be easily modelled more realistically.

## CONSTITUTIVE MODELLING OF SPRAYED CONCRETE IN TUNNEL DESIGN

This section will set out the context of this study in more detail by elaborating on the behaviour of sprayed concrete in compression, the theoretical models for this and previous approaches to replicating this in numerical models.

### Nonlinear stress-strain behaviour of concrete

Considering the behaviour in compression, concrete exhibits a nonlinear stress strain relationship when the applied stress exceeds about 30-40% of its peak strength, in a uniaxial compression test performed in the laboratory [1] – see Figure 1. As the applied stress increases, the tangent stiffness reduces due to microcracking within the concrete matrix. The same applies to sprayed concrete [2]. Various relationships have been proposed to simulate this and some design codes include a one-dimensional version of this (e.g. Eurocode 2). Incorporating nonlinearity has been found to reduce bending moments by up to 50% in numerical models of a similar tunnel, with a smaller reduction in the axial forces ([3] &[4]).

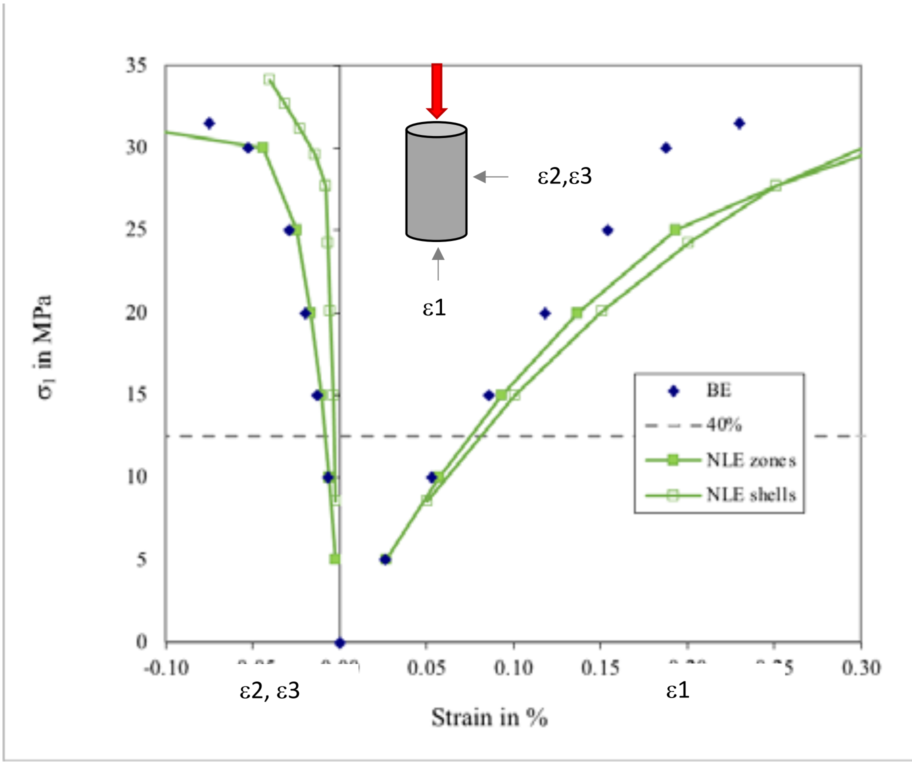


*Figure 1 Uniaxial compression test on sprayed concrete from the Brite Euram (BE) project with predictions using a nonlinear elastic model (NLE) with either zones or shell elements (after [3]). 1 = the directon of loading & 2,3 = the lateral directions; positive strains are inwards to the centre of the sample.*

Strictly speaking, a nonlinear stress-strain relationship should be formulated in a three dimensional sense because the stress state in the lining is biaxial, rather than uniaxial. The theory of nonlinear elasticity can be used to simulate this behaviour. The plasticity models simulate this nonlinear stress-strain behaviour by following the principle of strain hardening. Theoretically, this is more correct since concrete only behaves elastically up to about 30-40% of its peak strength.

Concrete also behaves in a nonlinear manner in tension. However, this is beyond the scope of this paper.

**Nonlinear elastic model**

A nonlinear elastic constitutive model is one way to simulate the stress-strain behaviour and the model based on the work of Kotsovos and Newman has been found to work well [3,5] The shear and bulk stiffnesses of the concrete vary depending on the actual octahedral shear stress vs the octahedral shear strength – i.e. how close the concrete is to yielding in shear. The same relationship is assumed for the behaviour in tension. The equations proposed by [5] for cast concrete have been extended to cover sprayed concrete at early ages and applied successfully in numerical models in previous studies [3]. This is the theoretical basis for the nonlinear elastic model referred to later.

One disadvantage of a nonlinear elastic model is that it is harder to reflect the irreversible nature of plastic deformation when unloading occurs. A Masing rule which resets the moduli to their initial values has been included in this model to simulate the hysteresis observed during unload-reload loops. Tunnel linings generally do not experience unloading (except at tunnel junctions). To avoid this model "overshooting" the ultimate strength, the moduli are set to very low values when the stress approaches the ultimate strength.

**Elastoplastic model**

Strain-hardening plasticity is an alternative theoretical framework for simulating the nonlinear stress strain behaviour. A key advantage of this type of model is that it ensures that the stress in an element will never exceed the peak strength – unlike a nonlinear elastic model. Furthermore, in numerical modelling programs like FLAC3D, elastoplastic models usually permit a different stress strain relationship to be set in tension from that in compression. A bilinear stress strain relationship is suitable for fibre reinforced sprayed concrete in tension.

A Mohr-Coulomb failure criterion is often used to describe the peak strength of concrete [3]. This is a very simple failure criterion, defined by two parameters – cohesion and the angle of friction. There are many other plasticity models for concrete but Mohr-Coulomb has the advantages of simplicity and a common availability in numerical modelling programs. Considering peak strengths over a range of confining stresses, a reasonable choice of these parameters can be found by setting friction, $\phi$, to 37° and using the following equation:

$$cohesion = f_c \cdot \frac{(1-\sin\phi)}{2.\cos\phi}$$

Equation 1

where fc = the uniaxial compressive strength [3].

When using a Mohr-Coulomb strain-hardening constitutive model to simulate the curved stress-strain behaviour, there is no unique relationship for these parameters. The parameters can be chosen arbitrarily at any point. Figure 2 shows two possible methods for setting these parameters. Above the point of yielding (e.g. 40% of the peak strength), the cohesion increases with the increasing plastic strain to mimic the behaviour of the concrete while the angle of friction remains constant (Method a)) or the cohesion can increase and at the same time the angle of friction can decrease (Method b)).

Varying the angle of dilation did not seem to affect the predicted results much.

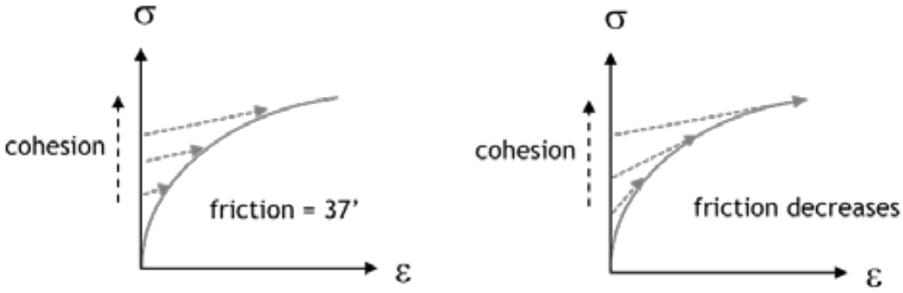


*Figure 2 Schematics showing options for setting Mohr-Coulomb parameters: a) cohesion increase while friction is constant and b) cohesion increase while friction decreases*

**NONLINEAR STRESS-STRAIN BEHAVIOUR IN SHELL ELEMENTS**

Validation is a critical step in numerical models. For example, where complex constitutive models are used, the behaviour of the code should first be checked with simple models and compared with the theoretical prediction and / or test data. The two theoretical models outlined above have previously been implemented successfully in zones which represented the sprayed concrete [3] – see Figure 1. Zones means solid 2D – or in this case 3D – elements in the numerical model. In this study, this was extended by using shell elements to represent the concrete in the modelling of the laboratory tests (and later tunnel linings). The FLAC 3D finite difference software from Itasca CG was used for all of the modelling.

Figure 3 shows how these nonlinear constitutive models for the sprayed concrete compare to an example of laboratory test data from a triaxial test, with a confining stress of 1.0 MPa in the transverse direction, on a sample of fibre reinforced sprayed concrete with an age of 12 hours (see [3] for further details of the original test data and validation). The sample was compressed in the main direction of loading as the load was increased while it expands in the lateral direction due to the Poisson's ratio effect. A uniaxial test was also successfully modelled for validation – see Figure 1, where only the nonlinear elastic model is shown for clarity.

Figure 3 shows a reasonable agreement with the test data in the main axis of deformation ($\varepsilon 1$). The match could be improved by varying the initial elastic modulus. The models shown in Figure 3 estimated the modulus using the standard equations in Eurocode 2. The nonlinear elastic model (NLE) matched best. Simply varying the cohesion at a constant angle of friction of 37' (as in MCSS coh only) did not provide a good match. Therefore the second approach of varying both cohesion and the angle of friction (from 45' to 27') was developed. This provided a better match and a similar shape to the nonlinear elastic model (NLE).

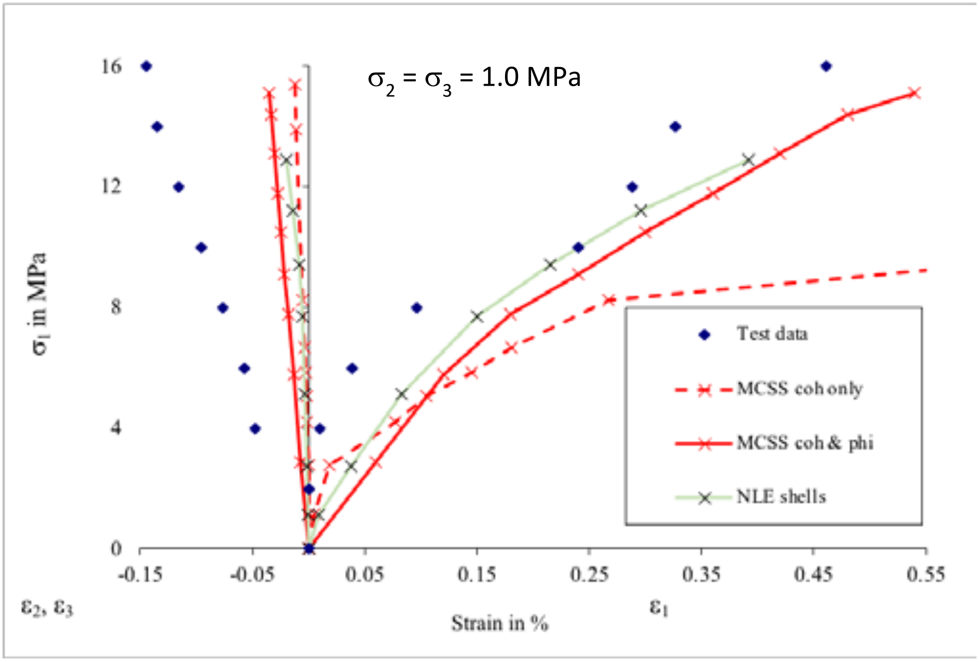


*Figure 3 Triaxial test on sprayed concrete with predictions from various numerical models & experimental data (after [3]) . 1 = the directon of loading & 2,3 = the lateral directions; positive strains are inwards to the centre of the sample.*

The goal in this study was to develop nonlinear models which could reliably simulate any sprayed concrete and therefore one which could be used in any design analyses. Of course, the input parameters can be tuned to achieve a better match with the experimental data (albeit only for strains in the main direction of loading, ε1). However, this could only have been done by diverging from known inputs (e.g. the stated uniaxial compressive strength of the samples) or established equations for estimating other inputs (e.g. elastic modulus from compressive strength).

Overall, this modelling showed that both the nonlinear elastic and plastic hardening models can mimic the nonlinear stress strain curve of concrete. They can provide a reasonable match with the test data in the primary direction of loading, particularly at lower levels of stress.

## DISCUSSION

This study has demonstrated the viability of including nonlinear stress-strain behaviour in numerical modelling of concrete using shell elements. This opens the path for the use of nonlinear shell elements in numerical models for tunnel design.

### Zones / solid elements vs shell elements

There are two important reasons why shell elements are better than zones or solid finite elements. Firstly, many of the latter assume a constant strain elements. When an object is in bending, there is a variation in strain through the object, normally assumed to be linear. Constant strain elements approximate this with a stepped profile (see grey blocks in Figure 4). The more elements there are, the better the approximation. To reduce the error in the response to bending, the thickness of a tunnel lining should be divided into more than 6 or 7 layers of zones. However, here we run into another drawback. In a numerical model, around a tunnel, the ground is typically divided into zones with a dimension of 200 to 400 mm. A sprayed concrete lining is typically 100 to 300 mm thick. If we divide that into 6 layers, each one will be 15 to 50 mm thick. This is much smaller than the neighbouring ground zones. This creates significant problems when discretizing the mesh.

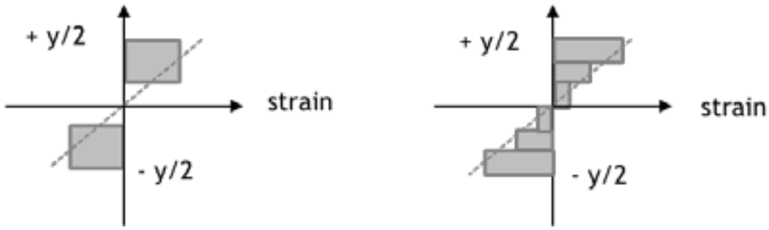


*Figure 4 Schematic of the approximation of strain in pure bending through the section of a beam a) using 3 constant strain zones and b) 6 constant strain zones vs simple beam theory (dashed line)*

The second drawback is bending moments cannot be easily calculated when using zones. The stability of a lining is usually evaluated on the basis of the axial force and bending moment at key locations. Significant manual post-processing is needed to estimate bending moments in zones. In contrast, in shell elements this is a standard output.

**Code compliance**

One interesting issue raised by the use of elastoplastic constitutive models is that of code compliance. In the case of linear elastic models, this is straightforward. Typically, following the Eurocode approach to design, the ground parameters are chosen as *Moderately Conservative* values and unfactored in the numerical model. The predicted lining loads are multiplied by the partial factor of safety on loads (e.g. 1.35) and compared with the concrete capacity curve, including a partial factor of safety on the materials (e.g. 1.5 for concrete strength). If the factored loads lie within the factored strength envelope, then the design is deemed to be safe.

However, what should we do in an elastoplastic model? The stress will be capped in the shell element at the "strength" of the concrete. If we apply a partial safety factor to the strength inside the model, then the loads could be constrained to fit within the capacity since the stress is limited (although there is still the partial factor on loads). In reality, the stress in the lining could rise above this factored limit. This creates the risk that the design could conceal the potential extent of yielding in a heavily loaded tunnel. An additional check on the magnitude of the predicted strains in the elements in comparison with the allowable strain is one way to assess whether or not "excessive" yielding has occurred. Arguably, it is better to use unfactored strengths in the numerical model. In that case, the stresses can rise above the limits that are normally accepted. This will be clearly visible when plotting the predicted (factored) loads on the normal capacity curve.

The case of a nonlinear elastic model is similar to the linear elastic one since there is no cap on the stress in the elements.

**The impact of nonlinear modelling on designs**

There is no space in this paper to present the full details of the further parts of the original research, in which the nonlinear models were applied to a sophisticated numerical model of the construction of large SCL tunnel junctions in soft ground. This extends research that has been published in [6] and a full paper on the nonlinear models is currently under review. As a sneak preview of the results from the latter, Figure 5 shows that – at least in this case – nonlinear models (either based on nonlinear elastic (NLE) or elastoplastic (MC) theory) predicted lower loads than the normal linear elastic models (Et)). The results from the nonlinear models also agree better with the monitoring from the real tunnel. This is in line with findings from other similar studies [3]. In this particular case, if a nonlinear model had been used for the design, the thickness of the sprayed concrete lining at the junction could have been reduced from 800 mm with bar reinforcement to 600 mm of fibre reinforced sprayed concrete. This 25% reduction in materials and the shift away from bar reinforcement would have saved construction time as well as cost. Considering the embodied carbon of this construction, most of this lies within the materials themselves so there would have been a similar reduction in the carbon footprint.

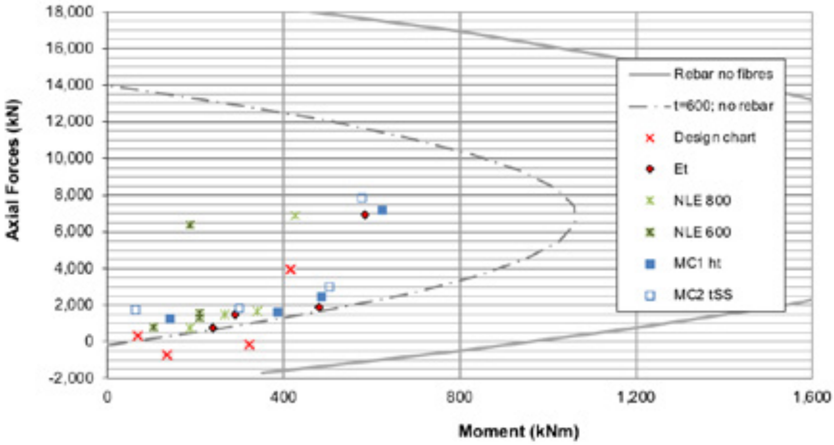


*Figure 5 Lining loads vs capacity at a large diameter SCL tunnel junction after [6]*

## CONCLUSIONS

The nonlinear behaviour of concrete – including sprayed concrete – is well known to have a considerable influence on the response to loading. This is rarely considered in the design of concrete structures such as tunnel linings yet it has been shown to have a considerable beneficial impact. Many structural analysis programs only offer the possibility of nonlinear constitutive behaviour in solid elements (zones). This is problematic when analysing thin structures embedded in the ground like tunnel linings. FLAC 3D has recently introduced the option of elasto-plastic shell elements. Several options for simulating nonlinear behaviour of sprayed concrete in shell elements were explored in this study. These were validated on small scale models against actual test data. When applied to a full scale model of a sprayed concrete lined tunnel, this study replicated previous findings by others that nonlinear design methods could lead to significant savings in time, cost and embodied carbon.

## ACKNOWLEDGEMENTS

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# FIRE RESISTANT CONCRETE TUNNEL LINING: IS THERE AN ALTERNATIVE TO 2 KG MICRO POLYPROPYLENE FIBRES?

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## Abstract

The method to avoid explosive spalling of hardened concrete/sprayed concrete under heavy fire exposure is to add micro polypropylene (pp) fibres during the mixing process. During a heavy fire, and rapid warming of the concrete, the pp-fibres melt increasing the permeability of the concrete creating channels for the vapor pressure to escape. Without pp-fibres the build-up of vapor pressure may cause concrete layers spall in a rather violent manner from the fire exposed side, with loss of concrete thickness/cover as a result.

In NPRA's handbook N500 "Road tunnels" fire resistant concrete is required as protection where inflammable material is used as frost insulation. The pre-accepted solution (requirement) in N500 is to use 2 kg pp-fibre pr. m$^3$ concrete. This requirement has been the standard now for 20 years. There has been a significant development of cements and binders since then.

During the last years, attention on the negative environmental impact of plastic has grown dramatically. Rebound of fibres on the tunnel floor during the spraying process is experienced challenging in water run-off and sewage clarification. Other side-effects of the micro pp-fibre are problems with fresh concrete workability, as well as the price of the fibre itself.

With respect to these side-effects, any reduction of the micro pp-fibre dosage would be positive. The paper will describe and give results from a collaboration project. The project involves fire tests on both sprayed and cast concrete elements made with different fibre dosages, and two fibre types.

## INTRODUCTION

Where concrete is to protect inflammable material in a tunnel, NPRA's handbook N500 Road tunnels [1] describe fire-resistant concrete fulfilling the following requirements: The average temperature on the backside of the fire-exposed concrete surface shall not exceed 250°C during a 60-minute hydrocarbon (HC) fire, and the inflammable material on the backside shall not develop continual flames.

The build-up of a typical Norwegian tunnel is illustrated in Figure 1. The focus in this paper is the water- and frost protection system, and in particular the fire-resistant concrete – which can be either mesh-reinforced sprayed concrete or cast concrete elements.

To claim fire-resistance a pre-accepted solution is to use 2 kg/m$^3$ of a micro polypropylene (pp) fibre with thickness 18 µm and length 6 mm. Minimum concrete thickness is set to be 80 mm, which is the normal thickness for an inner lining of sprayed concrete. Prefab concrete element linings have traditionally been 150 mm thick with two layers of mesh-reinforcement.

The thickness of the frost insulating material (either PE-foam or XPS) on the backside of the fire-resistant concrete is required to have a thickness ranging from 45 – 90 mm according to the frost exposure in the tunnel.

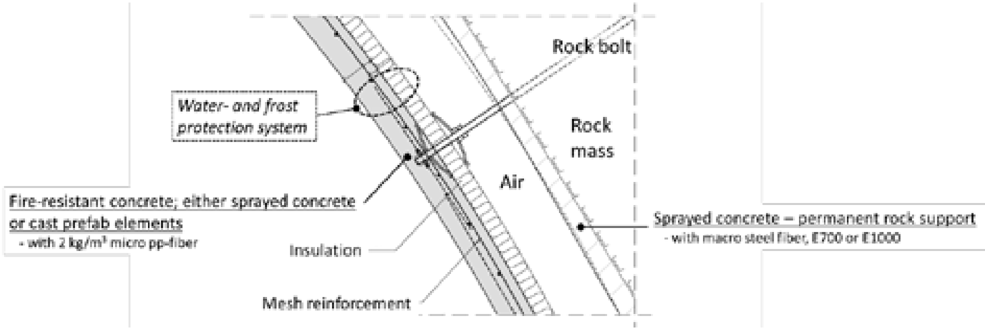


*Figure 1 Typical build-up of a Norwegian tunnel (based on [2]): First, sprayed concrete permanent rock support (with separate bolting according to the rock mass class) produced according to [3]. Then, an inner lining of water- and frost protection including insulation and mesh-reinforced fire-resistant concrete.*

During the last years, attention on the negative environmental impact of plastic has grown dramatically. For sprayed concrete rebound of micro pp-fibres on the tunnel floor during the spraying process is experienced challenging in water run-off and sewage clarification. Other general side-effects are problems with fresh concrete workability, as well as the price of the fibre itself. With respect to these side-effects, any reduction of the micro pp-fibre dosage would be positive.

The part of an inner lining, of either cast or sprayed concrete, being exposed to a heavy fire must be replaced afterwards independent of the degree of spalling since the concrete strength is jeopardized after such extreme treatment anyhow.

The pre-accepted solution (2 kg/m$^3$ micro pp-fibre) has been the standard now for 20 years (also in Eurocode 2 [4]. There has been a significant development of cements and binders in recent years. In the literature it is reported that lower pp-fibre dosages, higher fibre thicknesses [5][6][7][8][9] and even other types of fibres [10][11][12] may also give fire resistance. Another issue is that a standardized procedure to test fire resistance of concrete exposed to a HC-fire is lacking in Norway. For the more intensive RWS fire exposure we have though a specified procedure [13].

The paper will describe and give results from an on-going collaboration project, involving HC-fire tests carried out in a certain manner (inspired by the RWS-procedure [13]). The project involves, so far, tests on both sprayed and cast concrete elements made with different micro pp-fibre dosages, and two fibre types with different diameter. The paper also discusses design and production of test elements, aspects that may have impact on the fire test results.

## TEST METHODS

### Porosity and initial moisture content

The motive was to characterize one sprayed and one cast concrete (both with 1 kg 18 µm fibre) just before fire testing, after curing/storage and transport to the fire research laboratory. The measurements were done according to procedure described in [14] (based on [15]). Cores were drilled from companion specimens produced parallel with the production of the larger fire test elements, see later chapter. For each of the tested concretes, 2-3 samples were split out from each of two cores. The top and bottom layer from each core was not used. The procedure involves successive weighing of each sample: (1) in air, immediately after splitting, (2) in air, after 7 days water curing, (3) in air, after 7 days at 105 °C, (4) in air and water, after 7 days water curing, (5) in air, after water pressure treatment at 50 atm, (6) in air, after 7 days at 105 °C (for control of mass loss).

**Measurement of initial relative humidity (RH)**

From the same companion specimens, as above, one core was drilled for each of the two concretes. Parts of crushed concrete from the central part of each core were spread out on three test tubes placed in stabile 20 °C climate. On each of the three a Vaisala moisture sensor was installed from the top and sealed with parafilm. RH was registered over time and the reported result is after four days, which was the measured maximum value.

**Fire exposure tests**

The test elements were exposed to a 60-minute HC fire acc. to NS-EN 1363-2 [16], see Figure 2. The fire tests were performed at RISE Fire Research laboratory in Trondheim, Norway. A gas heated test own (called Pilot own) with inner opening of 1,55 m x 1,55 m was used. Each concrete test element (1,5 m x 1,5 m) was fixed to a steel frame with bolts in each corner. The small split between the steel frame and test element circumference was tightened with a strip of insulation (AES-fibre). The test elements were tested one by one, vertically oriented. During testing the intended HC temperature curve was given priority ahead of the tolerance for overpressure (20 Pa) given in NS-EN 1363-1 [17]. The tolerance was exceeded in periods, but it is believed that this had no influence on the fire spalling behavior of the test elements.

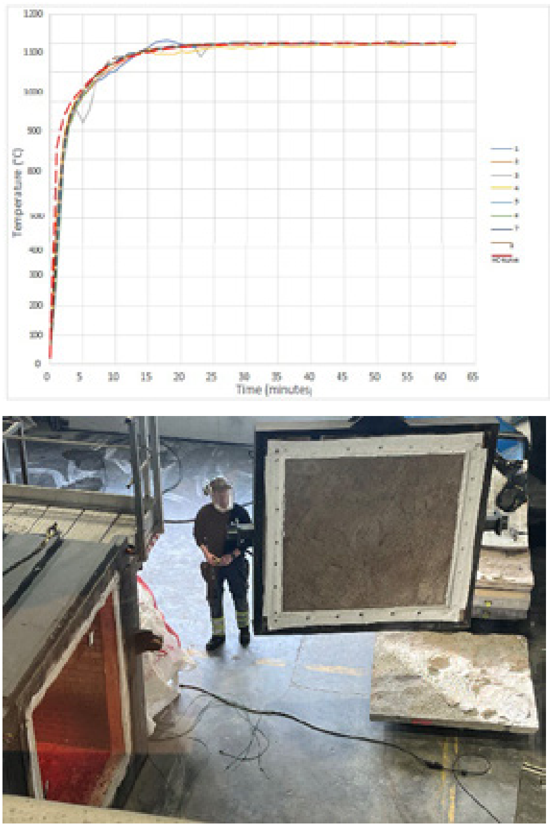


*Figure 2 (top) HC fire curve (red dotted line) and measured own temperatures [20] during the eight sprayed concrete element tests. (bottom) Fire test own and test element.*

## CONCRETES MIX-DESIGN

### Mix-design

The concrete mix design (except micro pp-fibre) of the sprayed and cast concrete is shown in Table 1. For the sprayed concrete the mix design is for the basic mix delivered from the concrete plant, before spraying. After spraying the water-to-binder ratio increased from 0.42 to close to 0.45 due to the extra water from the accelerator.

### Strength class and aggregate type

For the sprayed concrete strength-class C35/45 was described. The used sand (Grefsrud 0-8 mm) is washed natural aggregate with dominance of gneiss/granite and sandstone.

For the cast concrete strength-class C45/55 was described (denoted SV-Standard in the NPRA code). Most of the aggregate is crushed and consist of gneiss, granite, diorite, quartzite, felspathic rock (84%) and mafic rock (13%) and phyllite (3%). A part of the sand fraction (72 % of 0-8 mm) is natural sand and consist of mafic rock (77%) as well as gneiss, granite, diorite, quartzite and felspathic rock (23%).

*Table 1 Concrete mix design (kg/m$^3$), and average 28-days cube strength*

| Material | Sprayed concrete basic mix (B35M45) | Cast concrete mix (B45MF40) |
|---|---|---|
| CEM II/B-M 42,5 (c)* | 439 | 384 |
| Silica fume (s) | 20 | 12 |
| Extra fly-ash (FA) | 40 | - |
| Sand 0-8 mm | 1536 | 274 |
| Stone 8-16 mm | - | 992 |
| Stone 16-22 mm | - | 538 |
| Water | 218 | 155 |
| Superplastisizer | 5.7 | 4 |
| Air entrainer | 0.44 | 0.8 |
| Effective water-to-binder; $w/b = w/(c + 2 \cdot s + 0.7 \cdot FA)$ | 0.42**<br>0.45** | 0.38 |
| Average 28-days cube strength (MPa) | 57.7*** | 65.9 |

* *The given cement (CEM II) contains 13 % fly-ash (Heidelberg Materials, Adjusted Standard Cement FA)*
** *For the sprayed concrete the w/b is 0.42 in the basic mix before spraying, and (max.) 0.45 after spraying due to the extra water from the accelerator*
*** *Strength was measured on cast cubes before spraying*

## USED FIBRES AND TEST PROGRAM

Totally 13 large test elements for HC-curve fire testing were made. Type of pp-fibre, fibre dosage and number of test elements for each variable is given in Table 2. Product data for the two fibre types is given in Table 3. Note that two nominally identical elements were made for each of the three variants with 1 kg pp-fibre. This was done as a repeatability test, since high variability in fire testing appear to be a challenge and such data appear to be scarce in the literature [18].

The intension was to use an 18 μm and 32 μm fibre, but the latter was not purchasable in Norway at the time of element production. A 50 μm fibre was available from a repair mortar retailer, and this fibre was therefore used instead of the 32 μm fibre. The material properties of the pp-fibres are shown in table 3.

*Table 2 Test program: Fibre types, dosages, and number of test elements*

| Fibre type (diameter) | Dosage | Sprayed concrete | Cast concrete |
|---|---|---|---|
| 18 µm | 0.5 kg/m$^3$ | - | 1* |
| | 1.0 kg/m$^3$ | 2** | 2** |
| | 1.5 kg/m$^3$ | 1 | 1 |
| | 2.0 kg/m$^3$ | 1 | 1 |
| 50 µm | 0.5 kg/m$^3$ | 1 | - |
| | 1.0 kg/m$^3$ | 2 | - |
| | 1.5 kg/m$^3$ | 1 | - |

** An alternative fire curve was used during this test, first 450 °C for 1 hour (heating acc. to ISO-curve) followed by further heating to 1100 °C acc. to HC-curve for 1 hour.*
*** A small specimen was also made from this concrete to determine porosity and initial water content prior to fire testing*

*Table 3 Micro pp-fibres; product data*

| Property | Fibre type (nickname) 18 µm | 50 µm |
|---|---|---|
| Fibre diameter | 18 µm | 50 µm |
| Fibre length | 6 mm | 6 mm |
| Specific gravity | 0,91 kg/dm$^3$ | 0,91 kg/dm$^3$ |
| Tensile strength | 300 MPa | 152 MPa |
| E-module | >3500 MPa | 1135 MPa |
| Melting temperature | 160 °C | 160 °C |
| Number of fibres pr. kg | 725 million | 100 million |

## TEST ELEMENT PRODUCION

### Sprayed concrete elements

The sprayed concrete basic mix was delivered by Betong Øst, Gardermoen, and spraying was executed by AF Anlegg, see *Figure 3*. Element preparations and spraying was carried out at an AF Anlegg area at Blikkveien 151, Jessheim.

Element formworks (totally eight) with inner dimensions 1.5 x 1.5 m$^2$ and thickness 80 mm, and with 50 mm PE foam at the rear side were made. One steel mesh K335 reinforcement were installed in the center with the use of plastic spacers, see Figure 4. Two lifting anchors were placed on one of the sides, denoted the top. Three temperature wires type K were installed both in the center at the reinforcement and at the interface between concrete and insulation.

*Figure 3 Concrete delivering and equipment for spraying.*

The elements were sprayed in two layers, see Figure 5. The second layer were sprayed the day after the first, which is quite normal procedure for AF in tunnel projects. However, on test elements like this it was difficult to obtain even surfaces due to the small areas. After spraying the second layer some areas were scraped/trowelled to aim for 80 mm final thickness. The elements were then wrapped in plastic, see Figure 6, and stored on-site until transportation to the laboratory.

In addition, one smaller specimen was also sprayed (1 kg 18 µm fibre), 300 mm x 400 mm and thickness 150 mm. The specimen was given the same wrapping and storage as the elements and used later to measure moisture state and relative humidity of the concrete shortly before the fire testing.

The elements (and the smaller specimen) were sprayed in week 40-2023, stored on-site until week 4-2024, then transported to the fire test laboratory in Trondheim and tested week 6 to 8 in 2024. Hence, the sprayed elements had an age of 16-18 weeks when tested.

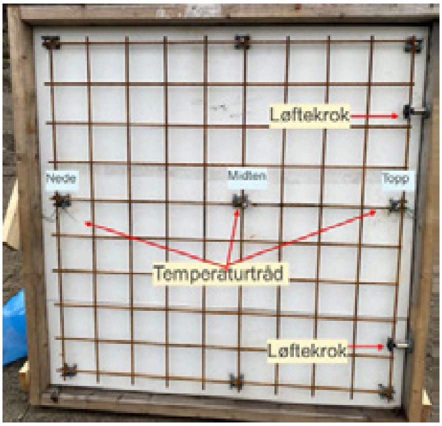


*Figure 4 Element formwork with PE-foam at the rare side, mesh reinforcement in center, thermo-couple locations and lifting anchors. Final locations during later fire test: topp=top, midten=center, nede=bottom.*

*Figure 5 After spraying first layer (left) and second layer (right).*

*Figure 6 Sprayed elements wrapped in plastic.*

**Cast concrete elements**

Element formworks (totally five) of 1.5 x 1.5 $m^2$, thickness 150 mm and with 50 mm PE-foam at the rear side were made. The cast elements were reinforced with two steel mesh K335, as well as 3 x 3 thermo-couples and lifting anchors like the sprayed elements, see Figure 7 (left). Concrete cover from the bottom mesh was 40 mm to the insulation and from the top mesh 50 mm to the surface (fire-exposed side). The three thermo-couples inside the elements were located approximately along the top mesh. One small specimen was also cast (1 kg 18 μm fibre), 400 mm x 400 mm and thickness 150 mm, for the supporting measurements.

The elements were cast by Sandnes & Jærbetong, Sandnes in week 44 – 2023, and covered with plastic and stored indoor, see Figure 7 (right). Transportation to Trondheim was in week 4 – 2024, and the testing was in week 6 to 8 in 2024. Hence, the cast elements had an age of 12 to 14 weeks when tested.

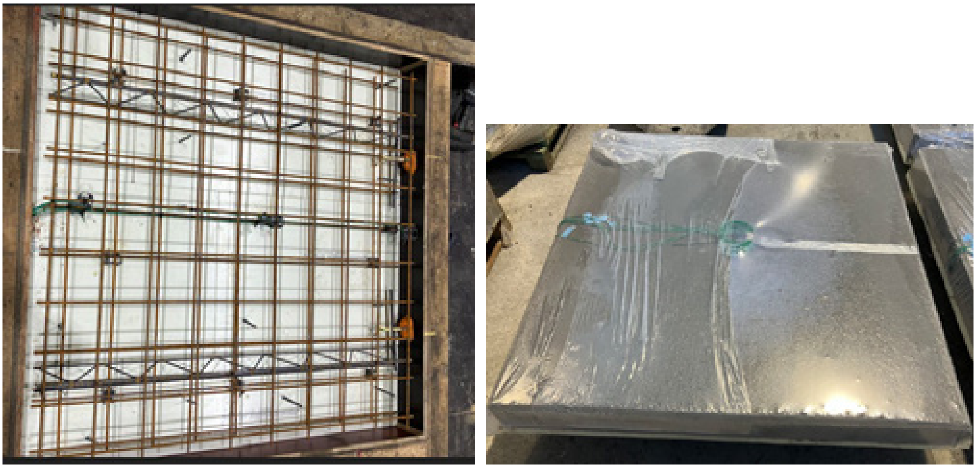

*Figure 7 Formwork with two layers of mesh reinforcement, thermos-couples and lifting anchors, ready for casting (left). Cast elements wrapped in plastic (right)*

**RESULTS AND DISCUSSION**

Test results from the laboratory were reported in [19][20]. The moisture and porosity measurements are shown in Table 4. Especially the results for the sprayed concrete are somewhat surprising, where moisture and porosity is very high, but density is low. The reason for this is unknown.

The sprayed concrete (w/b=0.45) shows both higher moisture content and higher porosity than the cast concrete (w/b=0.40), which in itself must be regarded as expected. For both concretes the initial moisture content is clearly above 3 weight% and therefore regarded susceptive to fire spalling if made without pp-fibre, according to [4]. The sprayed and cast samples were wrapped in plastic from after production and during the whole curing period, until coring and preparation, hence they should not have lost much moisture. In this way the measured relative humidity (RH) should be close to reflect the effect of self-desiccation (due to chemical shrinkage), but the values are however somewhat lower than could be expected. Hence, some moisture appears to have been lost during curing and/or during sample preparation for the RH-test (but this again does not fit with the moisture/porosity results discussed above).

Results and observations from the fire tests are summarized in Table 5. The overall result may be added up to that the 50 μm fibre used in the sprayed concrete do not perform well. Improvement can however be traced for the 1.5 kg dosage, being at least the only test with the 50 μm fibre that was not intermitted - despite a high maximum spalling depth, see Figure 8. Maybe 2.0 kg could work better, but this was not tested here.

The 18 µm fibre however performs well for all dosages both for the sprayed and cast test elements. Testing according to HC-fire is the topic discussed here, and in this context the relevance of the test on the cast element with 0.5 kg dosage is unclear since it was run with a deviating fire exposure curve with 450 °C the first hour and HC-curve for 1 hour afterwards (it belongs to another investigation).

The 150 mm cast elements are very robust since only about halve this thickness is still enough to protect the insulation (keep temperature below 250 °C). So even if there were rather significant spalling (which is absolutely not the case here) one could discuss whether such an element still could perform satisfactorily. So very low fibre additions, and maybe also other fibre materials, could be relevant for future testing.

*Table 4 Moisture state and porosity before fire test, companion specimens (1 kg 18 µm fibre in both concretes)*

| Measurement | Sprayed concrete | | Cast concrete | |
|---|---|---|---|---|
| | Average | *Std. dev.* | Average | *Std. dev.* |
| Initial moisture content, weight% | 9,0 | *0,4* | 5,0 | *0,2* |
| Initial moisture content, volume% | 18,2 | *0,6* | 11,8 | *0,5* |
| Degree of capillary saturation, % | 94,1 | *0,8* | 85,6 | *3,1* |
| Suction porosity, volume% | 20,1 | *0,6* | 13,8 | *0,2* |
| Macro porosity, volume% | 5,0 | *0,6* | 3,8 | *0,5* |
| Solid phase density, kg/m$^3$ | 2698 | *8,4* | 2848 | *9,8* |
| Dry density, kg/m$^3$ | 2020 | *18,7* | 2347 | *19,7* |
| Relative humidity, % | 85,4 | *0,7* | 78,0 | *0,8* |

*Table 5 Test results/observations from the fire tests*

<table>
<tr><th>Element type</th><th>Fibre type and dosage</th><th>Test accomplish-ment</th><th>Spalling (overall)</th><th>Max. spalling depth (mm)</th><th>Status insulation</th><th>Max. temp., transition concrete – insulation (°C)</th></tr>
<tr><td rowspan="8">Sprayed (80 mm thickness)</td><td>18 µm – 1.0 kg</td><td>Completed</td><td>Small flake</td><td>33 mm</td><td>Burned in confined area outside flake</td><td>180 °C</td></tr>
<tr><td>18 µm – 1.0 kg</td><td>Completed</td><td>No spalling</td><td>0</td><td>Undamaged</td><td>110 °C</td></tr>
<tr><td>18 µm – 1.5 kg</td><td>Completed</td><td>No spalling</td><td>0</td><td>Undamaged</td><td>160 °C</td></tr>
<tr><td>18 µm – 2.0 kg</td><td>Completed</td><td>Very small flake</td><td>20 mm</td><td>Undamaged</td><td>130 °C</td></tr>
<tr><td>50 µm – 0,5 kg</td><td>Intermitted</td><td>Significant</td><td>Not relevant</td><td>Ignited, burned</td><td>Not relevant</td></tr>
<tr><td>50 µm – 1.0 kg</td><td>Intermitted</td><td>Significant</td><td>Not relevant</td><td>Burned</td><td>Not relevant</td></tr>
<tr><td>50 µm – 1.0 kg</td><td>Intermitted</td><td>Significant</td><td>Not relevant</td><td>Ignited, burned</td><td>Not relevant</td></tr>
<tr><td>50 µm – 1.5 kg (see Figure 8)</td><td>Completed</td><td>Three areas/ flakes</td><td>71 mm</td><td>Burned area outside flakes</td><td>170 °C (not relevant)</td></tr>
<tr><td rowspan="5">Cast (150 mm thickness)</td><td>18 µm – 0.5 kg*</td><td>Completed</td><td colspan="4" rowspan="5">All tests completed with no spalling and with undamaged insulation.<br><br>Maximum temperatures in the transition between concrete backside and insulation were from 70 °C to 100 °C.</td></tr>
<tr><td>18 µm – 1.0 kg</td><td>Completed</td></tr>
<tr><td>18 µm – 1.0 kg</td><td>Completed</td></tr>
<tr><td>18 µm – 1.5 kg</td><td>Completed</td></tr>
<tr><td>18 µm – 2.0 kg</td><td>Completed</td></tr>
</table>

* First 450 °C for 1 hour (Iso), then HC-curve for 1 hour

*Figure 8 After completed fire test; 50 µm - 1,5 kg, sprayed concrete element: Exposed side of element (left) and backside with insulation (right).*

## SOME GENERAL OBSERVATIONS DURING THE FIRE TESTING

For the used own, denoted "pilot own", it may be an advantage to increase the size of the element from 1.5 m x 1.5 m to at least 1.6 m x 1.6 m. In this way the tightening of the split between the test steel frame and element will be superfluous.

The used plastic spacers (for the steel mesh) expand significantly during heating. This could be seen directly as they came partly out on the backside of the element. One of the spacers is placed in the center of the element and this may explain a small flake spalling out at the exposed side in elements with no other spalling at all. Spacers of concrete should be considered.

The position of the temperature sensors could wisely be placed in the diagonal of the element, and the outer sensors not so close to the ends – to avoid measuring possible end-effects.

For the sprayed concrete elements, the insulation loosened from the backside in all tests. For future tests the fastening details should be reconsidered. For the cast elements the insulation did not loosen.


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# VAE CO-POLYMERIC BINDERS FOR WATERPROOF SHOTCRETE AND SPRAY- APPLIED MEMBRANE

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**Abstract**
Cement is a very well-known binder for concrete. Although brittle, cement offers a good degree of strength and solidity. In contrast, polymeric binders are flexible, and so can offer a broad range of complementary uses. Polymeric binders can enhance cement and concrete products, and lead to new product properties and improved performance. These properties will depend on the ratio of polymer to cement (P/C). Low P/C ratios mainly confer adhesion and cohesion, while high P/C ratios provide added flexibility.
The amount of polymeric binder employed depends greatly on the requirements regarding strength, stability flexibility and water impermeability. Polymer modification is an interesting option for achieving project goals such as waterproofing. The use of polymers to modify sprayed concrete lowers material consumption and improves spraying efficiency and durability.
The positive effect of polymers in sprayed concrete is evidenced by the technical performance of wet- or dry-sprayed concrete, as demonstrated and measured under real application conditions.
The polymeric binder used in the described applications was based on a copolymer of vinyl acetate-ethylene and effected improvements in cementitious compositions. Current studies indicate that sprayed concrete is developing in line with the market. It is versatile, it can be applied vertically and on ceilings, it is suitable for tunneling and mining, and can be used on slopes and in canals.
The waterproofing effect of polymer-modified sprayed concrete is interesting, both in terms of positive pressure directed onto the concrete and negative pressure coming from behind the concrete and was a major finding in the SUPERCON study.
The spray-applied membrane technology works as a composite shell lining and allows concrete to be saved, as both concrete layers contribute to the complete concrete lining.

**Keywords**
Shotcrete, Membrane, Polymer-Binder, Watertightness, Rebound, Fiber, Adhesion, Concrete, Copolymer

## INTRODUCTION

### Copolymer
The polymeric binder employed is based on a vinyl acetate-ethylene copolymer (VAE). The physical performance of this copolymer is controlled by the polymer chain structure and the ratio of the monomers in the composition. The VAE copolymer is based on two monomers, namely ethylene, with a glass transition temperature ($T_g$) of -93°C, and vinyl acetate with a $T_g$ of 33°C. The flexibility of this copolymer is described by its glass transition temperature: a high ethylene content yields a low $T_g$ and signifies greater flexibility at low ambient temperature. The copolymer used in this study had a $T_g$ of about -7°C. One important advantage of this type of copolymer is that there is no need for an external plasticizer, as ethylene works like an internal plasticizer and also makes for a product with a very low volatile organic content (VOC).
Basically, the product is commercially available in two forms: the liquid dispersion itself and a spray-dried dispersible polymer powder, which offers equivalent performance for the construction industry.
During hydration and drying of the cementitious composition, the copolymer generates a polymeric film. This film is not dispersible in water and is water resistant. Admixing the copolymer to a cementitious composition boosts the adhesive/cohesive bond and the flexural strength, while also lowering the risk of crack formation and slashing the extent of capillary water absorption.

### Application
Underground construction and engineering cannot exist without sprayed concrete – whether in tunnels, mines, or equivalent structures. This is a vast field and will be of strategic importance in future urban development projects. Many existing tunnels will need short-term repair work or enlargement, even as demand for new construction is huge.
While polymer-modified sprayed concrete is suitable for most wet shotcrete applications, so also is commonly used dry shotcrete.

The amount of polymeric binder employed depends greatly on the project's requirements regarding strength, stability and water impermeability. The aim of modern shotcrete application is to reduce spray rebound – which is a time-consuming, costly and environmentally challenging issue, due to loss of material and changes in the properties of the concrete. Polymer modification is an interesting option for achieving project goals such as waterproofing. The use of polymers to modify sprayed concrete lowers material consumption and improves spraying efficiency and durability.
The positive effect of polymers in sprayed concrete is evidenced by the technical performance of wet- or dry-sprayed concrete, as demonstrated and measured under real application conditions.
At the Project SUPERCON, which stands for "Sprayed sUstainable Permanent Robotized CONcrete" a tunnel lining with functional watertightness was investigated and a comparison of unmodified with polymer modified concrete, showed a clear advantage of polymer at a crack with of up to 0,4mm [1].

**Spray-Applied Membrane**
A new design for lining tunnels utilizes a "double-bonding", sprayable, waterproofing membrane based on a VAE copolymer. The double-bonding capability gives rise to a composite structure comprising an inner and outer concrete lining. A test program has been devised that simulates subjecting the composite structure to temperature cycling for the purpose of inducing recurring stress momentum at the membrane interfaces. Simulating recurring stress induction in the composite tunnel lining provides valuable information about the long-term double-bonding properties and the durability of the waterproofing membrane in a tunnel environment. An interim assessment of the membrane bonding has been conducted after more than 50 stress cycles. Measurements are ongoing.

**Fiber**
Macro synthetic fibers have been used in concrete for many years now, particularly to prevent cracks from developing due to deformation energy or drying shrinkage. The choice of fiber depends on the requirements of the application. Synthetic fibers are available in different structures and chemical compositions designed to confer high pull-out resistance via fiber surface anchorage. Suppliers of fiber technologies exploit the typical material properties when developing the optimum geometry that will produce the best bonding.
Polymeric binders are used to improve adhesion to the substrate and to lower the modulus of elasticity. Pull-out samples were prepared so that the interaction between the polymeric binder and the fiber at the interface of the lower layer of fresh, partially cured mortar could be studied and so that a determination could be made about whether polymer-containing mortar exhibits better adhesion to the fiber, thus increasing the pull-out force and the desired modulus of elasticity.

## WATER IMPERMEABILITY IN TUNNELS

The degree of waterproofness afforded in tunnels must be commensurate with their usage, and this requires a closer examination of the term waterproofness or impermeability. Common concerns revolve around defining classes and creating categories that enable, on one hand, the contractor to propose or implement an appropriate waterproofing process and, on the other, customers to specify and monitor the degree of impermeability required for their building projects [2]. The fundamental question to be addressed in the development of a construction system is whether it is samples or a building component which are to be assessed. The advantage of samples is that sampling is performed under highly controlled, reproducible conditions; upon successful testing, the sprayed-concrete sample can then be classified as waterproof.

**Application in the Tunnel Mining Access Road: Wet-Sprayed Concrete in a Salt Mine for a Period of 10 Years**
The Clara tunnel, an access route to a salt mine in Stetten (Germany), was successfully waterproofed with polymer-modified sprayed concrete. The polymer content of the concrete formulation was 7.5% liquid dispersion with a solids content of 53%. The performance of the composite material can be described by the tensile adhesion strength, shown in Table 1. EN 1504-3 is interesting because it does not view shrinkage as a single property. Instead, it considers bonding, adhesion and toothing of interfacial layers. Similar values were obtained for the shotcrete applied in a tunnel system.

*Table 1 - Tensile adhesion strength in a sprayed-concrete bonding test acc. to EN 1504-3*

| Mixture | | Reference | Polymer-Modified |
|---|---|---|---|
| W/C | | 0.46 | 0.46 |
| Grain size | Mm | 0/8 | 0/8 |
| CEM II 42.5 A-LL | kg/m³ | 400 | 400 |
| Accelerator | % | 7 | 7 |
| Polymeric binder (liquid) | % | - | 7.5 |
| Superplasticizer | % | 0.8 | 0.5 |
| Flowability (immediate) | Cm | 58.0 | 57.5 |
| Fresh mortar density | kg/m³ | 2,183 | 2,160 |
| Early strength | Penetration tests | J2 | J2 |
| Tensile adhesion strength 28 d (sprayed on concrete slab) | N/mm² | 0.45 | 1.25 |

In 2008, the unmodified shotcrete, tested here at 430 m, showed typical lime efflorescence, due to existing water penetration through small hair cracks and large flaws (see Fig. 1). After 10 years' service, an additional inspection was conducted to determine the degree of impermeability. An area of polymer-modified shotcrete stretching 400 to 500 m was inspected, and the photographic documentation shows that impermeability had not deteriorated (Figs. 2 and 3).

The typical lime deposits that formed on the surface are a good indicator of impermeability and can be classified as follows:

- Standard concrete (2008) – highly susceptible to leaks: Fig.1,
- Polymer-modified concrete (2008) – waterproof: Fig. 2,
- Polymer-modified concrete (2018) – waterproof: Fig. 3.


*Figure 1 - Tunnel wall sprayed with standard concrete (without polymer) during construction in 2008: visible water infiltration and white patches of discoloration caused by $Ca(OH)_2$, etc. Similar observations were made during inspections in 2012 and 2018; never dried fully.*

*Figure 2 - Tunnel wall sprayed with polymer-modified concrete during construction in 2008, 2 months after application: dry surface, no water migration.*

*Figure 3 - Inspection of the tunnel wall sprayed with polymer-modified concrete at 480 m: waterproof during the first inspection in 2012 and the latest inspection in 2018; dry surface.*

## WATERPROOFNESS OF WET-APPLIED SHOTCRETE

### Field test at Hagerbach Test Gallery

A new test was devised for obtaining a clearer picture of water transport from the rock side of a shotcrete application. We worked with the Hagerbach Test Gallery (VSH) to develop a test setup comprising controlled water ingress through installed pipes to produce continuous water inflow between the rock and the shotcrete [3]. The piping consisted of a vertical main pipe for the water supply and a one-meter horizontal pipe every 50 to 60 cm, for a total of 11 pipes (Fig. 4). The pipe system was fixed mechanically onto the freshly excavated natural rock cavern. Each horizontal pipe was perforated three times to guarantee good water distribution while the test was being conducted. A water-permeable, porous fabric was wrapped all around the surface to prevent the perforations from becoming permanently blocked with shotcrete. A stopcock was installed at the end of the pipes to control the presence of water after the final installation of shotcrete. A total of six piping systems in this design were installed in order to test the two different types of cement, each of which was modified with polymer at two different concentrations and, finally, the unmodified reference shotcrete. The shotcrete samples were applied next to each other in overlapping layers to prevent weak points between the applied formulations. The concrete mix was prepared, tested and applied at the Hagerbach test Gallery in Switzerland.
The humidity in the tunnel was very high, and water condensation recorded on the shotcrete surface made the actual water difficult to observe. These factors led the team to install small fan heaters to promote drying and thus allow the researchers to see which sections were affected by actual water infiltration.
The test ran for 15 weeks at 1.5 bar and revealed clear differences between the various shotcrete formulations, regardless of cement type. The reference formulations showed signs of wetting after 5 to 6 weeks, but the polymer-modified formulations (7.5% and 10.0%) did not and they looked much dryer. After 15 weeks, the pressure was increased from 1.5 bar to 6.0 bar, at which level a difference became apparent between the 7.5% and the 10.0% polymeric binder. The reference shotcrete was already wet, and the 7.5% formulation became wet at this level as well, whereas the 10.0% formulation remained dry.

*Figure 4 - Detailed image of the piping system: the vertical water supply pipe is on the far left, with the horizontal, perforated pipe for water permeation visible on the right.*

*Table 2 - Wet shotcrete formulation*

| Section No. | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| Modification | | Reference | 7.5% Polymer | 10.0% Polymer | Reference | 7.5% Polymer | 10.0% Polymer |
| VSH Test Report No. | | FBK 2 | FBK 3 | FBK 4 | FBK 5 | FBK 6 | FBK 7 |
| Normo 4 CEM I 42.5 N | kg/m³ | 400 | 400 | 400 | - | - | - |
| Fluvio 4 CEM II A-LL 42.5 N | kg/m³ | - | - | - | 400 | 400 | 400 |
| 0/1 | kg/m³ | 133 | 133 | 133 | 133 | 133 | 133 |
| 0/4 | kg/m³ | 1102 | 1102 | 1102 | 1102 | 1102 | 1102 |
| 4/8 | kg/m³ | 665 | 665 | 665 | 665 | 665 | 665 |
| Polymeric binder (liquid) | % on cement | - | 7.5 | 10 | - | 7.5 | 10 |
| W/C | Ratio | 0.47 | 0.48 | 0.47 | 0.46 | 0.43 | 0.45 |
| FM Sika Viskocrete SC 305 | % on cement | 1.2 | 0.8 | 0.2 | 1.0 | 0.7 | 0.28 |
| VZ Sikatard 903 | % on cement | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 |

*Table 3 - Test results for compressive strength; drilled cores from shotcrete spray box (diameter: 50 mm)*

| Section No. | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| Modification | | Reference | 7.5% Polymer | 10.0% Polymer | Reference | 7.5% Polymer | 10.0% Polymer |
| VSH Test Report No. | | FBK 2 | FBK 3 | FBK 4 | FBK 5 | FBK 6 | FBK 7 |
| 7 d | N/mm² | 30.2 | 28.0 | 25.0 | 37.2 | 30.1 | 23.7 |
| Standard deviation | N/mm² | 1.4 | 0.9 | 1.0 | 0.6 | 1.6 | 0.7 |
| 28 d | N/mm² | 39.1 | 37.0 | 33.1 | 38.8 | 33.3 | 29.4 |
| Standard deviation | N/mm² | 2.2 | 0.6 | 0.7 | 1.4 | 0.9 | 1.8 |

Over the course of water-pressure testing, water penetration from the rock surface through the shotcrete and to the surface of the test fields was observed.

*Table 4 - Surface wetting of shotcrete, visual assessment over time*

| Section No. | | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|---|
| Modification | | | Reference | 7.5% Polymer | 10.0% Polymer | Reference | 7.5% Polymer | 10.0% Polymer |
| Test Report No. | | | FBK 2 | FBK 3 | FBK 4 | FBK 5 | FBK 6 | FBK 7 |
| Test Time | Duration | Pressure | | | | | | |
| 1 week | 7 d | 1.5 bar | Dry | Dry | Dry | Dry | Dry | Dry |
| 2 weeks | 7 d | 1.5 bar | Dry | Dry | Dry | Dry | Dry | Dry |
| 4 weeks | 14 d | 1.5 bar | Dry | Dry | Dry | Dry | Dry | Dry |
| 6 weeks | 14 d | 1.5 bar | Wet | Dry | Dry | Wet | Dry | Dry |
| 10 weeks | 28 d | 1.5 bar | Wet | Dry | Dry | Wet | Dry | Dry |
| 15 weeks | 35 d | 1.5 bar | Wet | Dry | Dry | Wet | Dry | Dry |
| 16 weeks | 7 d | 6.0 bar | Wet | Wet | Dry | Wet | Wet | Dry |
| 20 weeks | 28 d | 6.0 bar | Wet | Wet | Dry | Wet | Wet | Dry |

**Discussion of polymer-modified shotcrete**

Polymer-modified concrete has interesting potential – both technical and commercial. Different projects have different expectations: polymer modification of wet- or dry-sprayed concrete offers many benefits, such as less rebound, good workability (pumpability, etc.), improved adhesion and cohesion, waterproofing effects, and, which is also important, easy-to-install dispensing equipment. The waterproofing effect, too, is interesting, in terms of both positive pressure directed onto the concrete and negative pressure coming from behind the concrete. Besides the immediate effects of cost reduction and time savings, long term effects can be expected as well. System costs of construction projects are growing in importance, and polymer modification helps reduce total costs considerably. This gives polymer-modified concrete a great deal of technical and commercial potential. Current studies indicate that sprayed concrete is developing in line with the market. It is versatile, it can be applied vertically and on ceilings, it is suitable for tunneling and mining, and can be used on slopes and in canals or conduits.

## TESTING OF SPRAY-APPLIED MEMBRANE

### Test specimen

The test setup is based on a cast, L-shaped concrete beam (concrete class C35/45) onto which a membrane layer is applied and then over-sprayed with concrete (concrete class > C30/37), in the manner of tunnel lining (Figure 5) [4].

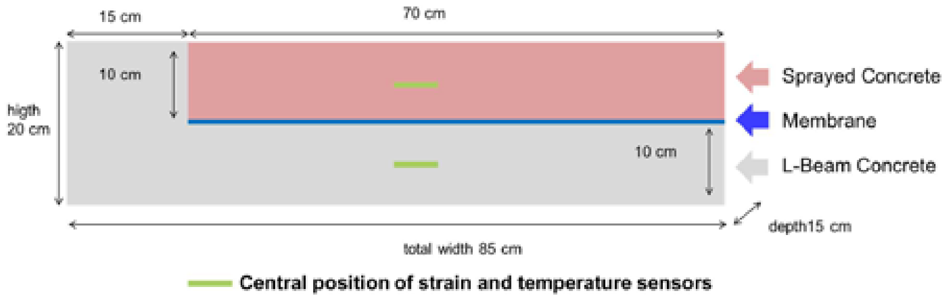



*Figure 5 - Schematic drawing of the cast L-beam as base, with sprayed membrane and sprayed concrete.*

The L-shaped concrete beam contains steel reinforcement to make the beam as rigid and immobile as possible. After preparation, the beams were cured for more than a year to allow shrinkage and any other deformations to occur. Furthermore, all beams were pre-stressed by temperature cycling, in a manner similar to the temperature cycles applied later, so as to avoid unexpected deformations during the test. All the L-beams had a rigid, massive counter-block on one side (Figure 5) connected to the reinforcement for the purpose of inducing a one-dimensional linear deformation in the upper sprayed concrete layer and consecutively a shear momentum in the membrane when the upper sprayed concrete is exposed to fluctuating temperatures.

The lowest shear momentum in the membrane is expected to occur close to the counter block and to be close to zero, but to increase with increase in distance from the counter-block. It must be expected that, when the system begins to fail, this is likely to start at the right side of the beam and the damage should migrate steadily toward the counter block. This L-beam setup has been used before to test shear stability of tile adhesive and tiles exposed to temperature changes and concurrently induced strain.

### Temperature cycles

For the study, two different temperature cycles of different temperature Δ were applied to the L-beams: a "mild" cycle involving a 15 K change and a "harsh" cycle involving a 35 K change. The reason for choosing these cycles was that, e.g. inside a tunnel or subway station, the "mild" cycle occurs quite frequently. The "harsh" cycle is intended to mimic the more demanding conditions that exist, for example, at a tunnel portal or that occur during drastic weather changes. Also, these harsh conditions would result in higher shear stresses at the membrane interface. The lower temperature of the "harsh" cycle was deliberately set to -5 °C to check how the concrete-membrane bond is impacted if the membrane temperature drops close to or below the $T_g$ of the used polymer and a concurrent shear momentum is apparent.

*Figure 6 - Cross-section of the L-beam, showing the spray-applied membrane lining in the center and the sprayed concrete on top.*

**Discussion**

Application of 50 “mild” and “harsh” temperature cycles failed to produce a system failure, e.g., debonding of the membrane. Because changes in temperature gradient and membrane reveal characteristic changes in displacement, relevant shear stress is applied to the membrane. Even at the highest applied shear stress, the results show the double-bonding capability of the membrane in its role of mediator between the primary concrete layer in the L-beam and the upper layer of sprayed concrete. Consequently, this composite shell-lining approach makes it possible to save on concrete, as both concrete layers contribute to the overall concrete lining – and not just a single thick, inner concrete shell.

Although the interface is subject to cyclical, induced stresses, a clear composite action of this setup, without any signs of delamination between layers, is observed. Testing will proceed until severe failure occurs. The durability of the membrane-concrete bond will be observed by cutting slices from some beams from time to time and performing adhesion tests on them.

## USING MACRO SYNTHETIC FIBERS IN SPRAYED CONCRETE

### Goal of the Study

As is usually the case with standard concrete applications, the fibers are added to the concrete as it is being mixed. This can be either a wet- or dry-concrete mix design (fiber-reinforced concrete or FRC) [5]. FRC’s functional principle is to mechanically anchor the polymer fibers in the dense matrix of the binder or concrete. The purpose of the synthetic fibers is to:

- create a modelled surface structure
- achieve high pull-out resistance via mechanical anchorage.

Suppliers of macro-fiber technologies draw on typical material properties to develop the best geometry for achieving the optimum composite structure.

*Table 5 - Fiber Type*

| Fiber Type | Fiber Weight in g | Typical Dosage / m³ (Manufacturer Specifications) | Number of Fibers / m³ |
|---|---|---|---|
| Polymer (BarChip 56) | 0.0274 | 5 | 182,482 |

The use of fibers, and particularly chopped fibers, in construction materials is current practice. Chopped polymer fibers are normally used to protect the concrete shell in the event of a fire. Polymeric binders are used for improving adhesion to the substrate and for lowering the modulus of elasticity. This new test method investigates the effect of polymeric binders on the bonding of fibers in concrete. More specifically, it determines whether polymer-containing concrete demonstrates better adhesion to the fiber, thereby increasing the pull-out force.

## UNIAXIAL TESTING OF FIBER PULL-OUT FORCE

### Experimental setup in the laboratory

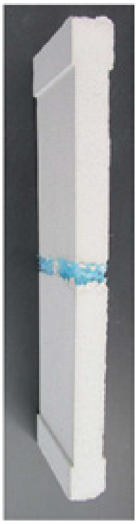
A measurement setup on a laboratory scale could improve the quality of the data available on the fiber material's basic properties and is an objective of this work. In developing products for the application of pumped concrete, it is essential to have a test method that yields comparative results. Comparative means being able, with the lowest-possible standard deviation, to differentiate between various products or formulations. This approach also has economic advantages, because the fabrication of large test specimens is much more time-consuming and cost-intensive than reducing the test to a laboratory scale [5, 6].

One of the challenges encountered in the development of the test method was finding a suitable mold for making the test piece. So-called "dog bone" test pieces are well-known, but a drawback associated with them is that they must be mounted with a high contact pressure, which means that damage in the clamping-jaw region cannot be ruled out. From our experience with other applications, we furthermore knew that the modified area – a fiber in this case – is reinforced and that the risk of the fracture occurring outside the fiber-reinforced zone was high.

Due to the material properties and the forces that were expected to arise, a reduction in the number of necessary variables to a simple test method based on EN 14891 was an obvious choice [7]. The aim of developing the test method itself was to use as few fibers as possible initially, and ideally only one. Since we were working with macro fibers (length, L = 30 - 60 mm), we were able to position these centrally within the test piece, thereby making a uniaxial tensile test possible.

As the anchorage of the fiber and not the mechanical strength of the concrete material was to be evaluated, the test piece was split during preparation. A 10-mm-thick PE foam pad was used for the splitting. This pad served as a divider between the two test-piece halves to be poured and also made it possible to position the fiber exactly in the center. Based on EN 14891, the test piece had the following dimensions: width 160 mm, height 40 mm and thickness 12 mm (15 mm at the ends). The fiber was located in the center at a height of 20 mm and depth of 6 mm in each case. The mounted length of the fiber half in the test piece is reduced by 5 mm per test-piece half, due to the foam. One mold was able to accommodate six test pieces, which could thus be produced in a single batch at the same time. This has a positive effect both on the statistical evaluation and on error prevention.

The test piece described in EN 14891 has a T-shaped widening at the ends that allows the clamping jaws of the tensile tester to be fitted with minimum contact pressure. The tensile test conducted subsequently was performed at a rate of 5 mm/min and the force was measured in Newtons (N).



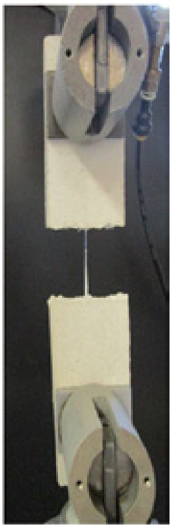


*Figure 7, left - Test piece after stripping; the fiber is embedded in the center of the test piece and stabilized by polyethylene foam.*
*Figure 8, right - Fiber pulled out of the test-piece halves toward the end of the measurement.*

## TEST MIXTURE

### Results for the test mixture

To simplify the test conditions, no coarse aggregates were used. The compressive strength was measured on identical test pieces, which were also used in the tensile tests. The dimensions were: width 40 mm, height 40 mm and depth 12 mm. This procedure reflected the strength generated under the test conditions. The mixture was prepared with the addition of water in a Toni mixer. The fine concrete mixture was premixed dry and all water-based additives, such as the polymeric binder and any other auxiliary materials used, were added to the water and then poured into the prepared mold (A) and stored at 23 °C and 50% relative humidity until the test (1 day in the mold (A) and then stripped).

*Table 6 - Composition of the fine concrete mixtures*

| Formulation for the Fiber Pull-Out Test | Quantity | Without Polymer | With Polymer |
|---|---|---|---|
| Portland cement: Milke CEM 42.5 N | g | 400 | 400 |
| Quartz sand: H 33 (grain size 0 - 0.5 mm) | g | 1,000 | 1,000 |
| Carbonate filler: Omyacarb 5 GU (5 µm / $D_{50}$%) | g | 775 | 775 |
| Thickener: Kelco-Crete DGF | g | 0.1 | 0.1 |
| Polymeric binder: VAE (solids content: 50%) 10% of cement | g | 0 | 40 |
| Plasticizer: Melflux 2651 F (BASF) | g | 4 | 4 |
| Total | g | 2,179.1 | 2,219.1 |
| Water/cement ratio (w/c, water from polymer taken into account) | | 0.775 | 0.775 |
| Compressive strength on 12 mm x 40 mm x 40 mm | N/mm² | 19.28 ± 2.71 | 19.51 ± 2.81 |

### Pull-out force for synthetic fiber

The reference rupture value for the single fiber, tested in the same test equipment, was 275.72 ± 0.08 N; this was the maximum force to be achieved in the test.

*Table 7 - Measured Values*

| Fiber Type | | BarChip 56 | | |
|---|---|---|---|---|
| Modification | | | Without polymer | With polymer |
| $F_{max}$ (force in newtons) | F | N | 166.83 | 210.03 |
| $F_{max}$ (path at $F_{max}$) | W | mm (m) | 2.8 (0.0028) | 3.3 (0.0033) |
| Standard deviation (newtons) | s | N | 22.98 | 21.32 |
| Number of test pieces | n | | 6 | 6 |
| Initial energy (force x path: F x W x $F_k$) $F_k$= area correction factor (0.6) | J | J | 0.280274 | 0.415859 |
| Improvement (without polymer = 1) | | | 1 | 1.48 |

### Discussion

The tests show that there was strong interaction between the polymer fibers and the VAE polymer used. It was possible to increase the relative pull-out force from 1 for the reference concrete to 1.48 (+50%) for the polymer

concrete. The polymeric binder has a high affinity for the fiber material, thereby enabling the production of a composite material that is clearly superior to mechanical anchorage to, or embedding in, the concrete. If composites are created by adding 10% polymeric binder, the system can also be considered to be more robust. Systems that are more robust are user-friendly and lead to better structures. Although the fine concrete formulation used does not claim to represent a compressed concrete, it does, in a direct comparison, show what can be expected from a weakly compressed concrete.
The use of polymer contents of less than 10% had a considerable influence on the pull-out force for the synthetic fibers. A considerable increase was achieved both at peak height and on the path to reaching Fmax, which means that the test piece absorbs more energy.

## CONCLUSION

The polymeric binder used in the described applications was based on a copolymer of vinyl acetate-ethylene and effected improvements in cementitious compositions.
The waterproofing effect of polymer-modified sprayed concrete is interesting, both in terms of positive pressure directed onto the concrete and negative pressure coming from behind the concrete.
The spray-applied membrane technology works as a composite shell lining, and allows concrete to be saved, as both concrete layers contribute to the complete concrete lining.
In fiber-reinforced concrete, the polymeric binder has a high affinity for the fiber material, thereby enabling the production of a composite material that is clearly superior to mechanical anchorage to, or embedding in, the concrete.

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# HIGH PERFORMANCE SHOTCRETE FOR TUNNEL RENEWAL

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Keywords: high-performance-shotcrete, refurbishment, waterproofing.

Abstract:

Italy's transport infrastructure is well-developed with a total road network of 512,400 km and a rail network spanning 20,048 km. However, it ranks below the EU average due to outdated infrastructures, part of which consists in tunnels in need of structural refurbishment.
The goal of the program named "Tunnel Renewal Strategy" by the Italian Motorway Authority (Autostrade Per l'Italia, ASPI) is to extend the design life of tunnels in the motorway network by another 50 years through the creation of new linings, designed to replace the original ones, while also ensuring adequate performance in the event of seismic activity. The first phase of the design of the pilot-project aimed at testing the typological interventions and specific innovative technical solutions, involved a series of tunnels, built between the end of the '50s and the beginning of the '70s distributed throughout the nation's motorway network.

During the preliminary phase, the Italian Motorway Authority considered standardized proposals based on different methods for the refurbishment works, including cast in situ lining and precast concrete elements. In this case however, Mapei UTT has proposed an innovative technical solution which combines an innovative waterproofing installation approach and high-performance shotcrete, which in consequently allows for faster refurbishment works without affecting the final technical performance of the permanent tunnel lining.
This new refurbishment method, named Single Shell WSL, has been successfully employed as a standardized solution in two case studies among the pilot-interventions selected by the Italian Motorway Authority (ASPI), with high regard for the requirements in terms of technical performance, ease-of-use, safety, quickness and most importantly, durability.

This paper will describe the innovations introduced with the new waterproofing installation method in combination with the high-performance shotcrete mix design, while highlighting the advantages in terms of economic efficiency of the entire tunnel refurbishment process.

## 1. INTRODUCTION

Italy's road and railway infrastructure is a critical component of its transport system. The country boasts a road network of over 512,400 kilometers, ranking 18th globally.
The railway system, with over 20,000 kilometers of active lines, plays a substantial role in domestic transport. Italy operates an extensive high-speed railway network of over 921 kilometers, making it the fourth largest in Europe. Despite these advancements, 80% of goods and passengers still travel by road.
Maintenance of the infrastructure requires significant investment, with billions spent annually on upgrades and maintenance. Moreover, due to its geographical and orographical characteristics, the Italian motorway network is considered the most complex in Europe and among the most heavily used. Most of the network - which saw its birth in 1924 with the inauguration of the first 40 km of the Milano Laghi - was in fact built between 1960 and 1980.

Along the country's strategic backbones, there are 595 tunnel crossings, more than 70% dating back to before 1975, which today are in need of interventions that are not limited to mere conservation but include a broader programme of upgrading and regenerative maintenance.

### 1.1. Tunnel Renewal Strategy

Starting in 2020, the Italian Motorway Authority, Autostrade per l'Italia, henceforth referred to as “ASPI”, has launched a major assessment plan to extend the design life of tunnels by 50 more years. This programme, named Tunnel Renewal Strategy (TRS) defines the strategic approach of ASPI towards in facing the renovation plan for the entire motorway network, specifically referred to tunnels (about 600 in total throughout the entire network).

This required, in the first place, an overall study carried out through specialised in-depth surveys, with the specific task of increasing and improving the information heritage on the infrastructural assets. Such revised and coherent information database would then be used to optimize both management and maintenance of the infrastructure network, as well as the definition of an innovative modernisation programme.
The ultimate goal is then to define a series of standardized interventions in order to quicken the refurbishment process for the entirety of the tunnels selected by the programme.

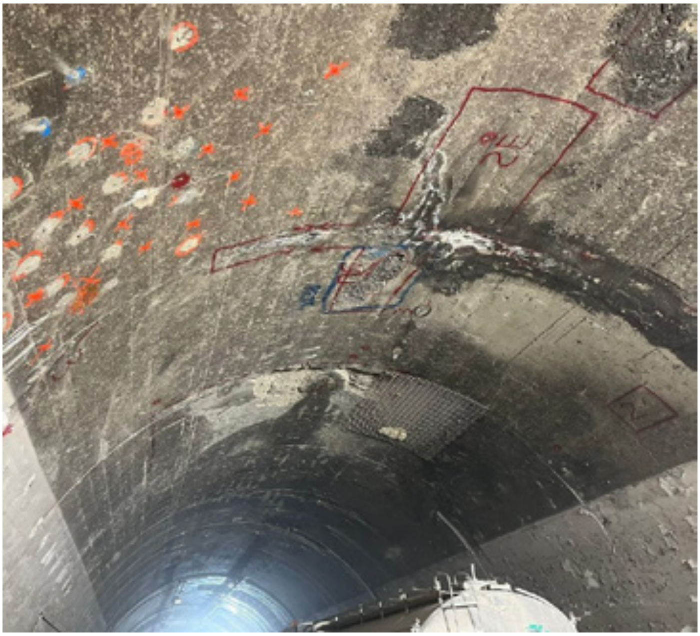

*Figure 1: example of the degradation state of one of the tunnels in need of restoration*

The design solutions are based on the realisation of new linings which can be either structurally independent, or connected to the original ones, partially to be demolished, without foreseeing any noticeable reduction in the internal section of the tunnels. The demolition of the original claddings is planned and implemented to the minimum extent necessary to allow the insertion of a new structural shell, safeguarding the integrity of the remaining portion.

To this end, one of the main conditions identifying the application context is the stress state of the existing lining. Demolition, when necessary, is therefore carried out using low-impact technological solutions, such as milling or hydrodemolition. In addition, specific additional protective interventions defined according to the contexts can be envisaged. For example: radial fibreglass nailing, cement mix injections to restore structural continuity, void fillings on the rear shell.

### 1.2. The “Manfreida” tunnel

Among the tunnels highlighted by the programme, this paper highlights the refurbishment works executed in the “Manfreida” tunnel using an innovative solution Studied, developed and engineered with a fruitful cooperation between MAPEI, Highway authority and Tecne.
The "Manfreida" Tunnel (left tube), located on the A26 Genoa-Gravellona Toce motorway, in the direction of Genoa, was subject to partial demolition and reconstruction of the final lining along the entire length of the tunnel, which is about 776 metres.

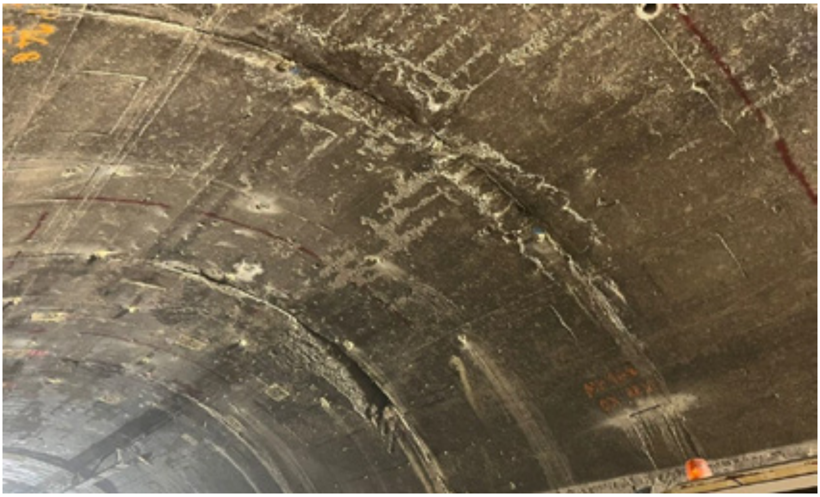

*Figure 2: state of degradation of the "Manfreida" tunnel.*

The general location in which the above-mentioned tunnel is located means that an additional amount of attention must be paid to the duration of repair work. Limiting the time window for the total closure of the tubes to the minimum possible was a priority from the first planning stage.

The work involved demolishing the final linings by milling over the entire length of the intrados, with thicknesses varying from 20 cm to 40 cm, depending on the original geometry of the existing linings.
Another essential requirement for the structural renovation of tunnels is thus the creation of a waterproofing layer, as well as the use of an inner lining made of a shotcrete mixture with high mechanical and physical performance, developed in accordance with ITA WG 12 Permanent Sprayed Concrete Lining guidelines.

## 2. MAPEI UTT’s SINGLE SHELL WSL

After an examination of the project’s requirements, Mapei UTT proposed an integrated solution called Single Shell WSL. In response to the needs expressed by the customer, the Single Shell WSL system allows the incorporation of a rapidly installable drainage waterproofing layer with a high-performance shotcrete inner lining.
The solution is composed by the following parts:

1. Milled substrate
2. Mapeplan Contact V fixing strips
3. Mapeplan 3S waterproof membrane
4. Structural reinforcement (if required)
5. High-performance shotcrete lining.

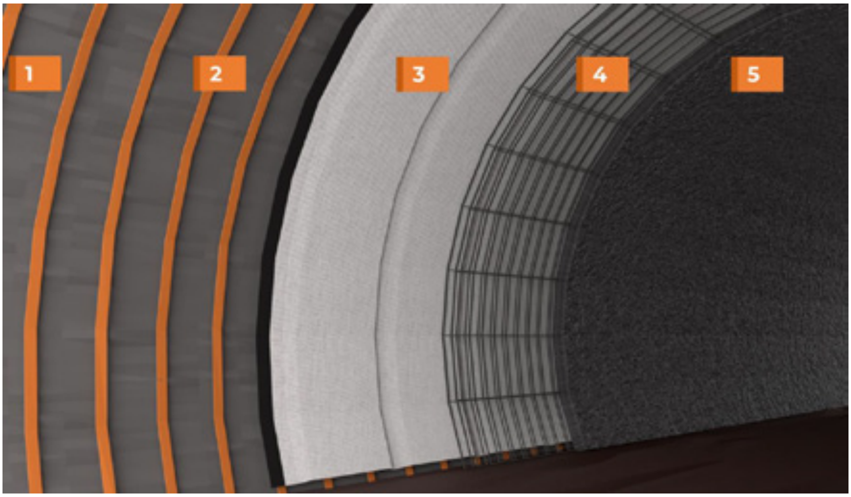

*Figure 3: Single Shell WSL layering scheme*

The interaction between the waterproofing membrane and the shotcrete lining is possible thanks to the composition of the membrane itself.

### 2.1. Waterproofing layer

Mapeplan 3S is in fact composed of a central waterproofing layer in TPO, pre-laminated on both sides with geotextile fabric.
These geotextile layers on the membrane itself have several advantages; first of all, the waterproof layer does not need a protective layer on the extrados, between the pre-existing substrate and the membrane itself, which makes it easier for the construction site, which only has to qualify and logistically manage one type of membrane.
The geotextile layer placed on the extrados side of the synthetic membrane also allows for a very rapid preliminary installation of the waterproofing system: the laminated layer of non-woven fabric with a higher grammage, mechanically adheres to the Mapeplan Contact V fixing strips, supporting the entire membrane and allowing it to be adapted to the geometry of the tunnel quickly and safely.


*Figure 4: installation of the Mapeplan 3S membrane by adhesion with the Mapeplan Contact V fixing strips*

The operative teams of the pilot site were able to experience first-hand the improvements in the speed of execution of the installation. An increase in the time required to install the waterproofing layer of approximately 50 % was observed when compared to the traditional method, in which the waterproofing membrane is initially fixed to the substrate via spot welding on PVC buttons.

One of the reasons for the quickness of installation lies in the hook-and-loop fastening method through which the Mapeplan Contact V fixing strips – previously nailed to the hydro-demolished substrate – allows for the main membrane to be affixed to the substrate simply by being physically appointed to it. More specifically, the installation of the Mapeplan 3S pre-bonded waterproofing membrane is carried out by fixing its one side which is pre-bonded with the heavier-weight geotextile, towards the extrados, in direct contact with the fixing strips.

The adjacent membranes are laid and secured to the fixing strips, and are then welded to each other via hot air welding on the ridge to ensure continuity of the waterproofing layer.
In accordance with the drainage requirements of the solution, the Mapeplan Drainage Profile is also laid, containing the drainage pipe surrounded by gravel to optimize water runoff. The water will flow by gravity into the pipe prepared inside the profile, which will then transport it to the outside of the tunnel.

### 2.2. Shotcrete layer integration

The geotextile layer on the interior face of the membrane allows for an excellent adhesion between the concrete inner lining and the waterproofing layer. This therefore allows for shotcrete inner lining layers in favor of the commonly used in-situ lining.
The advantages of shotcrete once again match the needs of this type of structural repair work: the absence of formwork brings several advantages in terms of logistics, while the very nature of shotcrete allows it to adapt to irregular surfaces that are difficult to accommodate with the formwork casting system, and also avoids the possibility of cold joints in the cladding itself.

This goes in addition to the intrinsic advantages of shotcrete in terms of execution time: the shotcrete technique is probably the most versatile among those available for the construction of reinforced concrete structures, as it can be applied on any surface, allows unlimited shape possibilities for structures with complex geometric configurations, and can be applied where traditional casting presents difficulties in execution; moreover, a shotcrete layer generally entails shorter construction times compared to in-situ concrete, especially considering the construction time required for the use of the formwork.

Shotcrete offers a rapid development of mechanical performance and is, moreover, designed as a permanent final coating. In light of these characteristics, it has the great advantage of developing high mechanical strengths from the very first hours after application, thus guaranteeing greater freedom of work in the area to be treated; in fact, during the first 24 hours, the development of mechanical strengths must fall in the area above the J2 curve, determined by the UNI EN 14487 (Figure 5) standard, thus already after one hour the shotcrete must have reached a mechanical strength of approximately 1 MPa.

Ultimately, given the particularly strict project requirements, a special grade shotcrete mix design has been developed through attentive choice of every single component. The project's specifications asked for an internal lining with a minimum compressive strength of 45MPa at 90 days, with good ductility, and anti-spalling behavior in case of fire.
For this reason, the mix design was characterized by a low w/c ratio (< 0,5) and it included CEM IV cement, which provides additional sulphate resistant capabilities to the concrete matrix. In order to account for the workability issues, a workability extending admixture was added into the mix, which in turns allowed for a workability time of up to 3 hours.

The mix also included both macro and micro synthetic fibers. While macrofibers provide a better mechanical performance and post-cracking reaction of the hardened mix, microfibers were also included to improve its fire reaction by means of anti-spalling action.

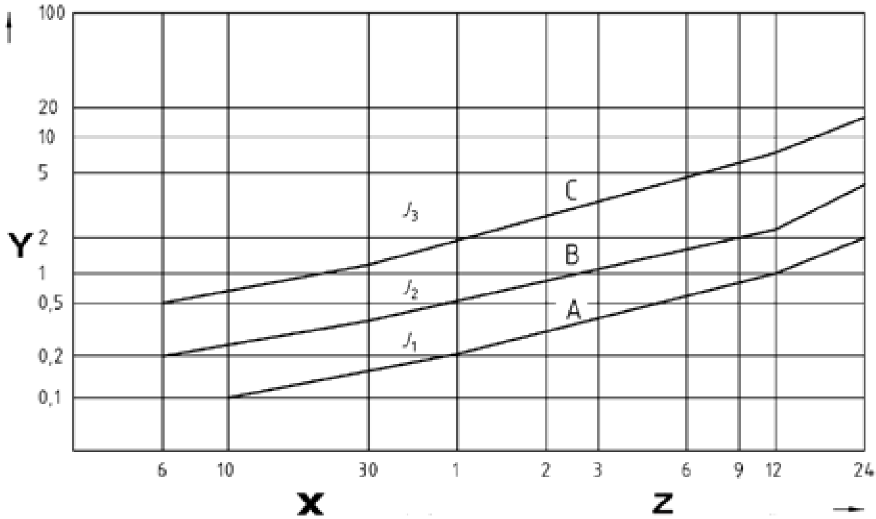


*Figure 5: compressive strength development curves in the first 24 hours, according to EN 14487-1.*

The shotcrete application was carried out by means of a "shotcrete pump" with a piston pump, with automatic dosage adjustment of the setting accelerator (approx. 7.5 %) depending on the concrete flow rate.
The application procedure of the shotcrete adhered to certain basic rules in order to optimize laying times, reduce rebound waste and achieve homogenous layering. In particular, for each zone of approximately 2-3 metres in length, the shotcrete was initially sprayed at the foothold of the rebar cage, until part of the reinforcement was incorporated for the first 1.5-2.0 m of the pier, while the upper part was gradually filled in by successive passes with thicknesses not exceeding 10 cm each.
As far as the application sequence was concerned, it was very important to apply a thin layer of shotcrete ontothe virgin surfaces of the waterproofing membrane, especially in the canopy, so that there would be a preliminary interpenetration of the conglomerate on the surface of the exposed geotextile. On-site experience in this direction has shown reduced waste when building up the shotcrete thickness in areas previously treated with this preliminary step.

*Figure 6: completely installed waterproofing layer and rebar arches (left), detail of structural connection passing through the waterproofing layer (center), completed shotcrete lining in the Manfreida Tunnel (right).*

## 3. SHOTCRETE AND WATERPROOFING LAYER INTERACTION

It should be noted that the geotextile fabric layer on the interior face of the waterproofing membrane provides excellent adhesion between the drainage layer itself and the shotcrete lining. Looking into a way to practically measure said cohesion effect, as well as its consequent advantage on the overall solution, a small-scale trial test has been carried out in Mapei R&D laboratories.

### 3.1. Shotcrete – membrane adhesion tests

In order to measure the adhesion development values of the shotcrete on the Mapeplan TPO 3S 21 membrane, 60x60x10cm shotcrete sample-boxes were sprayed, on the bottom of which a sheet of the membrane was placed. During the application phases of the shotcrete on the surface of the tunnel, these boxes were filled with the same projection technique, reproducing for all practical purposes the real application conditions.
The shotcrete mix used is the same as the one employed in the Manfreida tunnel, developed with the collaboration of UNICAL technicians and batched by UNICAL in Genoa.

#### *3.1.1. Adhesion test procedure*

Borrowing the adhesion measurement method from the EN 1542 standard, suitable notches were glued onto the surface of the membrane. The perimeter of the notch was then isolated from the surrounding membrane by cutting with an angle grinder equipped with a concrete cutting disk.

The notches were then subjected to a pull-off force in order to measure the adhesion to the membrane. The equipment employed for the tests conforms to that described in chapter 4.11 of the EN 1542 standard, including a digital dynamometer (Figure 7).

*Figure 7: notches glued to the external (extrados) surface of the waterproofing membrane on the shotcrete sample-box (left), digital dynamometer used during the adhesion strength (pull-off) test (right).*

#### *3.1.2. Adhesion test results*

Adhesion values were examined at 5, 10, and 24 hours from the moment of shotcrete application in the sample-box. Results of the median value among three pulls for each timeframe are expressed in Table 1.

| Time passed | Adhesion value |
|---|---|
| 5 hours | 0,11 MPa |
| 10 hours | 0,77 MPa |
| 24 hours | 0,95 MPa |

*Table 1: Adhesion test results, median value*

The above adhesion values show a very good development over time: at just about 5 hours, the shotcrete has incorporated the filaments of the geotextile fabric within it, thus offering a strong surface adhesion; the latter becomes very high already within 24 hours after projection, with values nearing 1 MPa.

## 4. CONCLUSION

The paper described the method proposed by Mapei UTT to meet the peculiar requirements of the re-furbishment works in motorway tunnels along the Italian infrastructure network. The combination of a quick-installing waterproofing and a shotcrete structural lining instead of in-situ cast concrete, helped speed up the entire refurbishment works.
Mapei R&D as well as Mapei technical service specialists worked closely with both the specialized company and the client, in order to make sure that the proposed method would be able to correctly adapt to the needs of both the pilot project and similar future cases.
The method, as well as the products proposed for these works have been successfully employed in a pilot project, which proved both the effectiveness of the system, as well as its suitability in responding to the project's requirement in terms of quickness of execution.
Mapei Single Shell WSL is currently in use in more similar cases, denoting a great potential to further facilitate and speed up the consolidation of existing tunnels.

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# INFLUENCE FROM AGGREGATES ON FRESH PROPERTIES AND RHEOLOGY OF SPRAYED CONCRETE

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**Keywords:** recycled concrete aggregates, fresh concrete properties, concrete rheology, sprayed concrete

## Abstract

One of today's main focusses in the construction industry is to produce and use more sustainable concrete mix designs. This is done by either producing a low carbon concrete with lower amount of Portland clinker, or by utilizing recycled materials, mainly aggregates. Lower amount of Portland clinker by either replacing cement with alternative binders or by using cements with higher amounts of supplementary cementitious materials can lead to significant lower early strength and changed rheology. Recycled materials as part of the aggregates face the same issues, as it can lead to challenging rheology and reduced strength. To make a sprayed concrete perform well, it is crucial that the fresh concrete is flowable, easy to pump and that the concrete is easy to atomize in the nozzle, to ensure the spraying accelerator is mixed well and evenly distributed into the sprayed concrete. This study analyses the properties of fresh sprayed concrete and how different aggregates and admixtures can affect the sprayed concrete in the fresh state.

## 1. INTRODUCTION

Concrete is the most used construction and building material in the world [1], and is responsible for around 8% of the worlds $CO_2$ emission [2]. To produce and use a more sustainable concrete mix design the industry can use a low carbon concrete, where the amount of Portland clinker has been reduced, or utilize recycled materials, mainly as aggregates. Lower amount of Portland clinker can be accomplished by either replacing cement with alternative binders or by using cements with higher amounts of supplementary cementitious materials (SCMs). Research has shown that this can lead to reduced early strength, as well as changed rheology, dependent on the type of SCM [3].

Even if the Portland cement is the main contributor to $CO_2$ emission from concrete production [4], aggregates account for nearly 55-80% of the concrete volume and will therefore have a great influence on the environment as well as the sustainability of the structures. As demolition of old and construction of new structures have increased due to rapid industrialization and urbanization, the amount of waste and the demand for aggregates has increased [5]. Extraction of these raw materials cause irreversible effects on the environment, for instance agricultural losses. By substituting parts of the natural aggregates with recycled aggregates, it is possible to both utilize waste generated by the construction sector and reduce the consumption of virgin aggregates. However, replacing natural aggregates with recycled materials can lead to challenging rheology [4] and reduced concrete strength [6]. The porosity of recycled concrete aggregates is higher than natural aggregates. As a result, a concrete with recycled concrete aggregates will have reduced workability compared to a concrete with only natural aggregates, if the water-cement-ratio is the same [5]. Solving these challenges will contribute to making it easier to use a more sustainable concrete.

To measure the workability of the concrete, the slump test is the most used method on site. However, it may not provide sufficient information about concrete flow behavior, and rheological measurements can be better suited for this purpose [7]. Several models can be used to describe rheology of yield-stress fluids, but the Bingham model is typically used for concrete. A Bingham fluid can be expressed by the following equation: $\tau=\tau_0+\mu\gamma$, where $\tau$ is the shear stress, $\tau_0$ is the yield stress, $\mu$ is plastic viscosity and $\gamma$ is the shear rate [8]. The yield stress, $\tau_0$, is the applied stress needed to initiate flow and is according to Roussel et al. [9] “the most important parameter” for most concrete applications, as it is related to whether the concrete will flow or not under an applied stress [9].

In Norway, sprayed concrete differs from ordinary concrete in that it only contains aggregates with maximum particle size of 8 mm. Unlike ordinary concrete, sprayed concrete is compacted by being hurled against a surface and is not vibrated. In addition to the admixtures used in ordinary concrete, alkali free sprayed concrete accelerators (AFAs) are also used in sprayed concrete. To make sure that the sprayed concrete performs well, it is crucial that the fresh concrete is flowable, easy to pump and that the concrete is easy to atomize in the nozzle. This is crucial to ensure that the spray accelerator is mixed well and evenly distributed into the sprayed concrete [10].

*NS-EN 206:2013+A2+NA: Concrete specification, performance, production and conformity* (EN 206 + NA) is the current national annex of the concrete production standard in Norway and describes the limitations regarding the use of recycled aggregates in concrete. According to EN 206 + NA, 20% of the aggregates in fraction 0/4 mm for concrete with strength class up to C30/37, and 40% of the aggregates in fraction 4/32 mm for concrete with strength class up to C35/45, can be replaced by recycled aggregates. However, according to EN 206 + NA, a higher percentage can be replaced “where this is specified in the concrete specification” [11].

The main objective of this study is to investigate the fresh properties of concrete where parts of the natural aggregates have been replaced with recycled concrete aggregates. A higher fraction of recycled aggregates than the national annex of EN 206 in Norway allows will be tested. This is to investigate if a concrete with more recycled aggerates than what is currently allowed is possible to use. In the experimental work the effect of the recycled concrete aggregate replacement rate, and different concrete admixtures will be investigated. Rheological measurements and workability of the concretes, as well as the early and late compressive strength shall be established through the experimental work. These results will then be used to discuss how the recycled concrete aggerates and admixtures affect the sprayed concrete in the fresh state.

## 2. EXPERIMENTAL

### 2.1 Test materials and conditions

The concrete mixes were produced at Mapei’s facility, with Standard cement FA, CEM II/B-M 42.5R from Heidelberg Materials (Std. FA) and silica fume. Std. FA is the single most used cement for concrete production in Norway, and the far most used cement for sprayed concrete production in Norway. It is mandatory to include at least 4% silica fume in Norwegian sprayed concrete. The concrete mix designs produced were of strength class C35/45, with water/binder-ratio (w/b) of 0.42. This is supposed to be maximum w/b = 0.45 after spraying, when water from AFA is included in the w/b.

The natural aggregate was 0/8 mm sand from Heidelberg Materials and the recycled concrete aggregate (RCA) was 0/8 mm recycled crushed concrete. The recycled concrete aggregates were provided by *Nordland Betong*, a concrete producer in Bodø, Norway and were most likely a mix of C35/45 and C30/37.

Re-Con AGG 100 is an acrylic polymer-based plasticizer/water-inhibitor developed by Mapei for concretes containing recycled aggregates, as it decreases the water absorption into the porous recycled materials. Re-Con AGG 100 must be used in combination with superplasticizers. The references, i.e. the mixes without Re-Con AGG 100, were all tested with the same dosage of superplasticizer. For the mixes with Re-Con AGG 100, the Re-Con AGG 100 dosage were adjusted, and the superplasticizer dosage was reduced, so that the start flow was similar to the respective reference. For instance, it was desired that the concrete mixes with 50% recycled aggregates with and without Re-Con AGG 100 had similar start flow. All admixture dosages are stated as the percentage of the cement weight. The initial test matrix is presented in Table 1 below.

*Table 1: Initial test matrix for the experiments.*

| Mix ID | Aggregates | Superplasticizer | Re-Con AGG 100 | Accelerator |
|---|---|---|---|---|
| 1.1 - Reference | 100% natural | 0.9% | | |
| 2.1 - Reference | 80% natural + 20% RCA | 0.9% | | |
| 3.1 | 80% natural + 20% RCA | 0.7% | 0.7% | |
| 4.1 - Reference | 50 % natural + 50% RCA | 0.9% | | |
| 5.1 | 50 % natural + 50% RCA | 0.8% | 0.9% | |
| 6.1 | 50 % natural + 50% RCA | 0.7% | 0.9% | 0.5% |

After evaluating the compressive strength results, the assumed absorbed humidity values for the RCA used in the mix design were suspected to be incorrect. It was therefore decided to repeat the mixes with 50% recycled concrete aggregates, presented in Table 2, but with lower absorbed humidity values.

*Table 2: The additional tests with 50% RCA, but with lower absorbed humidity values.*

| Mix ID | Aggregates | Superplasticizer | Re-Con AGG 100 | Accelerator |
|---|---|---|---|---|
| 4.2 - Reference | 50 % natural + 50% RCA | 0.9% | | |
| 5.2 | 50 % natural + 50% RCA | 0.7% | 0.9% | |
| 6.2 | 50 % natural + 50% RCA | 0.7% | 0.9% | 0.5% |

## 2.2 Procedure and apertures

The concretes were mixed in a free fall mixer, and the mix design presented in Table 3 was used.

*Table 3: Mix design for the concretes mixed during the experiments.*

| Aggregate type | 0/8 mm HM sand [kg/m$^3$] | 0/8 mm RCA [kg/m$^3$] | Std. FA [kg/m$^3$] | Silica fume [kg/m$^3$] | Total water [kg/m$^3$] |
|---|---|---|---|---|---|
| 100% natural | 1600 | 0 | 460 | 20 | 210 |
| 80% natural + 20% RCA | 1280 | 320 | 460 | 20 | 210 |
| 50% natural + 50% RCA | 800 | 800 | 460 | 20 | 210 |

All mixes were mixed for 7 minutes and in accordance with internal mixing procedure.

To investigate the fresh properties, the slump and flow, as well as yield stress of the concretes were measured. The slump and flow were measured right after mixing and then every 30 minutes for 2 hours. Yield stress was measured right after mixing and after 1 hour by using a viscometer developed by former ConTec in Reykjavik, Iceland. In addition to fresh properties the compressive strength was measured. 100x100 mm cubes were casted of each concrete mix and tested after 24 hours, 7 days and 28 days.

## 3. RESULTS

### 3.1 Fresh properties

Table 4 shows the fresh properties of the different concrete references, i.e. the mixes without Re-Con AGG 100, but with different percentage of natural aggregates replaced with RCA.

*Table 4: Fresh properties obtained from tests with the concretes without Re-Con AGG 100 with different amount of RCA.*

| Mix ID | Aggre-gates | AGG 100 [%] | Rheology, $\tau_0$ [Pa] | | Workability, slump/flow [mm/mm] | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 0' | 60' | 0' | 30' | 60' | 90' | 120' |
| 1.1 REF | 100% natural | x | 162 | 226 | 270/570 | 250/500 | 250/450 | 220/380 | 160/300 |
| 2.1 REF | 80% natural + 20% RCA | x | 167 | 321 | 250/470 | 250/420 | 220/370 | 145/290 | 60/200 |
| 4.1 REF | 50% natural + 50% RCA | x | 147 | 385 | 260/500 | 240/420 | 180/355 | 120/260 | 50/200 |

Table 5 shows the fresh properties of the concrete with 100% natural aggregates and the concretes with 80% natural aggregates and 20% RCA with and without Re-Con AGG 100.

*Table 5: Fresh properties obtained from tests with 80% natural aggregates and 20% RCA, compared to the reference with 100% natural aggregates.*

| Mix ID | Aggre-gates | AGG 100 [%] | Rheology, $\tau_0$ [Pa] | | Workability, slump/flow [mm/mm] | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 0' | 60' | 0' | 30' | 60' | 90' | 120' |
| 1.1 REF | 100% natural | x | 162 | 226 | 270/570 | 250/500 | 250/450 | 220/380 | 160/300 |
| 2.1 REF | 80% natural + 20% RCA | x | 167 | 321 | 250/470 | 250/420 | 220/370 | 145/290 | 60/200 |
| 3.1 | 80% natural + 20% RCA | 0.7 | 118 | 149 | 250/480 | 260/500 | 250/460 | 250/420 | 240/440 |

Table 6 shows the fresh properties of the concrete with 100% natural aggregates and the concretes with 50% natural aggregates and 50% RCA with and without Re-Con AGG 100.

*Table 6: Fresh properties obtained from tests with 50% natural aggregates and 50% RCA, compared to the reference with 100% natural aggregates.*

| Mix ID | Aggre-gates | AGG 100 [%] | Rheology, $\tau_0$ [Pa] | | Workability, slump/flow [mm/mm] | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 0' | 60' | 0' | 30' | 60' | 90' | 120' |
| 1.1 REF | 100% natural | x | 162 | 226 | 270/570 | 250/500 | 250/450 | 220/380 | 160/300 |
| 4.1 REF | 50% natural + 50% RCA | x | 147 | 385 | 260/500 | 240/420 | 180/355 | 120/260 | 50/200 |
| 5.1 | 50% natural + 50% RCA | 0.9 | 73 | 172 | 260/510 | 260/505 | 260/450 | 240/430 | 220/390 |

Table 7 shows the fresh properties of the concrete with 100% natural aggregates and the concretes with 50% natural aggregates and 50% RCA with and without Re-Con AGG 100 where a lower percentage of absorbed humidity has been used in the concrete mix design.

*Table 7: Fresh properties obtained from tests with 50% natural aggregates and 50% RCA with adjusted absorbed humidity values, compared to the reference with 100% natural aggregates.*

| Mix ID | Aggre-gates | AGG 100 [%] | Rheology, $\tau_0$ [Pa] | | Workability, slump/flow [mm/mm] | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | 0' | 60' | 0' | 30' | 60' | 90' | 120' |
| 1.1 REF | 100% natural | x | 162 | 226 | 270/570 | 250/500 | 250/450 | 220/380 | 160/300 |
| 4.2 REF | 50% natural + 50% RCA | x | 419 | 576 | 230/410 | 120/290 | 30/200 | 20/200 | 10/200 |
| 5.2 | 50% natural + 50% RCA | 0.9 | 207 | 310 | 240/420 | 235/410 | 220/400 | 170/310 | 100/230 |

Figure 1 - Figure 3 illustrate the workability of mix 1.1, concrete with 100% natural aggregates, mix 4.1, concrete with 50% RCA without Re-Con AGG 100 and mix 5.1, concrete with 50% RCA where Re-Con AGG 100 has been added, initially after mixing, after 1 hour and 2 hours.

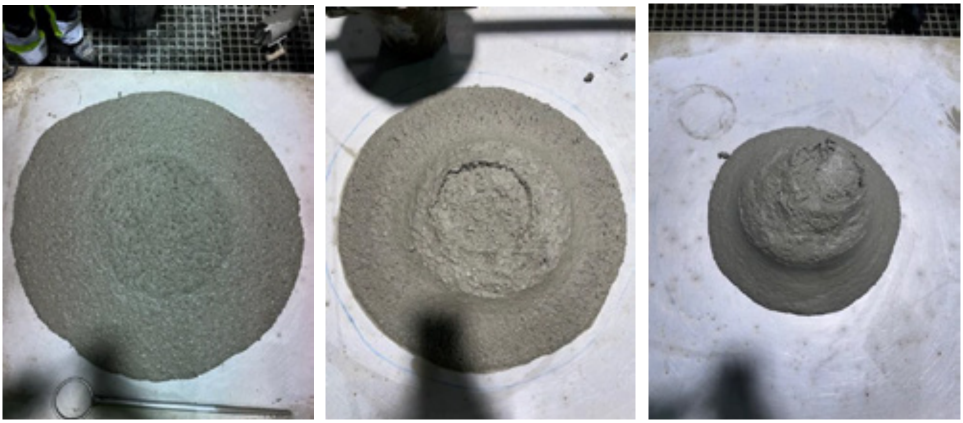

*Figure 1: Concrete mix 1.1 initially after mixing, after 1 hour and after 2 hours.*

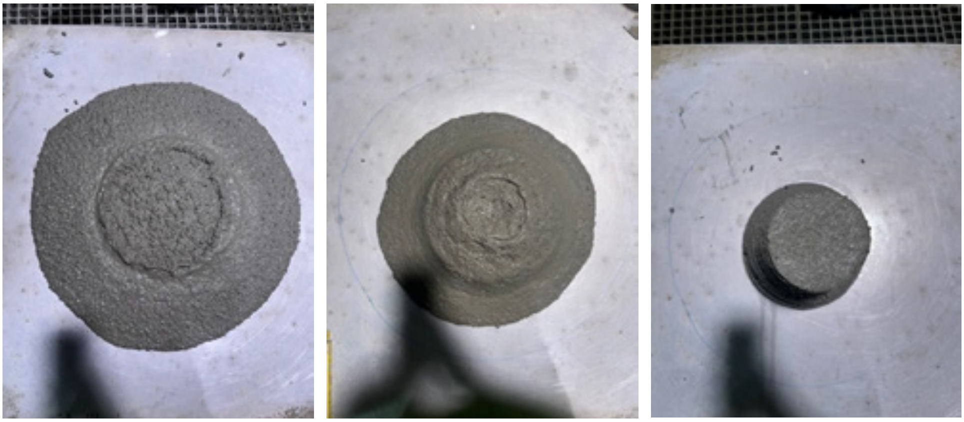

*Figure 2: Concrete mix 4.1 initially after mixing, after 1 hour and after 2 hours.*

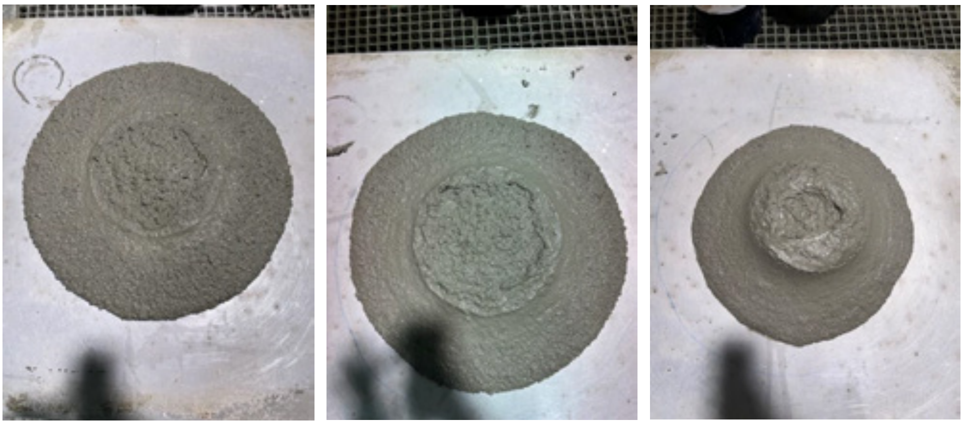

*Figure 3: Concrete mix 5.1 initially after mixing, after 1 hour and after 2 hours.*

### 3.2 Mechanical strength

Compressive strength 24 hours, 7 days and 28 days after mixing for all concretes are presented in Figure 4 below.

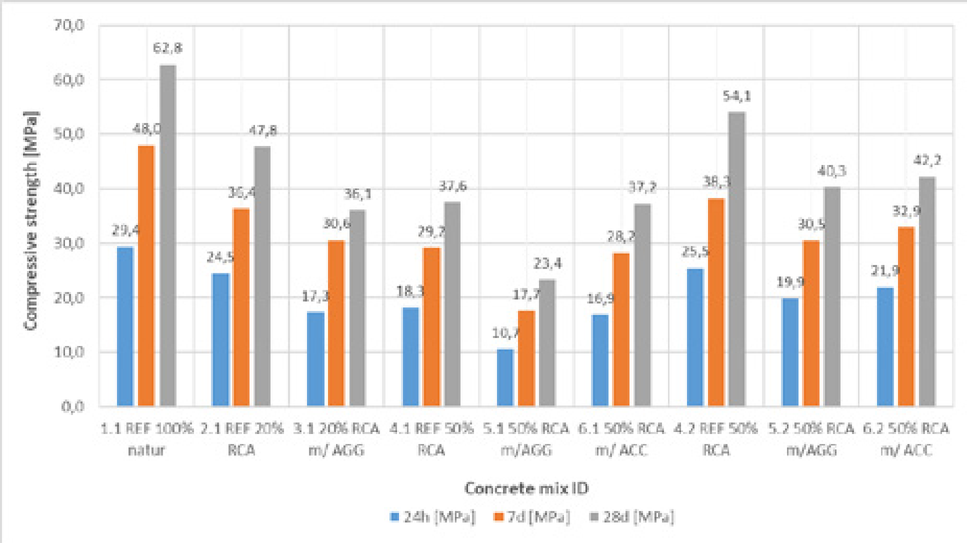


*Figure 4: Compressive strength after 24 hours, 7 days and 28 days for all concrete mixes.*

## 4. DISCUSSION

The results presented in Table 4 indicate that the flowability of the concrete is affected by replacing parts of the natural aggregates with recycled aggregates. The same dosage of superplasticizer gives significant lower flowability over time when parts of the natural aggregates are replaced by recycled aggregates. Increasing the percentage of recycled aggregates from 20% to 50% influences the flowability and reduces the flow further. However, when comparing mix 1.1, 2.1 and 4.1, the biggest effect on the flow is observed when the amount of recycled aggregates was increased from none to 20%.

The results in Table 5 – Table 7 show that by adding Re-Con AGG 100 in combination with the superplasticizer, the flowability is greatly improved. The concretes containing 20% and 50% recycled aggregates mixed with Re-Con AGG 100 presented in Table 5 and Table 6 had even longer workability time than the reference with only natural aggregates. The mixes with adjusted humidity content, 4.2 and 5.2 have the same trend. Comparing mix 4.2 and 5.2 illustrates that by adding Re-Con AGG 100 the flowability is greatly improved.

In terms of rheology, the yield stress, $\tau_0$ has been measured. The results presented above in Table 4 - Table 7 shows that $\tau_0$ increases from initial mixing time to 1 hour after mixing for all the concretes tested. This indicates that over time the concretes need more energy to move i.e., higher applied stress to initiate flow. When comparing the references, mix 1.1, 2.1 and 4.1, it can be observed that the initial yield stress is quite similar for the three mixes. However, the mixes with RCA, especially 50% RCA, has a larger increase in yield stress from the initial measurement to the 1-hour measurement. This indicates that the concretes with RCA requires higher applied stress to initiate flow 1 hour after mixing. When comparing mix 2.1 with mix 3.1, and mix 4.1 with 5.1, the results show that the concretes with Re-Con AGG 100 have lower yield stress at the initial measurement, and experiences less increase and more stable values over time. Comparing mix 1.1 and 3.1 indicates that the initial yield stress for the concrete with 20% recycled aggregates and Re-Con AGG 100 is lower than the reference with 100% natural aggregates and experience a lower increase in yield stress after 1 hour. The concrete mix with 50% RCA also experiences lower initial yield stress compared to the reference with 100% natural aggregates when Re-Con AGG 100 is added but experiences a bit higher yield stress increase after 1 hour compared to the reference with 100% natural aggregates.

Another issue by using recycled aggregates, in addition to challenging fresh properties, is the possibility of reduced early and late strength of the concrete. The reduced compressive strength of the concrete caused by replacing parts of the natural aggregates with RCA is illustrated in Figure 4. The results illustrate a significant decrease in both early and late compressive strength when parts of the natural aggregates are replaced with RCA. Increasing the percentage of RCA aggregates from 20% to 50% resulted in an even larger decrease in compressive strength. These results also illustrate a challenge by using Re-Con AGG 100, as they show a further reduction in compressive strength when Re-Con AGG 100 is added to the concrete.

The recycled aggregates tested in this study contained a high amount of moisture. The amount of absorbed humidity is difficult to establish for RCA as the concrete quality, porosity and other properties may vary. In this study, the absorbed humidity was estimated based on visual observations and prior experiences. Therefore, a certain uncertainty is associated with these assumed values. The actual w/c-ratio might be higher for the mixes with RCA due to incorrect assumed absorbed humidity. This might explain parts of the reduced compressive strength for the concretes containing RCA. It was decided to re-do the mixes with 50% natural aggregates and 50% RCA, but with reduced assumed absorbed humidity values. The compressive strength results presented in Figure 4 show that these re-done concrete mixes, mix 4.2 – 6.2, were closer to the reference with 100% natural sand, but still experienced a decrease in compressive strength when 50% of the natural aggregates were replaced with RCA.

Higher compressive strength can be achieved by using an accelerator. Comparing compressive strength for mix 6.1 and 6.2 with mix 5.1 and 5.2, presented in Figure 4, illustrates the effect of an ordinary concrete accelerator. The accelerator used in these tests is designed to influence the late strength of the concrete. Mix 6.1 and 6.2 contains both Re-Con AGG100 and the accelerator, but the RCAs in the two mixes have different assumed humidity. The humidity in the mix design for mix 6.2 has been reduced. The initial results, mix 4.1-6.1, shows that the accelerator compensates for the compressive strength loss caused by Re-con AGG 100, but the increase in compressive strength is not enough to also compensate for the loss caused by replacing parts of the natural aggerates with RCA. For the tests where the absorbed humidity value had been reduced, i.e. mix 4.2-6.2, the accelerator compensates only for a small part of the compressive strength loss caused by Re-con AGG 100. It is presumed that the Re-Con AGG 100 dosage is too high, and a lower dosage might not affect the compressive strength as much as the dosages tested in this study.

## 5. CONCLUSION

The fresh properties measured in this study, i.e. the workability and rheology proved to be challenging when parts of the natural aggregates were replaced with RCA. Re-Con AGG 100 had a positive effect regarding both workability and rheology on the concrete mixes tested, even on the mixes with as much as 50% RCA. When Re-Con AGG 100 was added, the concretes performed better than their respective references with 20% and 50% RCA in terms of slump and flow and had lower and more stable yield stress measurements. The concretes with 20% and 50% RCA even had longer workability time and lower initial yield stress than the concrete with 100% natural aggregates when Re-Con AGG 100 was added. The concrete with 20% RCA and Re-Con AGG 100 also had less increase in yield stress after 1 hour that the concrete with 100% natural aggregates.

Decreased compressive strength due to replacing the natural aggregates with RCA and use of Re-Con AGG 100 might be a challenge when using RCA in sprayed concrete. During further testing it is advised to investigate the absorbed humidity in more detail, as this will contribute to more accurate results. The scope of this study was to investigate the fresh properties of the sprayed concrete. Further testing regarding the effect of different dosages and accelerators in concretes with recycled aggregates is therefore advised, as well as investigating the effect Re-Con AGG 100 has on the compressive strength of the concrete. However, the degree of RCA replacement in these experiments has been higher than what is currently allowed according to EN 206 + NA. The results regarding fresh properties indicates that a replacement above the current limits might be possible, but further testing is required.

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# COMPARISON OF THE HEAVY METAL LEACHING PROPERTIES FROM CRUSHED SHOTCRETE AND CONCRETE WASTE

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**Abstract**

The leaching properties were investigated for shotcrete with accelerators based on both $Al_2(SO_4)_3$ and $Na_2SiO_3$, with the aim of comparing them to the leaching properties of regular crushed concrete waste. Shotcrete cores were drilled from two different Norwegian tunnels. In order to perform a scientifically sound assessment, the pH dependent leaching properties were determined for the major, minor and trace elements.

The analysis confirmed a high cement content and corresponded to the cement quantity specified in the provided recipes received from Norwegian Public Roads Administration. The content of trace elements was normal and lower than the limit values for concrete and brick waste in the Norwegian waste regulation. Leaching as a function of pH from shotcrete assessed in this study, was not found to be different from crushed concrete waste. Furthermore, the leaching is likely to be more dependent on the carbonation level than the type of accelerator. Based on the generic leaching patterns of the major elements and the toxic heavy metals, it seemed that the leaching behaviour was governed by the same reaction mechanisms in both shotcrete and regular crushed concrete. The findings may be used for decision making guidelines regarding the recycling of shotcrete.

## INTRODUCTION

Today's regulations for the recycling of concrete and masonry part of the construction and demolition waste (C&D waste) are regulated through the Norwegian Waste Regulations, Chapter 14a [1]. The regulation requires that the pollution level of the buildings or structures to be demolished must be below the regulation's specified values for the total content of chemical substances in concrete and masonry materials. The concrete must also not contain shotcrete. Based on the high admixture dosage, it is recommended that leaching should be assessed in each individual case when utilizing concrete waste from shotcrete [2].

The limit values for total content are based on leaching and the spread potential for the regulated chemical substances. Leaching from concrete in monolithic and crushed form has also been well documented in the literature over the past 20 years [3]. Shotcrete is cement-rich (400-480 kg/m$^3$) where 0/8 mm aggregates are used. The shotcrete accelerator (liquid) is added to the nozzle during spraying. In the last 15 years, accelerators based on $Al_2(SO_4)_3$ have been dominant. Accelerators based on sodium silicates ($Na_2SiO_3$) have been used in the past, where the proportion decreased in the period 1997-2005. Until 1997, alkali-free accelerators were more at the "test stage" in Norway (e.g. Fatima, Strømsåstunnelen, Bømlafjordtunnelen etc.) [4].

The reaction mechanism is different for the different admixtures and can be described according to equation (1) and (2) below.

$$Al_2(SO_4)_{3\,(aq)} + 6\,CaO_{\,(aq)} + 32\,H_2O_{\,(l)} = Ca_6Al_2(SO_4)_3(OH)_{12}\cdot 26\,H_2O_{\,(s)} \quad (1)$$

$$Na_2SiO_{3\,(aq)} + Ca(OH)_{2\,(aq)} + xH_2O = CaO(SiO_2)\cdot(H_2O)_{y\,(s)} + 2\,NaOH_{\,(aq)} \quad (2)$$

Aluminium sulphate binds to ettringite according to (1) [5], while sodium silicate forms calcium silicate hydrate and contributes to a higher pH by formation of NaOH. In carbonated concrete, the pH will be less than 10 and ettringite will be converted to $CaSO_4 \cdot 2H_2O$, $CaCO_3$ and slightly soluble $Al(OH)_3$. Use of $Al_2(SO_4)_3$ as an accelerator may therefore lead to a potential increased risk of leaching of sulphates due to carbonation.

This study investigated the leaching properties of shotcrete using accelerators based on both $Al_2(SO_4)_3$ and $Na_2SiO_3$. The goal was to compare the measured leaching properties of shotcrete with normal crushed concrete waste. This study has investigated drilled core samples from both the Lærdal tunnel and the Selta tunnel in Norway. To the authors best knowledge, no similar studies have been found in the literature.

## MATERIALS AND METODS

### Samples

The overview of drilled sample cores is shown in *Table **1***. The samples were selected in collaboration with Cowi and Norwegian Roads Public Administration and the cores were drilled out by Sørheim Riveservice AS (Bergen, Norway). The samples were crushed in a Retsch BB 250 XL jaw crusher to < 1 mm. After this, steel fibres were removed manually using a magnet. A laboratory sample was prepared for cores from Selta west (S-V), Lærdal north (L-N) and Lærdal south (L-S) by pulverizing the crushed samples to less than 200 µm in a Culatti MFC stainless steel disc mill.

*Table 1 – Overview of drilled core samples for chemical analysis and leaching assessments*

| Sample name | Tunnel | Age of shotcrete | Accelerator | Amount of binder[a] (C +MS) kg/m$^3$ |
|---|---|---|---|---|
| S-V-440-2 | Selta west | 2022 | Aluminium sulphate | 455 + 19 |
| S-V-440-3 | Selta west | 2022 | Aluminium sulphate | 455 + 19 |
| S-Ø-1060-2 | Selta east | 2002-2003 | Aluminium sulphate | Not given |
| S-Ø-1060-4 | Selta east | 2002-2003 | Aluminium sulphate | Not given |
| L-N-12250-1 | Lærdal north | 1995-1999 | Sodium silicate | 493 +27 |
| L-N-12250-2 | Lærdal north | 1995-1999 | Sodium silicate | 493 +27 |
| L-N-12250-5 | Lærdal north | 1995-1999 | Sodium silicate | 493 +27 |
| L-N-12250-6 | Lærdal north | 1995-1999 | Sodium silicate | 493 +27 |
| L-S-16100-2 | Lærdal south | 1995-1999 | Sodium silicate | 460 + 17 |
| L-S-16100-3 | Lærdal south | 1995-1999 | Sodium silicate | 460 + 17 |
| L-S-16100-5 | Lærdal south | 1995-1999 | Sodium silicate | 460 + 17 |
| L-S-16100-6 | Lærdal south | 1995-1999 | Sodium silicate | 460 + 17 |

[a] C + MS = Cement + Microsilica

### Leaching tests

Leaching from pulverised material was assessed by pH-dependent leaching with acid/base addition in accordance with NS-EN 14429:2015. This method involves assessing the leaching properties of the inorganic elements with regard to equilibrium concentrations at different pH values. Sample portions of 40 g of pulverised material for S-V and 25 g for L-N and L-S were accurately weighed and used in each of the leaching steps.

Each of the different leaching steps was prepared with demineralised water (>18.2 MΩ cm) in 500 ml LDPE containers. Necessary amounts of 1.44 M $HNO_3$ (Suprapur) or 1.0 M NaOH (analytical grade) were added to the suspensions to achieve final pH at regular intervals between 2.0 and 13.5. A suspension without acid or base addition was also included in all leaching tests. The final liquid/solid ratio (L/S) used was 10.0 ± 0.4 ml/mg. The suspensions were brought to equilibrium by rotating for 48 hours (REAX 20, Heidolph Instruments) followed by sedimentation for at least 4 hours. After this, the pH was determined, and the suspensions filtered through a 0.45 µm membrane filter and the conductivity measured. The measured pH value for the suspension without acid/base addition is referred to as the material's native pH (material pH).

### Chemical analysis

Determination of Al, As, Ca, Cd, Cu, Cr, Cr(VI), Fe, K, Mg, Mn, Na, Ni, P, Pb, Si, Ti and Zn in the crushed sample materials was carried out using sector field inductive coupled plasma mass spectrometry (ICP-SFMS) according to SS-EN ISO 17294-2:2016 and US EPA Method 200.8:1994. Determination of Hg was carried out using atomic fluorescence spectrometry (AFS) according to SS-EN ISO 17852:2008.

The following methods were used for the eluates. Ca, K, Mg, Na, S, Si were determined using inductively coupled atomic emission spectrometry (ICP-AES) according to SS-EN ISO 11885:2009 and US EPA Method 200.7:1994, whereas Al, As, Cd, Cu, Cr, Fe, Ni, Mn, Pb and Zn were determined with ICP-SFMS according to SS-EN ISO 17294-2:2016 and US EPA Method 200.8:1994. Determination of Hg was carried out using AFS according to SS-EN ISO 17852:2008 and Cr(VI) by ion chromatography with spectrophotometric detection according to CSN EN 16192, EPA 7199, SM 3500-C. The elemental analyses were carried out by ALS Laboratory Group Norway.

For loss on ignition (LOI) determination, dry sample was accurately weighed and dried at 105 °C before being heated for 75 minutes at 1000 °C. The cooled sample was weighed accurately to calculate the LOI. These analyses were conducted by ALS Laboratory Group Norway.

Carbonation depth was assessed using 1.0% (m/V) thymolphthalein indicator. The solution was prepared by dissolving the indicator in a solution of ethanol (70 mL) and demineralized water (30 mL). The indicator was sprayed to the freshly broken surface of shotcrete. No colour reaction indicated a pH < 10, while a blue coloration indicated a pH > 10.

The acid-soluble content was determined using a method based on the Nordtest method NT BUILD 437. The samples that were crushed in the disc mill were dissolved in nitric acid. Acid-insoluble residue is filtered off and the weight was determined accurately. The acid-soluble proportion indicates the amount of cement paste and any limestone filler.

## RESULTS AND DISCUSSION

### Total contents

The chemical contents of the main elements and loss on ignition (LOI) are shown in Table 2. A higher amount of binder was found in L-N compared to S-V and L-S. This was also shown in the content of CaO. There is no prescription for S-Ø, but the results indicated that the concrete was rich in binder. The binder content is reflected in the amount of acid-soluble proportion shown in Table 4, where the amount was determined to be 27-31%. The results indicated that the cement paste content is highest in L-N (30.6%), which agrees with the concrete recipe provided by the tunnel owner. Furthermore, an uncarbonated new concrete with a cement quantity of 325 kg/m$^3$, will give an acid-soluble content of 19% [6]. This is consistent between the amount of cement and the acid-soluble proportion in Table 4. The investigations verified the high cement content in the samples taken, which was important because the leaching results were compared with previous results on concrete with less cement content and without accelerator.

Based on the total content of Al, Na and K, it is difficult to distinguish between shotcrete with sodium silicate and aluminium sulphate. This also applies to N/K and Al/S ratios. This is because the contribution from aggregates is significant for Al, Na and K in a full decomposition carried out in this study. However, this is generally not the case with $SO_3$, which must be below 0.1% $SO_3$ in aggregates for concrete. The results in Table 2 show a tendency towards a lower sulphur content for L-S. This is expected in shotcrete with sodium silicate. The results indicated that sodium silicate has been used in L-S and aluminium sulphate in S-V, because approximately the same cement content has been used.

The content of trace elements is indicated in Table 3 and normal concentrations of all elements were found. All the concentrations were lower than the limit values for concrete and brick waste in chapter 14a of the Norwegian Waste Regulations [1]. Based on the amounts of heavy metals in the samples, they are within the requirements set for ordinary concrete and masonry structures to be demolished.

*Table 2 – Determination of the total contents of major elements and LOI (1000 °C). Results are given as % dry weight.*

| Para-meter | S-V 2 | S-V 3 | S-Ø 2 | S-Ø 4 | L-N 1 | L-N 2 | L-N 5 | L-N 6 | L-S 2 | L-S 3 | L-S 5 | L-S 6 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $Al_2O_3$ | 11.1 | 11.6 | 13.1 | 13.3 | 11.0 | 10.6 | 12.0 | 11.3 | 10.7 | 9.4 | 10.2 | 8.6 |
| CaO | 12.0 | 12.2 | 18.1 | 19.0 | 15.1 | 15.5 | 13.6 | 15.4 | 13.8 | 15.1 | 14.8 | 13.3 |
| $Fe_2O_3$ | 3.2 | 3.6 | 6.7 | 6.8 | 4.3 | 4.1 | 4.6 | 4.8 | 3.4 | 3.1 | 3.4 | 2.9 |
| $K_2O$ | 3.4 | 3.5 | 2.0 | 2.0 | 2.4 | 2.3 | 2.7 | 2.6 | 3.6 | 2.7 | 2.8 | 2.7 |
| MgO | 0.94 | 0.97 | 3.1 | 3.2 | 1.6 | 1.6 | 1.5 | 1.6 | 1.0 | 0.9 | 1.1 | 1.2 |
| MnO | 0.04 | 0.04 | 0.11 | 0.11 | 0.09 | 0.09 | 0.09 | 0.09 | 0.05 | 0.05 | 0.06 | 0.05 |
| $Na_2O$ | 2.6 | 2.8 | 2.9 | 2.9 | 2.6 | 2.4 | 2.8 | 2.6 | 2.8 | 2.6 | 2.7 | 2.3 |
| $P_2O_5$ | 0.10 | 0.14 | 0.22 | 0.21 | 0.14 | 0.17 | 0.17 | 0.17 | 0.10 | 0.08 | 0.12 | 0.11 |
| $SiO_2$ | 55 | 56 | 44 | 44 | 53 | 48 | 54 | 53 | 59 | 51 | 55 | 49 |
| $SO_3$ | 0.38 | 0.37 | 0.38 | 0.46 | 0.47 | 0.45 | 0.36 | 0.42 | 0.20 | 0.25 | 0.22 | 0.22 |
| $TiO_2$ | 0.36 | 0.41 | 0.47 | 0.50 | 0.70 | 0.88 | 0.52 | 0.60 | 0.38 | 0.40 | 0.56 | 0.81 |
| LoI | 6.5 | 5.2 | 6.6 | 7.6 | 8.5 | 10.6 | 6.4 | 6.8 | 6.1 | 10.7 | 9.4 | 12.2 |

*Table 3 – Determination of total content of heavy metals. All results are given as mg/kg (of dry weight)*

| Para-meter | S-V 2 | S-V 3 | S-Ø 2 | S-Ø 4 | L-N 1 | L-N 2 | L-N 5 | L-N 6 | L-S 2 | L-S 3 | L-S 5 | L-S 6 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| As | <3 | <3 | 3.3 | 3.3 | 4.3 | 4.3 | <3 | 3.5 | <3 | <3 | <3 | 3.0 |
| Cd | <0.05 | 0.1 | 0.5 | 0.4 | 0.2 | 0.1 | 0.1 | 0.1 | 0.1 | 0.1 | <0.05 | 0.1 |
| Cr | 26 | 33 | 91 | 84 | 27 | 34 | 39 | 40 | 35 | 43 | 41 | 32 |
| $Cr^{6+}$ | 2.3 | 0.3 | <0.3 | <0.3 | 1.8 | 0.6 | <0.3 | 1.4 | 2.7 | 0.7 | 1.5 | 2.1 |
| Cu | 30 | 30 | 89 | 80 | 20 | 18 | 19 | 19 | 8.2 | 12 | 10 | 9.3 |
| Hg | <0.01 | <0.01 | <0.01 | <0.01 | <0.01 | <0.01 | <0.01 | <0.01 | <0.01 | <0.01 | <0.01 | <0.01 |
| Ni | 12 | 15 | 31 | 27 | 22 | 19 | 21 | 21 | 10 | 13 | 11 | 11 |
| Pb | 23 | 25 | 16 | 17 | 22 | 21 | 19 | 20 | 19 | 17 | 17 | 19 |
| Zn | 125 | 115 | 90 | 92 | 70 | 66 | 66 | 62 | 57 | 51 | 52 | 50 |

*Table 4 – Acid-soluble content (%) determined in S-V, L-N and L-S samples*

| Sample name | Acid-soluble proportion | Amount of binder[a] kg/$m^3$ |
|---|---|---|
| S-V-440-Mix | 26.5 | 474 |
| L-N-12250-Mix | 30.6 | 520 |
| L-S-16100-Mix | 28.3 | 477 |

[a] Amount of cement and silica fume (Microsilica)

**Leaching**

The three samples tested for leaching in this study were S-V-440-Mix, L-N-12250-Mix and L-S-16100-Mix. These samples consisted of a combination of two samples from each corresponding sample series in Table 1. This was to ensure a representative composition. S-V-440-Mix consisted of: S-V-440-2 and S-V-440-3. L-N-440-Mix consisted of: L-N-440-1 and L-N-440-2. L-S-16100-Mix consisted of: L-S-16100-5 and L-S-16100-6. The two samples in each series were first crushed to a fine powder, then thoroughly mixed using the homogeneous mixing method (internal SINTEF method). The results are compared with leaching from crushed concrete (RCA 0/4 and RCA 4/8) and crushed mortar (RCA-M) from previous studies [6], [7].

The results for the binder's main elements (Al, Ca, Fe, Mg, S and Si) and alkalis (Na and K) are compiled in Figure 1. Leaching of main elements as a function of pH will generally show the typical leaching patterns of cement-based materials. The results showed that leaching from shotcrete is not different from previous results for crushed concrete and crushed mortar. This indicates that the leaching will be controlled by the same reaction mechanisms such as solubility controlling mineral phases, sorption to Fe and Al oxide surfaces, ion exchange and complex formation to humic substances.

The results in Figure 1 also show that variation in carbonation affects certain elements such as Al, Ca, S and Si. L-N-1250-Mix and L-S-16100-Mix had lower material pH (11.3 and 11.4) than S-V-440-Mix (12.4). Material pH for uncarbonated concrete in this leaching test (L/S = 10) will be 12.5-12.8. This indicates that the L-S and L-N samples were more carbonated than S-V which was confirmed by the carbonation measurements with thymolphthalein pH indicator. This is also shown in Figure 1 in the following way. Al leaching increases at high pH with increasing carbonation because the calcium aluminate phases are destabilized. This is also clearly shown for S and Si in Figure 1, in that the ettringite phase is destabilized (formation of gypsum) and that the C-S-H phase is decalcified so that leaching of S (sulphate) and Si increases at high pH with increasing carbonation. During carbonation, $Ca(OH)_2$ will be converted to $CaCO_3$. Leaching of Ca should therefore be lower with increasing carbonation, because $CaCO_3$ has far lower solubility than $Ca(OH)_2$ at pH > 8, which is clearly shown in Figure 1.

Figure 1 also shows, as expected, that the Na and K leaching was far less pH dependent than the main elements. They are more governed by availability (quantity), since the alkalis have a high solubility. The K/Na ratio at the material pH is shown in Table 5. No correlation was found between the samples where the use of sodium silicate will lead to the expectation of increased leaching of Na and therefore reduced K/Na ratio.

It can be summarized that pH-dependent leaching did not differ significantly between shotcrete and crushed concrete waste. Leaching from shotcrete with alkali and alkali-free accelerators is probably more dependent on carbonation than type of accelerator.

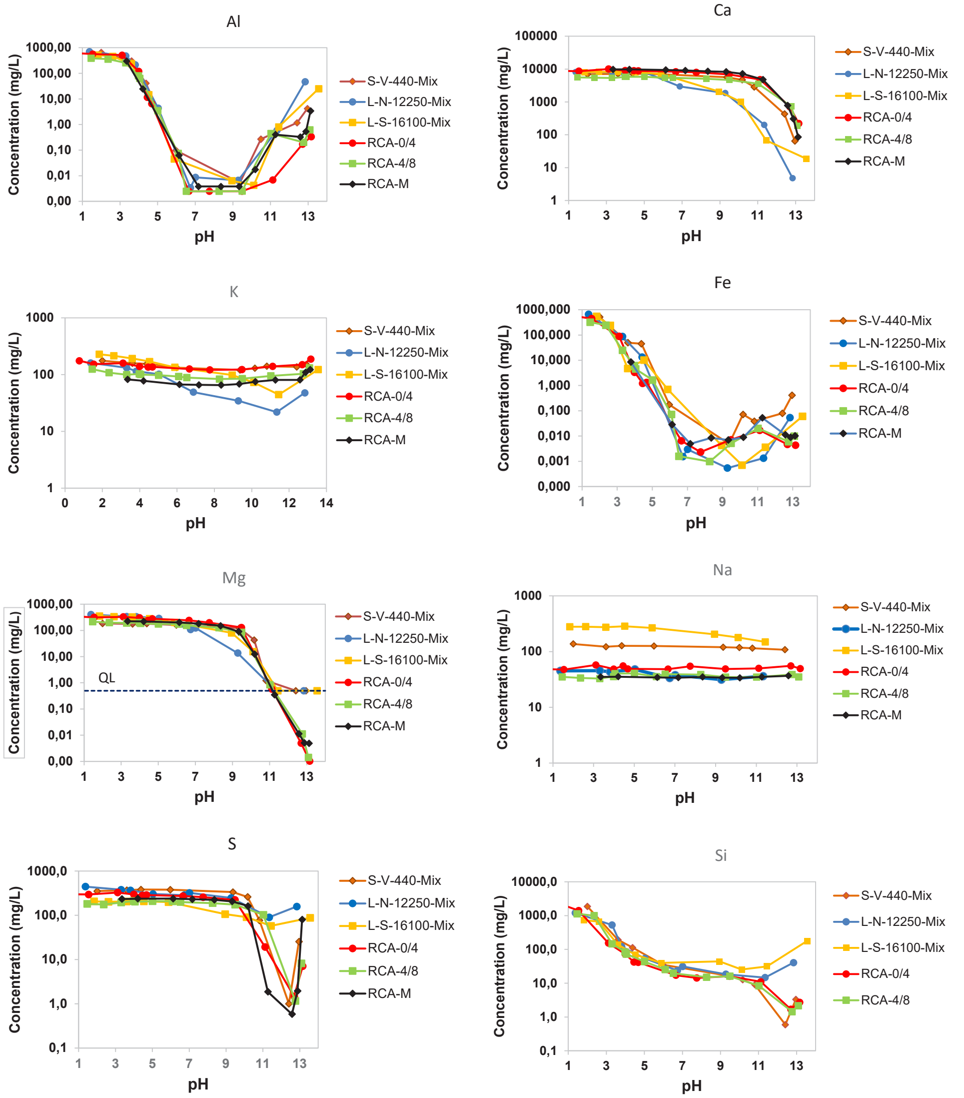


*Figure 1 - Leaching as a function of pH. The quantification limit for Mg in this study is indicated as QL (Quantification Limit) with a horizontal dashed line. The second last basic data point on each measurement series is the material pH. For Na, the material pH is the last data point. RCA 0/4 (crushed concrete), RCA 4/8 (crushed concrete) and RCA-M (crushed mortar prism) are leaching from crushed concrete and mortar from previous studies [6], [7].*

*Table 5 - Leaching of Na and K at the material pH*

| Sample name | Na mg/L | K mg/L | K/Na-ratio | Material pH |
|---|---|---|---|---|
| S-V-440-Mix | 108 | 137 | 1.27 | 12.4 |
| L-N-12250-Mix | 36.2 | 21.9 | 0.60 | 11.3 |
| L-S-16100-Mix | 149 | 44.6 | 0.30 | 11.4 |
| RCA-0/4 | 55.5 | 150 | 2.71 | 12.7 |
| RCA-4/8 | 38.8 | 104 | 2.68 | 12.8 |
| RCA-M | 36.8 | 110 | 2.20 | 12.6 |

**Leaching of heavy metals**

The leaching of toxic heavy metals (As, Cd, Cu, Hg, Cr, Ni, Pb and Zn) and hexavalent chromium (Cr(VI)) are shown in Figure 2 and Figure 3. In the Water Directive, environmental quality standards (EQS) have been set for a several substances. The directive's limit values for Annual Average Environmental Quality Standards (AA-EQS) are indicated and shown in Figure 2, to illustrate the leaching concentrations in environmental context. It is emphasised that a direct comparison would be too conservative and site specific information must usually be applied to perform a relevant environmental risk assessment**.**

The results showed that the same generic leaching patterns was found for shotcrete and crushed concrete. The metal cations $Cd^{+}$, $Cu^{2+}$, $Ni^{2+}$, $Pb^{2+}$ and $Zn^{2+}$ showed low leaching at high basic pH, because the metals are bound to the cement hydrate phases, so that the leaching is not controlled by the solubility of the corresponding pure metal hydroxide. The substances that form oxyanions ($CrO_4^{2-}$ and $AsO_4^{3-}$) will typically have increased leaching during carbonation. This means that the leaching increases when the pH is lowered from the material pH, as shown for As and Cr in Figure 2 for both shotcrete and crushed concrete.

Based on SINTEF's experience with leaching from crushed concrete waste, there will always be somehow a variation in the leaching concentrations from different samples of crushed concrete waste because the amount of cement, type of cement and heavy metal content will vary. Leaching from the shotcrete in this study seemed to be within this variation, and the same leaching mechanisms will apply to crushed concrete waste and crushed shotcrete. Leaching at material pH is indicated in Table 6 (L/S = 10) and shows that there is little difference between the concrete types.

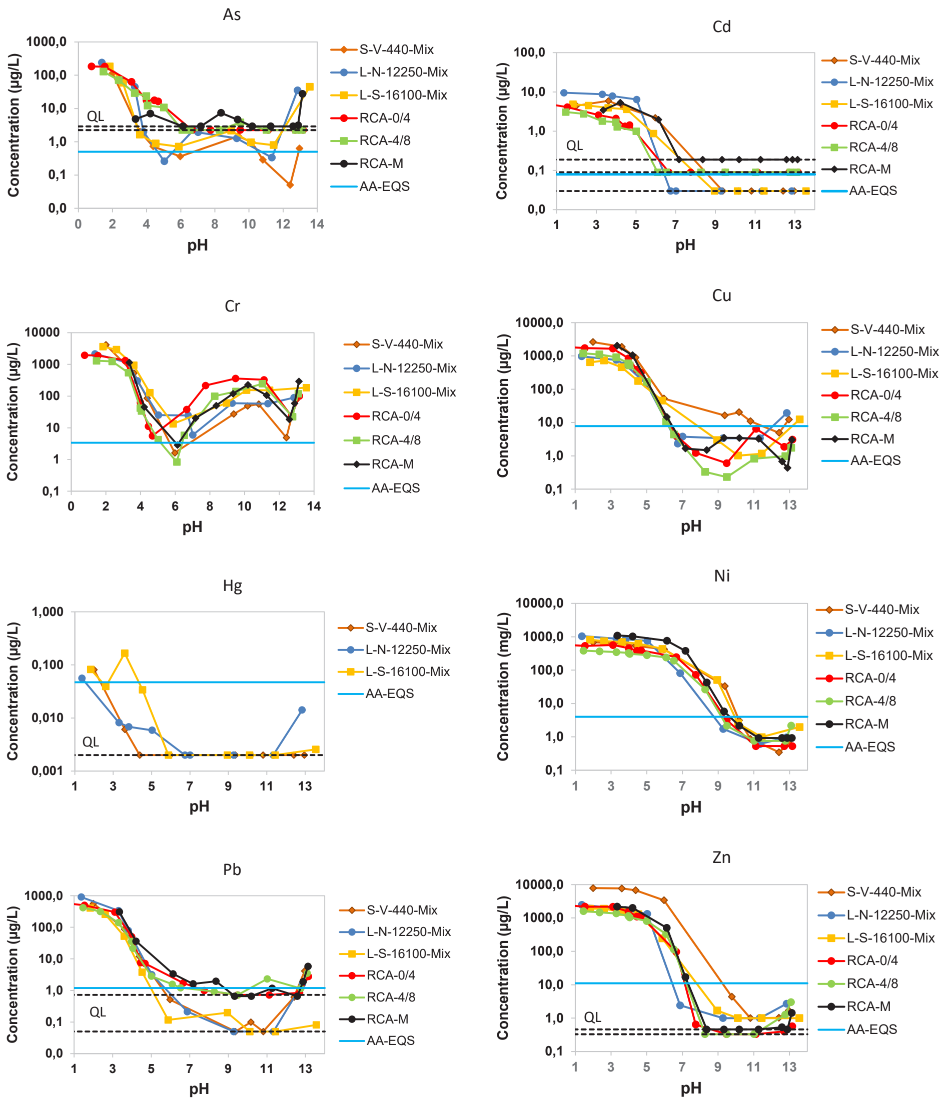


*Figure 2 - Leaching as a function of pH for As, Cd, Cu, Cr, Hg, Ni, Pb and Zn. Quantification limits are indicated as QL (Quantification Limit) with horizontal dashed lines. The second last basic data point on each measurement series is the material pH. AA-EQS is the Annual Average Environmental Quality Standard. RCA 0/4 (crushed concrete), RCA 4/8 (crushed concrete) and RCA-M (crushed mortar prism) are leaching from crushed concrete and mortar from previous studies [6], [7].*

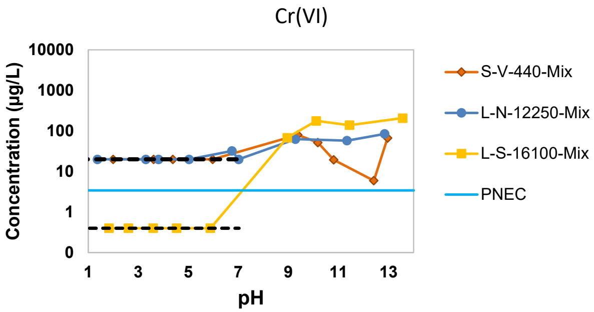


*Figure 3 - Leaching as a function of pH for Cr(VI). Quantification limits are indicated as QL (Quantification Limit) with horizontal dashed lines from pH < 7.0. The second last basic data point on each measurement series is the material pH. PNEC is the predicted no-effect concentration.*

*Table 6 – Leaching at the material pH (L/S = 10). All concentrations are given as µg/L.*

| Sample name | As | Cd | Cr | Cu | Hg | Ni | Pb | Zn |
|---|---|---|---|---|---|---|---|---|
| S-V-440-Mix | < 0.05 | < 0.03 | 4.9 | 5.2 | < 0.002 | 0.35 | 0.61 | 1.02 |
| L-N-12250-Mix | 0.34 | < 0.03 | 58 | 3.4 | < 0.002 | 0.61 | < 0.05 | 0.27 |
| L-S-16100-Mix | 0.79 | < 0.03 | 154 | 1.2 | < 0.002 | 0.99 | < 0.05 | < 1.0 |
| RCA-0/4 | < 2.25 | < 0.09 | 24 | 1.9 | n.m.[a] | < 0.53 | 0.85 | 0.40 |
| RCA-4/8 | < 2.25 | < 0.09 | 22 | 1.0 | n.m.[a] | < 0.76 | 1.16 | 1.26 |
| RCA-M | < 2.9 | < 0.19 | 18 | 0.7 | n.m.[a] | < 0.93 | < 0.66 | 0.53 |

[a] n.m. = not measured

## CONCLUSION

In this study, the leaching properties were assessed for shotcrete with accelerators based on both $Al_2(SO_4)_3$ and $Na_2SiO_3$. The purpose was to compare the measured leaching properties of shotcrete with ordinary crushed concrete waste.

Drilled shotcrete cores had a high cement content and corresponded to the amount of cement stated in the provided recipes. The content of trace elements was found in normal concentrations of all substances. All the concentrations were lower than the limit values for concrete and masonry waste in Chapter 14a of the Norwegian Waste Regulation.

Leaching as a function of pH was found to be the same for shotcrete and crushed concrete in this study. No different effect of alkali and alkali-free accelerator on leaching was found, which is probably more dependent on carbonation than type of accelerator. From the generic leaching patterns of the main elements and toxic heavy metals, the leaching is controlled by the same reaction mechanisms (solubility, sorption to oxide surfaces, ion exchange and complexation) in shotcrete and normal crushed concrete. The leaching at the material pH from the different concrete types were found to be in the expected concentration regions.

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## CONSEQUENSES OF DIFFERENT USE OF SCANS AS A TOOL FOR THICKNESS CONTROL

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**Abstract**

The purpose of this article is to highlight challenges that arises as a consequence of detailed scans. Furthermore, to discuss different perspectives as a basis for thickness measurement. It will not go deeper into consequences with reduced thickness, as this depends on many different factors.

The control frequency in Norwegian Publication no.7 (2022) demands minimum 10 single measurement for every 1000 m2. In recent years, we have been moving away from manually thickness controls towards scans.

The discussion is not related to how many scanning points are needed pr square, but how this scan should be presented and assessed. Both single measurements area and the average area.

If the area that define single measurement and minimum thickness are too big, we risk having under dimensioned rock support, or at least less control of the thickness. If it is too narrow it is likely to assume that we track a higher amount of undermeasures than before, especially on rock elevations. If that is, it is a high risk that the concrete consumption increases and results in side effects as higher costs and CO2 effluent. Generally, in modern tunnels it is reasonably to ask ourselves, does this improve the final product, compared to the past? The fundamental mindset in Norwegian engineering is having a dynamic and satisfactory enough rock support. This means not exaggerating the support to out rule every uncertainty. Based on this discussion it should be a general rule described by the public client on how to use scans as thickness measurement.

**Introduction**

Many different papers and presentations have been produced recent years with the topic tunnel scanning. However, very few challenge the question of how the tunnel scannings should be interpreted.

In recent time some tunnel projects are moving away from manually thickness controls, towards tunnel scans. It's beneficial to obtain a higher grade of accurate information about the execution and a map of each square of the tunnel. When we have access to new technology, providing a higher grade of detailed information, some new challenges arise that the industry must address. Today we have the possibility to monitor down to millimeters. What would happen if we combine this data with today's demands? Will this improve the rock support and what will be the consequences?

The paper will not delve into the specifics of the statics and how different thicknesses react to forces, as this depends on many different, complex and unknown factors.

**Theory**

One must document the calculation that forms the basis for the correct amount of concrete. The calculation includes area, rock factor, rebound factor, and so on. This secure the total volume of concrete is enough to secure described thickness [1]. However, this is not a guarantee that concrete is distributed correctly over the total area. Therefore, we perform a control to check that executed thickness fulfill an average similar or higher than described. This control is distributed over ten holes for every 1000 square meters completed shotcrete, across the tunnel profile (random sample). One measurement/hole can be 50 % or more of the demanded thickness [2]. Norwegian Concrete

Association's Publication no. 7 also allows scanning as a method without precisely determining the correct way to assess a scan [3].

Shotcrete is one component in a rock reinforcement structure. Primarily aimed at strengthening the rock mass. The principle in Norwegian engineering is that the rock mass serves as the permanent load-bearing element in a tunnel or cavern, and the rock reinforcement helps to maintain the load-bearing properties of the rock mass. This principle allows for relatively small shotcrete thicknesses compared to practices in several other countries, where experience with rock conditions necessitates shotcrete reinforcement to bear static loads [4].

If the rock quality is solid, the spray concrete is used as surface support to prevent light rockfall [5]. As the rock quality deteriorates, the concrete thickness increases [6]. In situations where the rock quality is poor, we must use a shotcrete reinforcement rib. The Q-system is a tool in Norwegian engineering, applied to determine the rock support and use of shotcrete. The recommended thickness in the Q-system is based on experiences from previous projects. Projects with 1,050 cases from main road tunnels and 440 cases from hydropower tunnels [7]. Throughout the years, the requirements for shotcrete have changed and become stricter. They concern material requirements, greater thickness, energy absorption demand, and types of fibers.

There is documented research for older tunnels, but for few, or none, modern tunnels with updated demands and 100 years lifetime. The experience with modern shotcrete in tunnels covers only 25 of 100 years design lifetime. Modern shotcrete is defined as concrete with qualities described in Norwegian Concrete Association's Publication no. 7 [8].

It's not only the rock quality that determines how important thickness is, also the tunnel environment. According to the report made on behalf of the Norwegian public roads administration, thickness is important for resistance against degradation mechanisms. The worst climate is for example subsea and in alum shale environment. In such areas 100 mm thickness is recommended to extend the lifespan of steal reinforced concrete. It is observed that minimum thicknesses to withstand degradation over time varies depending on the rock type and environment [9].

**Discussion**

Random sample method leaves some degree of uncertainty regarding whether the remaining area has the correct thickness or not. A scan with today's technology can operate down to millimeters accuracy and give a picture of the surface in square centimeter. As long as the scan is high quality, this method leaves small uncertainty of how the concrete is distributed on the surface. How this scan is presented

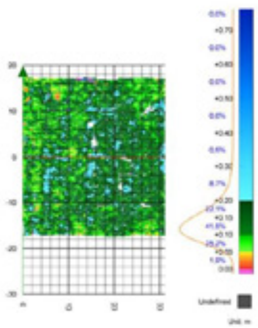

*Figure 2, Scanning over 1000 m2*

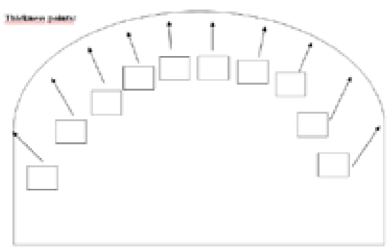

*Figure 1, Profile with random sample*

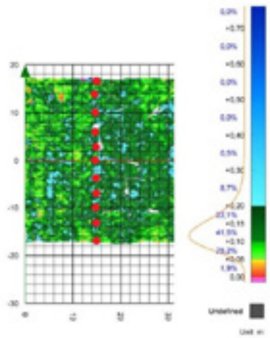

*Figure 3, Overview random sample*

will decide how big the difference is between the method random sample and a scan. If the scan is presented with high resolution with several points per square meter, where each point is a single measurement that must fulfill minimum 50 %, this creates a great difference between the control methods. Random control is ten holes per 1000 m2. In figure 3 the red spot is one random check sample with holes pr 1000 m2 measurement. While scanning has total 16 000 scanning point in this area.

This difference between the control method, can lead to the party responsible for the risk of under-measurement having to increase the amount of concrete when the high-resolution scan is used as the basis. Even though one can reapply thickness to the local points later, this will lead to more labor-intensive work. Then it may be less resource-intensive to increase the amount of concrete

The Q model is based on gathered information from previous projects about described thickness and executed thickness. This, along with other historically executed elements of rock support, forms the basis for the recommendation. If there is a significant tightening of thickness control, this may change the basic for this model. If we use great resources and can exclude lower thicknesses, it may lead to less conservatively when recommending a concrete thickness for a Q-value.

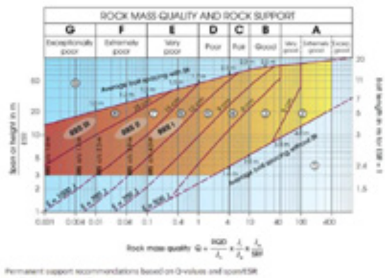


*Figure 4. Q-Model, NGI, Using the Q-system, 2015*

Most of the executed projects use random sample as a control method, with a specified interval determined by the contract. Even with experienced and skilled spray operators, that distribute the concrete with a high level of precision, this control method leaves some uncertainty regarding local thickness variations. It is likely to believe that there are some undocumented areas under 50 % of ordered thickness, especially on rock peaks. Figur 6 illustrate the rock surface (pink), an imaginary layer with 10 cm concrete (green) and executed shotcrete (dark blue).

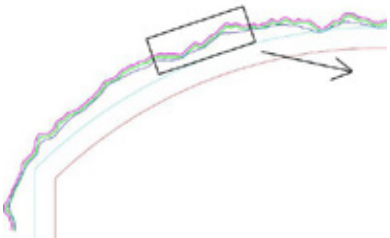

*Figure 5. Tunnel profile*

*Figure 6. Rock Peaks*

How important it is to meet the thickness requirements at this particular local peak, depends on the rock quality, sulfates, leakage levels, etc. In some cases, it may be less critical, while other times it may be more desirable to have sufficient thickness to withstand forces and degradation. If the scan has very high resolution, there is a risk of getting many under-measurements, even in areas where the effect of increased thickness provides little or no benefit. This indicates a need for a more dynamic described demand which adapt the minimum thickness requirement to the various purposes of shotcrete.

The purpose of shotcrete varies depending on the tunnel environment and rock quality. If the tunnel environment is demanding, it is important to secure correct thickness to maintain the resistance to degradation. Such conditions appears in subsea or alum shale where a minimum described average thickness is 10 cm. The greater described thickness the higher is the possibility to be above the minimum that many rapports point at to withstand decay and deterioration. In challenging conditions, it is important to have control over thickness. The scanning method will have a greater ability to ensure this. However, in tunnel environments that are not that demanding, the shotcrete mainly works as a surface support. This control upgrade can lead to increased general thickness without any form of benefit or increased lifespan for the tunnel.

The data collection has not been able to find any tunnel rehabilitation project with modern shotcrete and the higher demands of thickness, where the reason for rehabilitation is shotcrete reinforcement alone. Mostly the reason for rehabilitation is new requirements for installations and safety, inadequate water and frost protection. In some projects where rock support was the issue, the reason was under dimensioned rock support with partial elements as bolts, misinterpreted rock quality, and not performed shotcrete. Alongside with the quality of partials materials at that time. With these assumptions, the shotcrete alone would not have prevented the tunnels of upgrades. How this scenario is after100-year lifespan is difficult to predict. However, if the securing is inadequate in that scenario, one can assume that the interaction between bolts, shotcrete, and other rock support together contributes to requiring repair.

**Conclusion**

Even though we believe that today's requirements, material development, and control methods are satisfactory enough, one should also be careful not to claim that increased focus on uniform thickness is irrelevant. Specially in a 100-year lifespan. Scans can be a great tool to obtain this.

As illustrated, there are great difference in accuracy between random sample and some of the scans described today. If the option is to try to exclude the possibility of under-measurement by 100%, one must calculate with an increased safety margin when calculating the concrete consumption.

One way to not change the method drastically is to present scans with middle thickness over greater areas, example per square meters, and not use single measurements with the 50% rule. Single measurement is more natural to use in random sample.

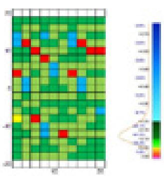

*Figure 7. Scan with m2 presentation*

To regulate and set a minimum defined presentation area that are not too narrow gives an opportunity to upgrade the thickness control and prevent a drastic change from the random control. One should base those regulations on how it affects the rock support and how it changes the assumptions we have today. This decision will influence the concrete consumption which also affects the cost structure and the CO2 accounts.

Regardless of control method, there must be regulations that determine how tunnel scans should be presented and interpreted as a basis for thickness control.

This to secure that the practice is consistent from project to project, and among different contract forms, in addition to create predictability for both the client and contractors.

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# POST-CRACKING PERFORMANCE CRITERIA FOR FIBRE-REINFORCED SPRAYED CONCRETE IN PERMANENT TUNNEL LININGS

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## Abstract

Fibre reinforced sprayed concrete linings offer potential safety, cost, and environmental benefits compared to traditional solutions. However, current standards and recommendations may not sufficiently persuade the tunnel construction industry to adopt this material for permanent tunnel lining applications. There is a pressing need for further exploration into the post-cracking performance to improve the design process of sprayed FRC permanent tunnel linings. In addressing this need, two critical aspects come into focus: the characterization of FRC material and the subsequent determination of uniaxial tensile law.

Concerning the first aspect, EFNARC proposed the use of three-point bending test on square panel with notch, which is now integrated into the new EN14488-3. In this regard, experimental tests were carried out on concrete panels reinforced by macro synthetic fibres. Two fibre types and contents were considered.

Regarding FRC uniaxial tensile law for use in the lining design process, the procedure to be adopted is still an open matter. It is herein proposed to apply the inverse analysis method, typically used in case of standard beam notched samples, to the results retrieved by square panels' tests. The main goal is to achieve the most accurate description of FRC post-cracking performance for use in the numerical simulations of embedded in ground condition of permanent sprayed concrete linings.

## Keywords

Fibre-reinforced concrete, sprayed concrete, permanent tunnel lining, three-point bending test on square panel with notch, uniaxial tensile law.

## 1 INTRODUCTION

Over the last 30 years, the use of fibre reinforced sprayed concrete for permanent tunnel lining has undergone rapid development. Sprayed Concrete Lining (SCL) is used in case of soft ground tunnelling when sprayed concrete is employed to support the excavation both temporarily and permanently [1]. When compared to traditional lining solutions, using fibre reinforced sprayed concrete lining can result in lower capital costs, operational costs, and carbon footprints [2]. However, there have been advancements in the construction environment, and the current available standards and research in the area is insufficient to fully persuade the tunnel construction industry to use fibre reinforced shotcrete for permanent tunnel linings. One of the major issues is the insufficient availability of standards and guidelines for the characterization of FRC sprayed concrete that align with the available tunnel lining design guidelines.

The current *fib* Model Code 2010 [3], hereinafter MC 2010, is used as a reference document in recommendations regarding FRC segmental tunnel linings [4, 5], and it suggests the EN 14651 [6] bending test for material characterization and classification of Fibre Reinforced Concrete (FRC). This beam geometry and test are suitable for general FRC structural applications but not for the fibre reinforced sprayed concrete. In fact, contrary to *fib* Model Code [3], the European sprayed concrete standard EN 14487-1 [7] describes various methods for specifying the ductility of fibre-reinforced sprayed concrete in terms of residual strength according to the EN 14488-3 [8] and of energy absorption capacity through the EN 14488-5 [9].

The energy values provided by tests carried out according to EN 14488-5 [9] are typically considered when designing a temporary sprayed FRC lining and are not generally used to characterize the material for designing a permanent tunnel lining. The current Method A reported in EN 14488-3 [8] seems incongruent with MC 2010 [3] as it is based on four-point bending test on un-notched beams as it was proposed in the former version of this standard. To convenient with the MC 2010 [3], EFNARC produced a document [10] advising the use of three point bending tests on panels with notch having the same geometry as EN 14488-5 [9] and recommending a correlation between the EFNARC [10] notched panel test and the EN 14651 [6] notched beam test. Following that, European sprayed concrete standard EN 14487-1 [7] was recently updated to include the three-point bending notched panel test, as described in Method B of current EN 14488-3 [8], which is becoming the reference method for Sprayed Concrete Lining (SCL) fibre reinforced. It is worthwhile noticing that unlike temporary sprayed concrete, PSCL must have the same service lifetime as a typical tunnel or underground facility [2]. In the recent years, several types of macro-synthetic fibres, adequate for structural purposes, have been introduced into the market as an alternative solution to steel fibres, especially in aggressive environments as in case of underground structures [11, 12, 13].

Within this framework, concrete reinforced with macro-synthetic fibres (MSFRC) were considered. Two fibre types and contents were used to cast square panels. The effect on the residual strengths with varying fibre dosages and varying aspect ratios are discussed. For further exploration into the post-cracking performance to inform the design process of permanent sprayed fibre reinforced concrete lining, in addition to the characterization of FRC, the subsequent determination of post-cracking uniaxial constitutive laws are fundamentals. The latter are expected to be used in numerical modelling of the embedded in ground condition of Permanent Sprayed Concrete Lining (PSCL) making necessary to provide accurate laws. Despite to the importance of these constitutive relationships, to the best of the Authors knowledge it is still a matter of discussion within the scientific community and among practitioners the procedure to get concretes' uniaxial post-cracking laws in case of FRC panels. The inverse analysis method, typically used in case of standard beam notched samples, is herein proposed to be applied on results retrieved by square notched panels' tests.

The main goal is to achieve the most accurate description of FRC post-cracking performance by also addressing opportune criteria to compare the fracture energy exhibited by MSFRCs based on panel tests with notch.

## 2 EXPERIMENTAL PROGRAM

### 2.1 Materials

The target concrete class of strength is C35 according to MC 2010 [3]. Two types of structural macro-synthetic fibres were used, namely MB 48 and BG 55, both featuring an embossed surface made of polyolefin, having the same diameter and different lengths (Table 1). Thus, two different aspect ratios 60 and 69, were considered to study the effect of this parameter on the post-cracking response of MSFRCs under investigation.

Two different fibre contents were selected: 5.5 kg/m$^3$ and 7.7 kg/m$^3$, corresponding to volume fractions ($V_f$) of 0.60% and 0.85%, respectively. The combination of the previously mentioned parameters defines a specific series, identified according to the notation MS-X-Y, where: MS stands for macro-synthetic, X represents the fibre length and Y denotes the fibre dosage in kg/m$^3$. Four concrete batches were prepared, one for each series (Table 1). In any concrete batch two square panels (600 mm side) were cast for being tested according to EN 14488-3 [8]. The age of the specimens at the time of testing was about 35-41 days. After these tests, cylindrical core samples were directly drilled from panels to measure concrete compressive strength according to EN 12390-3 [14]. The corresponding mean compressive strengths ($f_{cm}$) at the age of testing panels are reported in Table 2. Based on $f_{cm}$, taking advantage of relationships suggested in current Eurocode 2 [15], the elastic modulus ($E_c$) and the mean tensile strength ($f_{ctm}$) were estimated, as reported in Table 2.

*Table 1. Main properties of fibres adopted and dosages in the batches prepared.*

| | **MS-48-5.5** | **MS-48-7.7** | **MS-55-5.5** | **MS-55-7.7** |
|---|---|---|---|---|
| Shape | Flat embossed | Flat embossed | Flat embossed | Flat embossed |
| Length, $L_f$ [mm] | 48 | 48 | 55 | 55 |
| Diameter, $\phi_f$ [mm] | 0.8 | 0.8 | 0.8 | 0.8 |
| Aspect ratio, $L_f/\phi_f$ [-] | 60 | 60 | 69 | 69 |
| Elastic modulus, $E_f$ [MPa] | 6000 | 6000 | 6000 | 6000 |
| Filament tensile strength [MPa] | 525 | 525 | 525 | 525 |
| Fibre dosage [kg/m$^3$] | 5.5 | 5.5 | 7.7 | 7.7 |

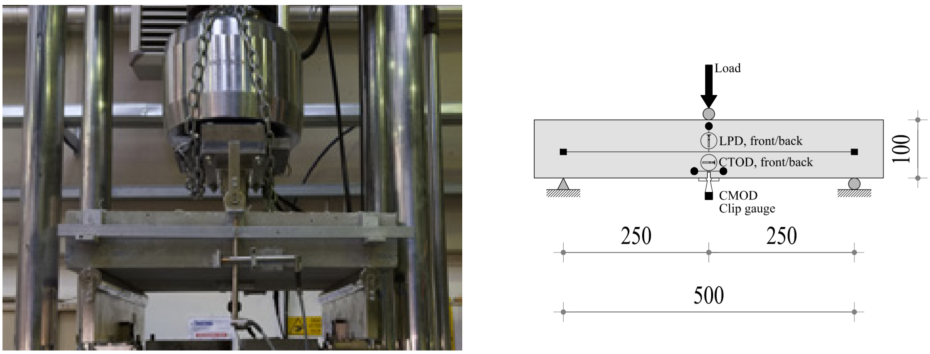
*Table 2. Main mechanical properties of MSFRCs tested.*

| **Series** | **Age [days]** | **$f_{cm}$ [MPa]** | **$f_{ctm}$ [MPa]** | **$E_c$ [MPa]** |
|---|---|---|---|---|
| MS-48-5.5 | 35 | 41.1 | 3.08 | 33600 |
| MS-48-7.7 | 36 | 43.9 | 3.25 | 34250 |
| MS-55-5.5 | 35 | 42.9 | 3.19 | 34000 |
| MS-55-7.7 | 41 | 48.9 | 3.55 | 35400 |

## 2.2 Specimen geometry and test set-up

The specimen geometries and testing configurations followed the guidelines outlined in the relevant standard. Figure 1a exhibits the test setup adopted for all panels, while in Figure 1b the instrumentation details are shown. All experiments were performed using an INSTRON 1274 closed-loop servo-hydraulic machine with a loading capacity of 500 kN, as shown in Figure 1a. The tests were carried out under displacement control according to EN 14487-1 [7] and EN 14488-3 [8]. They were performed on notched square panels (600 mm side) having a depth of 100 mm. The notch depth is 10 mm resulting in an effective depth at the mid-span ($h_{sp}$) equal to 90 mm. The span between the supporting rollers is 500 mm (Figure 1b).

The Crack Mouth Opening Displacement (CMOD) was measured through a clip gauge (Figure 1b). To provide detailed information for inverse analysis method, the deflection at the middle of the panel (Load Point Displacement, LPD) and the Crack Opening Displacement at the Tip of the notch (CTOD) were measured at both the front and back sides of the panel (Figure 1b). Linear Variable Differential Transducers (LVDTs) were utilized to measure CTODs and LPDs.




(a) (b)

*Figure 1. Test set-up (a) and instrumentation details (b) of square panels with notch tested according to EN 14488-3 [8] (dimensions in mm).*

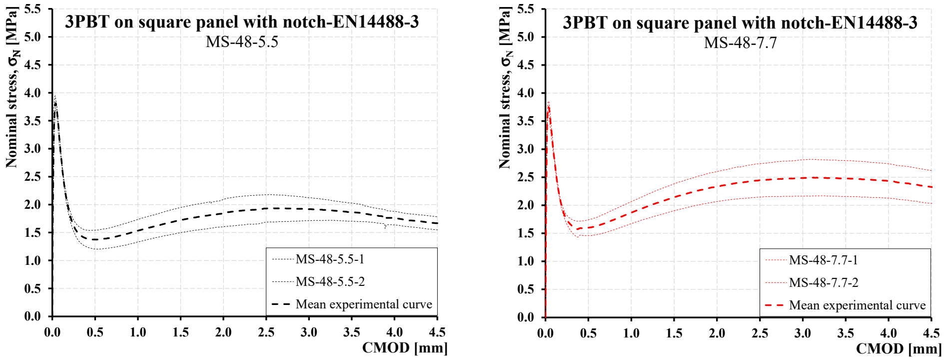
The tests were carried out by using the CMOD as a control parameter (up to 4.5 mm). Two loading rates were employed, namely 0.05 mm/min (up to CMOD=0.1 mm) and 0.2 mm/min (up to CMOD=4.5 mm); afterwards, tests continued under LPD control until descending to 1% of the maximum load, or reaching a CTODm (mean value of front and back CTOD) of about 8 mm. During the first two minutes, all the measurements were recorded at a rate of 5 Hz; afterwards it was reduced to 2 Hz.

## 3 EXPERIMENTAL RESULTS

In Figure 2a and b, as an example, results of series MS-48-5.5 and MS-48-7.7 are respectively reported in terms of nominal stress, $\sigma_N$ vs. CMOD. Note that curves of each panel tested are plotted together with the corresponding mean experimental curve, which aims to describe the average flexural response exhibited by MSFRC panels belonging to a given series. It is worth noticing that these diagrams are based on $\sigma_N$, which is calculated assuming a nominal linear distribution of stresses along the mid-span cross-section. Hence $\sigma_N$ is useful for describing or comparing the post-cracking flexural strengths, but it cannot be used directly as a uniaxial post-cracking law parameter.

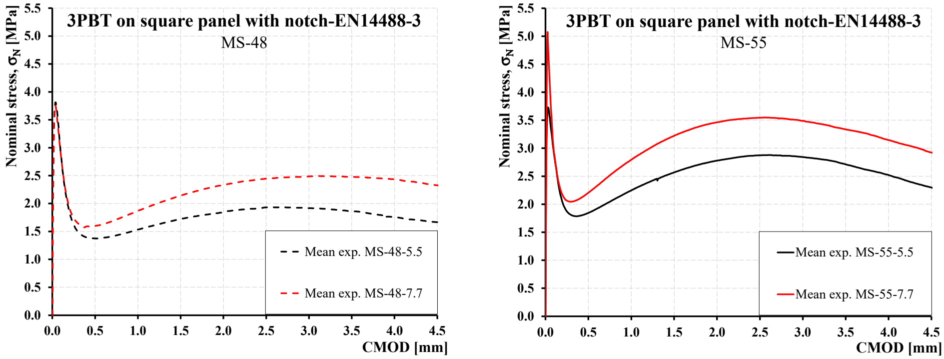
Macro-synthetic fibres with varying fibre lengths and dosages were examined, considering about a 15% change in fibre length (from 48 mm to 55 mm) and a 40% change in fibre dosage (from 5.5 kg/m$^3$ to 7.7 kg/m$^3$). Since the fibres present the same diameter (Table 1), the change in fibre length results in the same entity in terms of fibre aspect ratio (from 60 to 69, Table 1).

In Figure 3, for a given fibre type (or fibre aspect ratio) the effect of fibre dosage is investigated by comparing mean experimental curves; as expected, by increasing the fibre content the post-cracking strength increases. More in details, in case of MS-48 fibre (Figure 3a), the residual stresses exhibited by MS-48-7.7 in the CMOD range of 0.5-3.5 mm are on average about 1.26 times those of MS-48-5.5. Similar trend is appreciated for MS-55 fibre (Figure 3b); in the same CMOD-range, MS-55-7.7 series presents post-cracking strengths on average 1.24 times those of MS-55-5.5, resulting from 40% change in fibre content.




(a) (b)

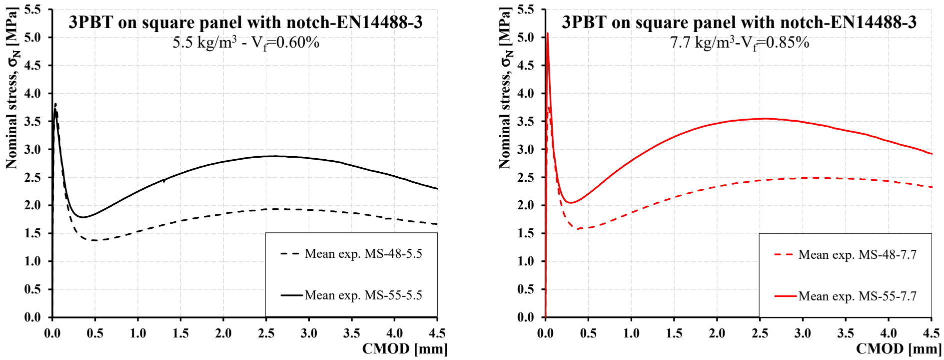
*Figure 2. Experimental results of MS-48 panels with evidenced mean experimental curves: 5.5 kg/m$^3$ ($V_f$=0.60%) (a) and 7.7 kg/m$^3$ ($V_f$=0.85%) (b).*




(a) (b)

*Figure 3. Comparison of mean experimental curves in terms of fibre contents for a given fibre type: MS-48 panels (a) and MS-55 panels (b).*

In Figure 4, for a given fibre content the effect of fibre aspect ratio (60 vs. 69 for MS-48 vs. MS-55) is investigated by comparing mean experimental curves: by increasing $L_f/\phi_f$ the post-cracking strength increases. More in details, in case of 5.5 kg/m$^3$ (Figure 4a), the residual stresses exhibited by MS-55 ($L_f/\phi_f$=69) in the CMOD range 0.5-3.5 mm are on average about 1.48 times those of MS-48 ($L_f/\phi_f$=60). Similar trend is appreciated for 7.7 kg/m$^3$ (Figure 4b); in the same CMOD-range, MS-55 series presents post-cracking strengths on average 1.45 times those of MS-48, as result of a 15% change in fibre aspect ratio.




(a) (b)

*Figure 4. Comparison of mean experimental curves in terms of fibre aspect ratio for a given fibre content: 5.5 kg/m$^3$ ($V_f$=0.60%) (a) and 7.7 kg/m$^3$ ($V_f$=0.85%) (b).*

From the $\sigma_N$–CMOD curves of each panel tested, the limit of proportionality (LOP) $f^f_{ct,Ls}$ can be determined as reported in EN 14488-3 [8]. Similarly, the residual strengths ($f_{R,1s}$, $f_{R,2s}$, $f_{R,3s}$, and $f_{R,4s}$) are defined by considering the CMOD values of 0.5 mm, 1.5 mm, 2.5 mm and 3.5 mm, respectively. For a given series, the corresponding mean values that can be evaluated are summarized in Table 3.

In agreement with tendencies observed in Figure 3 and Figure 4, by considering $f_{R,3ms}$, in case of MS-48 fibre the MS-48-7.7 series presents a value 1.26 times that of MS-48-5.5. The MS-55-7.7 series shows a $f_{R,3ms}$ value 1.23 times that of MS-55-5.5 (Table 3). Considering a given fibre content of 7.7 kg/m$^3$, once again, $f_{R,3ms}$ value of MS-55 series ($L_f/\phi_f$=69) is 1.45 times that of MS-48 ($L_f/\phi_f$=60), confirming previously described trends.

*Table 3. Mean values of residual flexural tensile strengths according to EN 14488-3 [8].*

| Series | $f^f_{ctm,Ls}$ [MPa] | $f_{R,1ms}$ [MPa] | $f_{R,2ms}$ [MPa] | $f_{R,3ms}$ [MPa] | $f_{R,4ms}$ [MPa] |
|---|---|---|---|---|---|
| MS-48-5.5 | 3.84 | 1.37 | 1.72 | 1.93 | 1.86 |
| MS-48-7.7 | 3.76 | 1.60 | 2.14 | 2.44 | 2.48 |
| MS-55-5.5 | 3.74 | 1.84 | 2.57 | 2.87 | 2.71 |
| MS-55-7.7 | 5.08 | 2.20 | 3.22 | 3.54 | 3.34 |

## 4 POST-CRACKING UNIAXIAL CONSTITUTIVE LAWS

The stress crack width (σ-w) laws of MSFRCs were evaluated through the inverse analysis procedure [16] by assuming a multi-linear post-cracking cohesive law (according to the fictitious crack model, FCM [17]) and by performing non-linear numerical analyses to get the best-fitting law. The numerical analyses were carried out by using a discrete crack approach; the FE mesh was made by triangular plane stress elements and interface elements for the crack (Figure 5). A proper mesh refinement was adopted at mid-span to capture non-linear post cracking phenomena. Accordingly, forty-six interface elements were used along $h_{sp}$, as shown in Figure 5 with the red line.

To successfully conduct the inverse analysis, the numerical response is fitted to the mean experimental curve with the minimum error as possible. In this regard, by means of a trial-and-error procedure, the fracture parameters used in the finite element model were varied until the difference between the curves is below a certain tolerance: a maximum percentage error of 3% was assumed in the CMOD range 0.5-3.5 mm. A typical best-fitting numerical curve with respect to mean one is reported in Figure 6a for MS-55-7.7 series: it can be observed the very good agreement. To further check the reliability of the numerical model adopted, based on eight notched panels tested, the corresponding CTOD vs. CTOD curves were plotted (Figure 6b) to retrieve an experimental trend line, shown in red in Figure 6b. The same trend line can be plotted using numerical results, as superimposed in Figure 6b (red dotted line). It can be noticed a really good agreement between trend lines; the mean absolute percentage difference is less than 2% for CMOD values higher than 0.5 mm, which confirm the goodness of FE model used.

It is worth noting that, together with the robust numerical model adopted, a proper shape of uniaxial post-cracking law was chosen. In this regard, a post-cracking three linear branches law was used to capture the typical behaviour exhibited by MSFRC (Figure 7a). The mean tensile strength ($f_{ctm}$) of each series and elastic modulus ($E_c$) were assumed in accordance with values reported in Table 2, while the slope of the first post-cracking branch (up to $\sigma_1$-$w_1$, Figure 7a) was assumed similar to that expected for plain concrete. The post-cracking laws of all series investigated are shown in Figure 7b and their main parameters are summarized in Table 4.

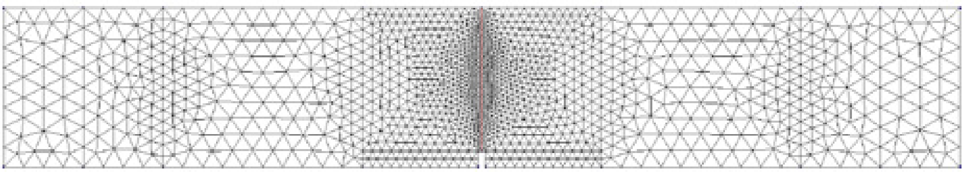
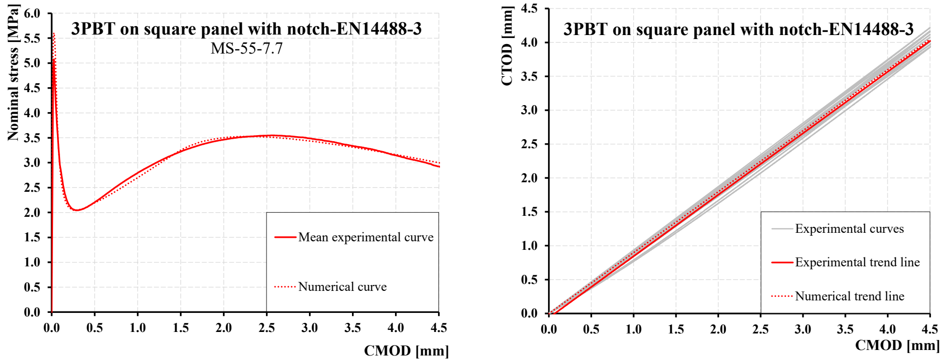


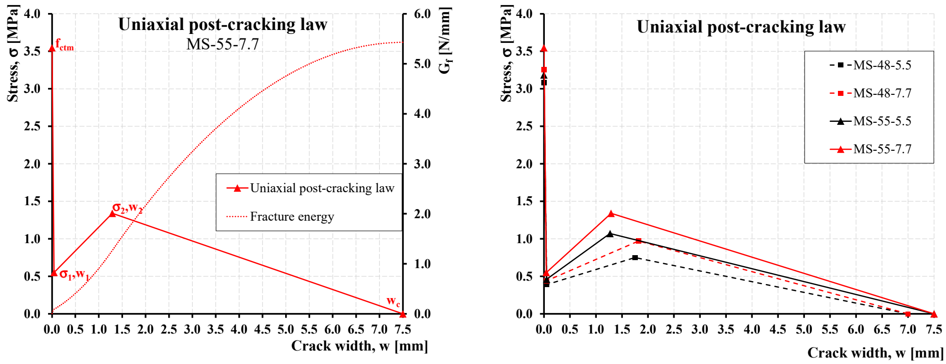
*Figure 5. Mesh adopted with evidenced interface elements used for the discrete mid-span crack.*

To address opportune post-cracking performance criteria, the fracture energy $G_f(w)$ retrievable by uniaxial constitutive law was also considered. The latter represents the area under the multi-linear curve up to the to a given crack width (w); the $G_f(w)$ function is plotted in Figure 7a for series MS-55-7.7. The fracture energies at two fundamentals level of crack widths equal to 0.5 mm and 2.5 mm ($G_{f,0.5}$ and $G_{f,2.5}$) were also calculated, while the total fracture energy ($G_{f,tot}$) was evaluated at the ultimate crack opening ($w_c$). The corresponding values are collected in Table 4.




(a) (b)

*Figure 6. Comparison: mean curves experimental and numerical as retrieved through inverse analysis for MS-55-7.7 series (a) and numerical vs. experimental trend lines CTOD vs. CMOD (b).*




(a) (b)

*Figure 7. Typical uniaxial post-cracking law with evidenced main parameter and development of $G_f$ (a) and post-cracking uniaxial constitutive laws of all series investigated (b).*

In accordance with the flexural response studied in Section 3, similar tendencies could be evidenced from the uniaxial constitutive laws retrieved through inverse analysis procedure (Figure 7b). by considering the parameter $G_{f,2.5}$ of series MS-48-7.7 is 1.23 times that of series MS-48-5.5. Similar trend can be point out for MS-55-7.7, those $G_{f,2.5}$ is 1.24 times that of series MS-55-5.5. It is worth noting that these values are completely aligned with those reported for $f_{R,3ms}$ in Section 3, but they regard uniaxial tensile laws for use in numerical analyses of PSCL, not simple nominal stress $\sigma_N$.

*Table 4. Main parameters of the σ-w laws obtained from inverse analysis.*

| Series | $f_{ctm}$ [MPa] | $\sigma_1$ [MPa] | $w_1$ [mm] | $\sigma_2$ [MPa] | $w_2$ [mm] | $w_c$ [mm] | $G_{f,0.5}$ [mm] | $G_{f,2.5}$ [mm] | $G_{f,tot}$ [mm] |
|---|---|---|---|---|---|---|---|---|---|
| MS-48-5.5 | 3.08 | 0.39 | 0.054 | 0.75 | 1.75 | 7 | 0.29 | 1.58 | 3.03 |
| MS-48-7.7 | 3.25 | 0.44 | 0.048 | 0.97 | 1.82 | 7 | 0.32 | 1.95 | 3.85 |
| MS-55-5.5 | 3.19 | 0.46 | 0.051 | 1.07 | 1.27 | 7.5 | 0.35 | 2.21 | 4.36 |
| MS-55-7.7 | 3.55 | 0.55 | 0.044 | 1.34 | 1.29 | 7.5 | 0.41 | 2.73 | 5.43 |

# 5 CONCLUDING REMARKS

The present paper concerns the post-cracking performance exhibited by Fiber Reinforced Concretes for use in permanent sprayed concrete linings (PSCL) and a proposed procedure for the evaluation of their corresponding uniaxial post-cracking laws. Two types of macro-synthetic fibres and two fibre dosages were considered for cast MSFRCs panels to be tested according to EN 14488-3, which is becoming the reference testing methodology for the mechanical characterization of FRC used in PSCL.

Based on the results and the discussion presented, the following main concluding remarks can be drawn:

1) for both fibre types adopted (MS-48 and MS-55), the residual stresses exhibited when using 7.7 kg/m$^3$ are approximately 1.25 times those of 5.5 kg/m$^3$, with a change in 40% of fibre content;
2) for both fibre contents (5.5 and 7.7 kg/m$^3$), the residual stresses exhibited when considering a higher fibre aspect ratio (MS-55, $L_f/\phi_f$=69) are approximately 1.47 times those of MS-48 ($L_f/\phi_f$=60), with a change in 15% of fibre aspect ratio;
3) the inverse analysis approach proposed for evaluating MSFRC uniaxial post-cracking laws is effective and provide accurate σ-w relationships for use in structural numerical simulations of sprayed concrete tunnel linings;
4) the post-cracking performance criteria based on fracture energy, $G_f$, retrievable by σ-w laws provides tendencies aligned with those observed from flexural responses of MSFRC notched square panels.

## ACKNOWLEDGMENTS

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9th International Symposium on Sprayed Concrete

# Performing Energy Absorption Test Panels for Fibre Reinforced Sprayed Concrete

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## Abstract

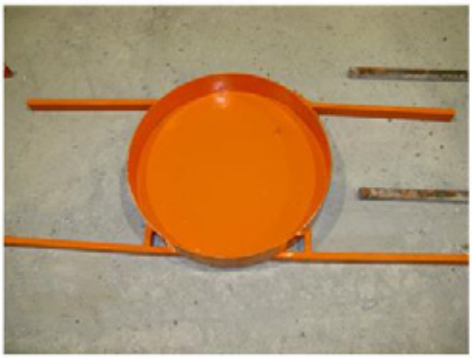
The requirements for functional testing of fibre reinforced sprayed concrete can vary within countries. Some perform the functional control on beams, casted or sprayed. Most common is to test on panels, either round or square. Common for all test panel execution is to produce the test panels in a way which does not affect the result in a negative way. This presentation is meant to be a recommendation of how to perform the test panel spraying, how to set up the test panel forms and how to handle the filled test panels in the critical minutes and hours after spraying.

## Introduction

In Norwegian Concrete Association's Publication no. 7 (NB7) [1] the method for how to execute test panels for Energy absorption (E-abs) is described. In Norway it is free to choose if you want to produce square or circular shaped test panels. Almost 100 % are produced as round test panels, with thickness of 100 mm and 600 mm diameter.

Theoretically, the concrete volume of one circular test panel will be 28 litres and if sprayed concrete has a density of 2 300 kg/m$^3$, one test panel will have a mass of 65 kg. Compared to a square test panel with dimensions 0.6 x 0.6 x 0.1 meter thickness, a square sample will have a mass of 83 kg.

Forms for Energy absorption test panels (EATP) must be made in metal material. These can be re-used many times and is not a huge investment.



Picture 1 and 2: Circular Energy-absorption test panel forms [1].

**Planning**

It is wise to plan the execution of the production of EATP well before the process starts. Make sure the concrete producer is involved and concrete producer should take samples and measure fibre content and fibre distribution in the fresh concrete, according to method described in NB7. If the variation in fibre content is huge, it is wise to not use this concrete load for producing the EATP. Then a new test should be planned, and it is recommended to modify the method of how to dose and mix in the fibres into fresh concrete.

In Norway, it is normal to test and document E-abs by spraying two loads of concrete with different fibre content, and being able to interpolate between the two obtained results [1]. The dosages do not have to result in 700 and 1000 Joules, but it is wise to obtain a result below 700 J with low dosage and a result above 1000 J with high dosage. In this way, it is possible to interpolate between the two results and calculate the required dosage for obtaining 700 and 1000 J.

An E-abs test of 2 x 3 EATP has cost of ~ 3 100 € (for two series of three panels in each), plus work to produce the test panels. If a failure result is obtained, it takes some days to plan and execute a new test and it also involves waiting for age 28 days before the nest E-abs test can be done. Therefore, make sure the EATP are well produced and homogenous fibre distribution is obtained in the concrete.

**Execution**

1. Be prepared and make sure enough people are involved and everyone knows what to do.
2. For documenting E-abs in one concrete batch, it is needed to have minimum three test panels in one series. Make sure to have equipment for scraping off the remaining concrete in the top of the form, and spray paint to mark the sample and plastic sheet and tape to wrap up the test panel.
3. Before spraying the first panel, it is wise to spray ~1 – 1.5 m3 of concrete on the rock and see that everything is functioning normal. Then, before the first panel is sprayed, it is important to adjust both the concrete pump rate and accelerator dosage to a level which is suitable for spraying the test panel.
   Accelerator dosage should be significantly lower than during normal spraying, to avoid too fast setting. Accelerator dosage must be reduced by maybe 50 – 70 % from normal dosage rate.
   Also, the concrete pump can not be operated on full speed and a pump rate should be maximum 15 m3/h.
   Finding the correct accelerator dosage and pumping speed can be a bit tricky, but it is important to optimize these two factors before the samples are executed. Do not rush and take the time to do proper work.
4. Spray just one EATP at a time and prepare the EATP-sample and finish all the preparation, before the next EATP is sprayed.
5. Make sure the spraying and filling of the EATP-form is done in one operation. Always start spraying on the rock surface near the EATP and move the concrete spray onto the EATP. Start to fill the EATP in the low part first and move slightly upwards. This is a good way to avoid rebounded particles from aggregates to be sprayed into the test panel. Distance from nozzle to EATP-form should be ~1.5 meter. See pictures 3 and 4 in next chapter.
6. NB7 recommends spraying EATP while the form is standing in an angle of ~15° from the vertical tunnel wall.
7. Further it is written in NB7 that the EATP should be left just scraped off and wrapped in plastic at the execution location for minimum 18 hours, before the EATPs can be moved. This is not always possible to do, while the EATP can be “in the way” of the further tunnel

excavation. Therefore, it can be wise to move the freshly sprayed EATP and prepare them while lying down flat on the ground.
If remaining concrete (thicker than 100 mm) is scraped off while the EATP is raised up against the tunnel wall, there is a risk that the scraping can cause the concrete to slip slightly downwards and can cause the test specimen to not be perfectly circular, and slightly thicker in the lower part compared to the upper part.

8. A functional way of doing a series of three test panels with one fibre dosage is to spray one EATP after spraying 1 – 1.5 m3. Then two operators can carefully loose the EATP from the wall and carry it carefully to a place somewhat behind the execution location. There, the EATP is placed on a pallet, called working pallet. In this position (lying down flat on the pallet), the EATP is prepared and remaining concrete (thicker than 100 mm) is scraped off. Then the sample is marked with spray paint and wrapped in plastic sheets. Then the EATP is moved carefully and placed onto another pallet. See pictures 5 – 8 in next chapter.
9. Then the process of 8. is done for the second EATP and after spraying, the EATP is carried carefully to the working pallet. The same procedure of scraping off remaining concrete, marking and wrapping in with plastic sheets. Then the second EATP is moved carefully and placed on top of the first EATP.
10. The process is repeated once more and after having three freshly sprayed EATP placed on top of each other, one free pallet is placed on the top and this "pile" and is strapped together with pallet in the bottom. Then "a sandwich" of pallet, three EATP and pallet on top is made. See picture 9 and 10 in the next chapter. Then leave this "sandwich" at a safe place in tunnel or move it carefully out of tunnel.
11. The nozzle-man is recommended to wait between the spraying of each EATP. In this way, the operators who are preparing the EATP is not becoming stressed and afraid of not working fast enough to reach the remaining two EATP before the load of concrete is completely sprayed.
12. If the scraping of remaining concrete (thicker than 100 mm) is done sufficient, placing three and three EATP onto each other should not do any harm. Then the backside of the above EATP should rest against the 100 mm thick circular wall on the below EATP.
13. If something strange or irregular is observed during spraying of an EATP, spraying process should be aborted, EATP-form should be emptied, and a new sample should be sprayed. Possible irregularity could be:
    a. Larger amounts of rebound observed in the test panel.
    b. Stop in the spraying process, which can lead to deviation between layers of concrete in the sprayed test panel.
    c. If accelerator dosage is at the lower amount, the concrete can be observed to slip slightly down while the EATP-form is raised up against the wall. Then accelerator dosage must be adjusted.
    d. If un-even distribution of fibres is clearly observed.
    e. ….

    Do not hesitate to abort, empty the EATP-form and spray a new sample, if any possible problem is observed.

14. The freshly sprayed samples should be protected from frost and must be stored in 28 days before they are tested for E-abs.
15. If possible, the test specimens can be left inside the EATP-forms for a week or more, as long as they are well wrapped into the steel formwork and plastic sheets. NB7 [1] requires the E-abs test panels to be stored in water for 3 days before E-abs is tested. This means that the test specimens don't need to be transported to the test lab, until ~one week before testing the E-abs at age 28 days.
16. Concrete producer is responsible for measuring fibre content and fibre distribution. According to NB7 [1] this is done by taking out three samples of fresh concrete. Sample volume should

be minimum 8 litres for each individual sample. Required fibre content should be ± 20 % for an individual sample and ± 15 % for average of a series of three samples from one concrete load. The % deviation is based on deviation from the theoretical fibre content which is added to the concrete load. The limits of ± 20 % (individual sample) and ± 15 % (average of all three samples) are quite wide. If your average result is more than 10 % off the theoretical dosage, do consider performing a new test. The risk is to spend money on a test report which show low E-abs-results and time of ~4 – 5 weeks are also lost.

**Picture gallery**

Picture 3 and 4: Moving the spraying nozzle sideways from wall and into the EATP-form, starting at the lower part. Then moving upwards and filling the EATP-form completely.

Picture 5 and 6: Loose the EATP-form from the wall after spraying, and then carry it manually and carefully to the location where samples are treated.

Picture 7 and 8: Scraping off remaining concrete (thicker than 100 mm) and marking and wrapping in sample with plastic sheet.

Picture 9 and 10: Placing three EATP-forms with sprayed concrete together in a “sandwich”, with pallet under and on top.

## Conclusion

To conclude the practical work of spraying and preparing Energy Absorption Test Panels (EATP) is not very scientific, but many years of practical experience has provided some experience and knowledge.

In a conclusion, it is worth repeating the major and most important factors of success:

- Do propper planning and be well prepared.
- Spray one and one EATP at a time. Then scrape off remaining concrete (thicker than 100 mm) and mark sample and wrap in with plastic sheet.
- Adjust concrete pump (max. 15 m3/h) and accelerator dosage (reduction of 50 – 70 % of normal dosage).
- Spray EATP with a distance of ~1.5 meter from nozzle.
- If any observations indicate that there are any problems or challenges, do abort the spraying and empty the EATP and spray a new sample.
- Concrete producer is supposed to take three samples of fresh concrete and measure fibre content. If fibre content is not homogenous, do not use the sprayed EATP for Energy absorption (E-abs) test. Then do modifications to the way fibres are added and mixed into concrete and perform new spraying test.

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# INNOVATIVE CROSS PASSAGE DESIGN & CONSTRUCTION USING SPRAYED CONCRETE AND GROUND FREEZING BETWEEN TWO HS2 TBM-DRIVEN TUNNELS

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**ABSTRACT**

This paper presents an integrated method for safely constructing cross passages in challenging geological conditions between two Tunnel Boring Machine (TBM)-driven running tunnels on the HS2 Project in the UK. There are a total of 20 cross passages to be constructed, with 13 utilising Artificial Ground Freezing (AGF) and 6 employing dewatering and depressurisation methods. Conventionally, AGF has been utilised primarily for rescue operations and has gained wider adoption outside the UK in recent years. However, its application within the UK has been limited due to cost, time constraints, and insufficient knowledge. The implementation of AGF on this project marks it as the most extensive AGF application undertaken on a single project within the UK.

## 1. INTRODUCTION

The cross passages are constructed concurrently with the bored tunnel operation and beneath an urban environment. This necessitates meticulous coordination and communication for the safe delivery of the project. The paper examines the use of ground freezing technology to establish a stable working environment for cross passage excavation, covering setup through to decommissioning. It discusses the design and execution of the ground freezing process, utilising a network of freeze pipes installed around the cross passages' perimeter. Following the establishment of frozen ground, construction proceeds with a sprayed concrete lining technique, offering rapid application of primary support and early structural ground support. Emphasis is placed on material selection, mix design, application methodology, and quality control to ensure compliance with requirements. The paper explores innovative approaches and lessons learned during the cross-passage construction, including reduced lining thickness and the development of early age strength access requirements in frozen ground. It also emphasizes the adoption of lean processes to maintain efficient cycle times.. Comparative analyses of construction time, cost-efficiency, and structural performance underscore the advantages of the integrated approach over alternative methods considered during the design phase.. The insights in this paper contribute to enhancing safer and more sustainable tunnel construction practices, benefiting engineers, contractors, and stakeholders engaged in underground infrastructure development including major projects.

## 2. GEOLOGICAL CONTEXT

The cross passages are constructed through the geology of the London Basin, principally the Thames Group, the Lambeth Group and the underlying Seaford Chalk. The majority of the cross passages are excavated through the notoriously variable Lambeth Group, comprising in this location stiff clays, with water bearing sand and silt horizons of varying thicknesses. Figure 1 provides the geotechnical long section.

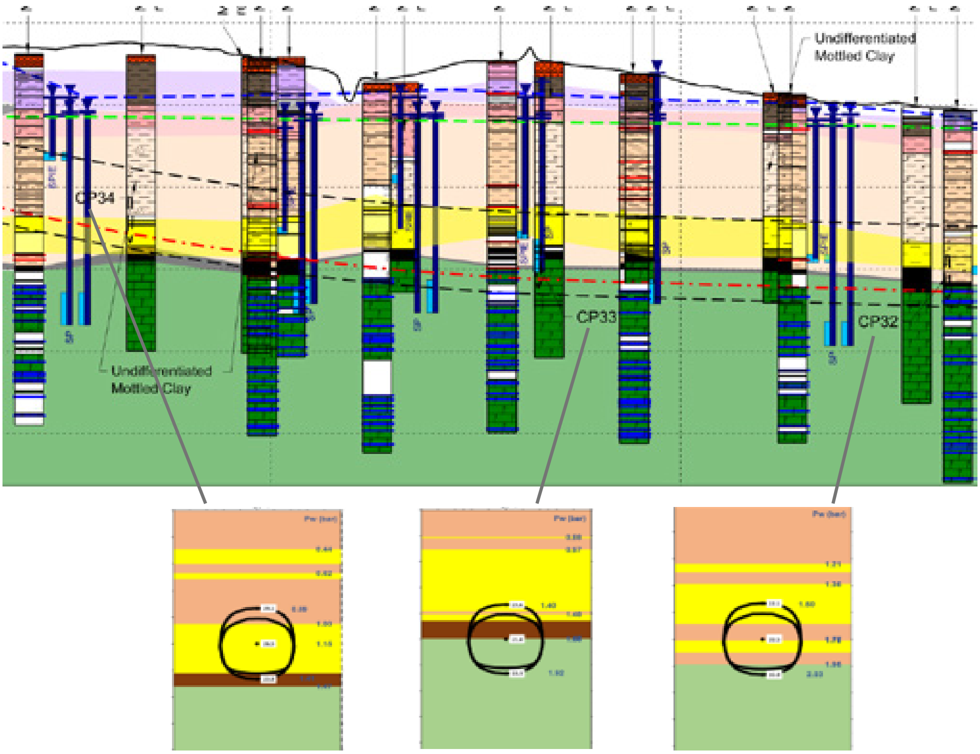



*Figure 1 – Alignment of a number of CPs along the 8km route*

## 3. DESIGN

The ethos of the design was to stabilise the granular geology and mitigate against water ingress, without adversely impacting the TBM segments. The chosen method for construction of the cross passages was a Sprayed Concrete Lining (SCL) lined with a sheet membrane for waterproofing, followed by a cast in place secondary lining. Following development of the geological model, the design specified that any granular geology above axis level of the tunnel would be treated to be stable for excavation. Ground Freezing was selected as the chosen method of stabilization. The design specified a minimum 2m freeze zone, shown in Figure 2, with an average temperature of -10ºC within the frozen body. To verify the frozen soil properties, in situ lab testing was undertaken. The results confirmed the strength properties of the soil, permeability, and frost pressures that the soil would exert on the lining. To achieve a freeze zone around the cross passage, arrays of freeze pipes needed to be drilled through the TBM segmental lining. Drill zones were specified through the steel reinforced pre-cast segmental lining. Segment). The drill zones for the freeze pipes had to ensure that embedded steel within the opening set segments was not damaged.

Using a risk-based approach it was decided to apply a pre-determined treatment/dewatering approach for each cross passage. Following further consultation with an 'HS2 Independent Technical Expert Panel' full freeze rings rather than half rings (in combination with dewatering) were specified at all cross passages. This would remove any interaction between the different techniques and remove a complex interface around the cross passage opening. This approach needed the AGF arrays to be modelled for each cross passage, taking into account the position and roll of the bored tunnel.

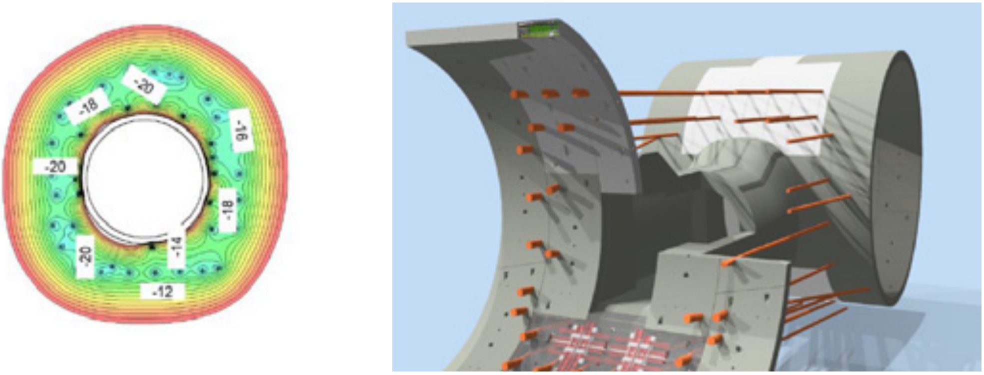



*Figure 2 – Freeze zone with a minimum temperature of -10°C (LHS), AGF at CP (RHS)*

*Comparative Analysis*

The initial design concept incorporated a hybrid approach, employing freezing in the crown and dewatering or depressurisation in the invert; 2no. full freezing, 10no. hybrid, and 7no. dewatering CPs. However, leveraging lean processes, risk management, and peer reviews, the design was optimized. Consequently, the revised scope discarded the hybrid option and now comprises 13 full freezing and 6 dewatering/depressurisation CPs.

CP production team undertook analysis of the proposed CP workflow using lean/ six sigma tools, resulting in some notable results and efficiencies. Regression analysis was used to study the time-activity relationship for dewatering and ground freezing planned for the original hybrid solution (applicable to 10no. cross passages). This resulted in (1) reduced the timeline on 10 CPs from 170 days each to 140 days each, and (2) improvement in programme critical path and (3) removal of hybrid solution. From both a commercial and safety standpoint, the decision was taken to separate the ground freezing process to the Upline tunnel and the dewatering/depressurisation process to the Downline tunnel. This segregation minimises the interaction between subcontractors, thereby mitigating any interface risks.

**Mix Design, Testing and Trials**

*Frozen ground sprayed concrete best practice and targets*

The project team engaged with Normet UK in a very early stage to assess the sprayed concrete options, as Normet's team have a wealth of experiences from successful application of wet-mix sprayed concrete from the few international projects, such as the Deep Tunnel Sewer System in Singapore (Klados 2002), the Oslo Fjord Tunnel and the Hull Wastewater Transfer Tunnel in UK (Brown 2004 and Eddie 2002) . The main targets for the successful application of sprayed concrete onto frozen substrates are to have as much thermal heat in the mix to support concrete setting and early age strength development consistent with the J2 requirements. To do this the key approaches based on experience were:

- To increase mix temperature to min 25°C using hot water and potentially aggregates for batching.
- To use reactive binders such as CEM I or similar
- To increase the cement content to around 450-480kg/m3
- Target water: binder of 0.42 max and a pumpable mix with target flow 650mm
- To accelerate the concrete with high performing alkali-free set accelerators that create early reactions with the cement and generate heat to initiate setting and continue to support early age strength development. Maintain accelerator at 20°C storage temperature.
- The use of hydration control admixture to maintain the cement-accelerator reactivity during transit from batching plant to the time of concrete spraying.
- Adopt the latest sprayed concrete equipment to ensure homogenous material application through low pulsation pumping and accurate synchronised accelerator dosing at the spraying nozzle.

*Unique concrete batching approach*

The construction of the 20 cross passages of 4m – 5.2m dia. requires relatively small quantities of sprayed concrete to be batched and delivered.. The project decided to opt for well-proven dry silo mix (DSM) developed by Cemex-Normet for tunnelling projects in the UK. This negated the need for bespoke site batching plants and allows to produce concrete for spraying on a 24/7 basis all contained within the site. Essentially the DSM solution is a blend of separately controlled aggregates, cement, fibres, additions and additives that are pre-blended off site, delivered as a powder product in road-based bulk tankers, which fill by blowing into 4 sites based vertical 40tonne silos. All the materials conform to European Sprayed Concrete standards and also the HS2 Specification. The DSM sprayed concrete mix design for this application was similar to that used on a number of UK tunnelling projects for both permanent and temporary ground support, and is summarised in Table 1.

Pre-construction spraying trials were conducted on the surface and two set accelerator dosages of 6% and 8% by weight of binder were used to prepare test panels in accordance with EN14488-2. The positive impact of batching a relatively high cement content, with a water cement ratio of 0.42, a mix target temperature of 25°C and addition of the high-performance alkali-free set accelerators at the nozzle demonstrates how upper J2 and J3 early age strength performance can be achieved. This performance is what was required to assure safety in the frozen tunnel environment where overhead application and bonding to the frozen substrate was required, including the absence of steel mesh reinforcement.

| Mix Component | Description | kg/m3 |
|---|---|---|
| **Cement** | CEM I 52.5N | 450 |
| **Aggregates and sand** | Crushed limestone to project grading (0-4mm sands, 2-6mm aggs) | 1630 |
| **Structural fibres** | High tensile steel fibres 75/35 | 35 |
| **Additions** | Silica fume | 30 |
| **Additives** | Powder PCE based performance superplasticiser and stabilisers | 2.5 |
| **Mix Water** | Heated to 60°C, target w/c 0.42 | 190 |
| **Hydration control** | TamCem HCA liquid admixture added with mix water at silo contiguous mixer | 0.8% by wt binder |
| **Alkali-free set accelerator (added at spray nozzle)** | Tamshot 110AF | 6-8% by wt binder |

*Table 1 – SCL mix design used reflecting the requirements for frozen ground*

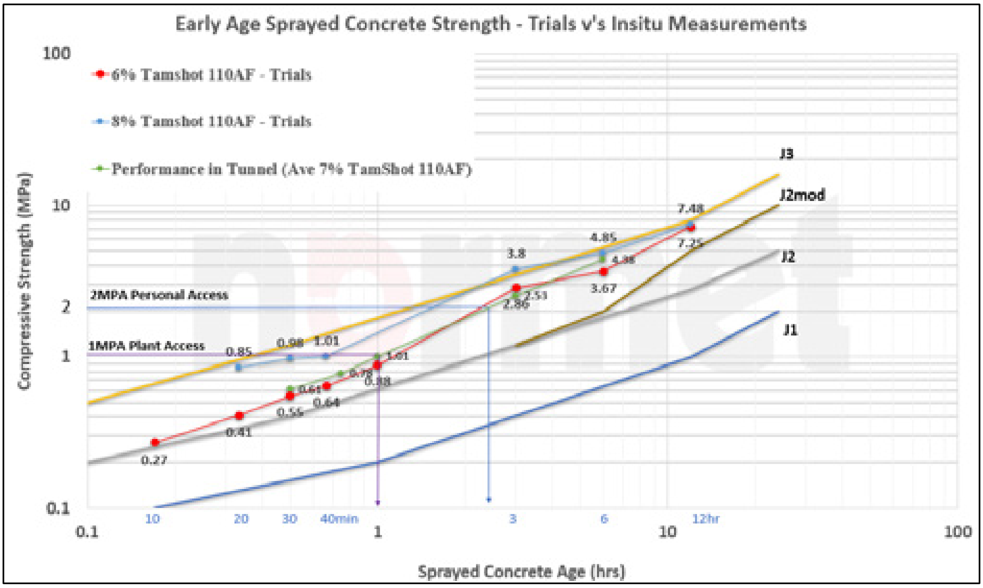
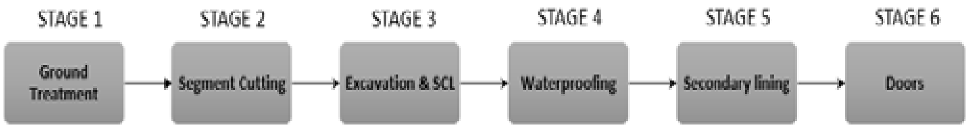



*Figure 3 – EAS results: Insitu testing vs. surface trials*

## 4. CONSTRUCTION

The cross passages (CP) are located at up to 500m intervals along an 8km twin bored 10m. diameter tunnel drive. The CPs are serviced using a sprayed concrete plant setup on the surface, as well as a water treatment plant, muck bin and lay down areas. 4no. 40T Cemex dry mix silos are used to batch the concrete directly into concrete mixer trucks. These then travel to the CP face, and discharge into a concrete pump which supplies a concrete line directly into the sprayed concrete robot. CP construction sequence consists of 6 stages: ground treatment, segment cutting, excavation & sprayed concrete lining, waterproofing, secondary lining concrete and finishes with the installation of fire doors, before being handed over for fit-out and commissioning works. During peak construction production, there are 6no. CPs being constructed at any given time.




The function of the ground freezing design is to form a watertight freeze wall and to provide a structural element of sufficient thickness allowing safe excavation until installation of the SCL. Each AGF CP comprises of freeze pipes, temperature pipes, and water depressurisation pipes installed from the Upline tunnel, along with temperature sensors installed on the Downline tunnel. Furthermore, a layer of insulation was installed on both UL and DL segmental tunnels. As mentioned in the Comparative Analysis subsection, the drilling of the freeze pipes is undertaken from the Upline tunnel with the holes terminating at the extrados of the Downline tunnel.

The alignment of the inclined temperature pipes must consider the constraints from the TBM segments, the soil stratigraphy, and the distances between the freeze pipes. The alignment of the temperature pipes is essential to ensure comprehensive monitoring of freeze wall development around the circumference of the excavation area.. However, the temperature sensors are installed on the Downline CP face, and at locations between the freeze pipe where the warmest temperatures within the freeze wall are expected, which is between the freeze pipes.

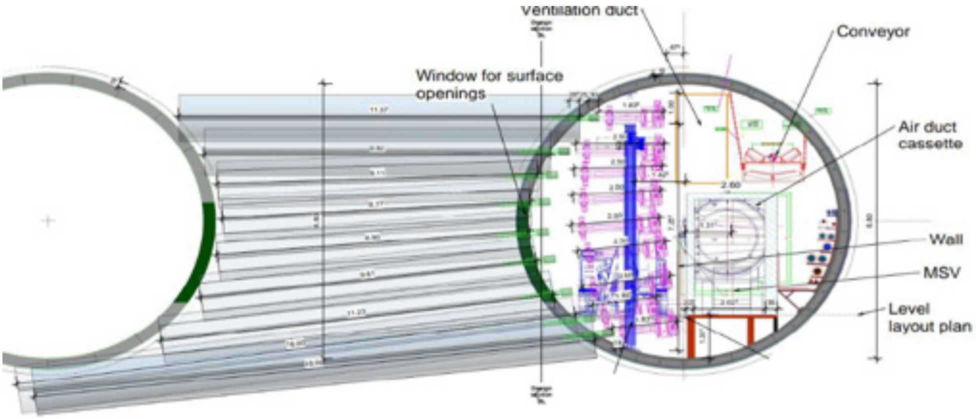



*Figure 4 – Generic freeze pipe layout from Upline tunnel to Downline Tunnel*

The below graph, Figure 5, illustrates the effect of excavation and application of SCL on the freeze body temperature.

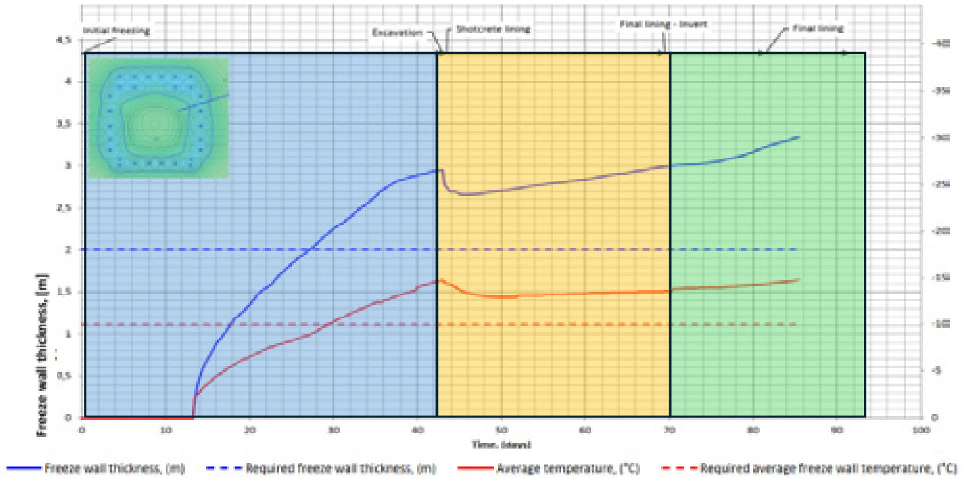



*Figure 5 – Freezy body development in relation to SCL construction; Blue – initial freezing duration; Yellow – excavation and SCL; Green – final lining.*

The CP are constructed through an opening size of 1.9m x 3.1m. Probe drilling was conducted as a means of verifying the ground conditions assumed in the design. Scaffold access is set up to facilitate stitch drilling and track sawing. The segments are systematically removed, enabling a gradual assessment of ground conditions and the application of SCL as necessary. The sequencing of the openings typically follows a conservative approach based on anticipated ground conditions. If sand is anticipated above the tunnel axis, excavation begins with the top heading and invert; otherwise, full face excavation is initiated. This approach minimises exposure of the ground to tunnel temperatures and allows for verification of ground conditions. The initial stage involved shifting the mindset away from viewing the process as tunnelling in soft ground to treating it as a full frozen ice body, akin to a rock tunnel. Figure 6 illustrates different design sequences being applied to different geological conditions in two CPs.

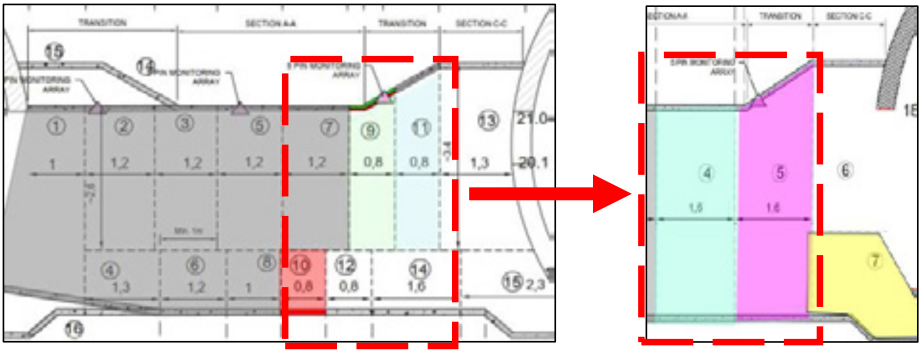


*Figure 6 – SCL construction sequence based on poor (LHS) and stable (RHS) ground conditions*

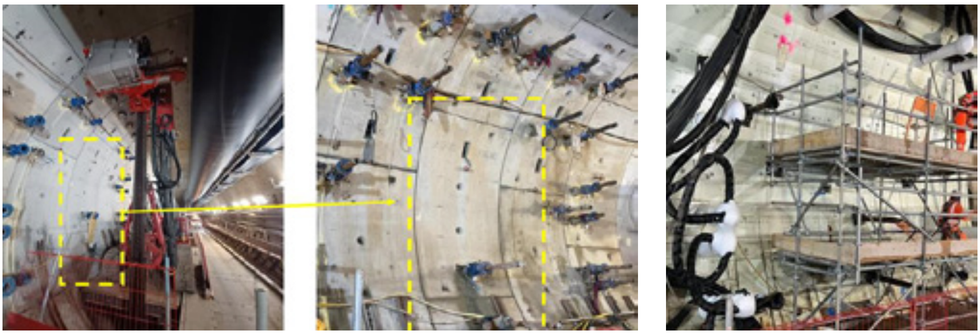


*Figure 7 – Construction stages during CP AGF construction; drill rig setup (LHS), CP opening (Middle), segment coring (RHS)*

The CPs are excavated using 1m - 1.6m advance lengths, with extended lengths in the enlarged collar areas. Excavated material and shotcrete are transported to the CP face via dumper and mixer trucks. During construction, it was observed that the heat generated from the SCL would cause the freeze body to thaw, leading to lack of adhesion during spraying operations and resulting in material wastage. Therefore, several measures were implemented as lessons learned were incorporated while construction gradually progressed through out the CPs.

- In cases where the time between removing insulation from the CP segments and removing the segments exceeded expectations, extra drainage holes were incorporated to eliminate excess water before proceeding with the opening up process.

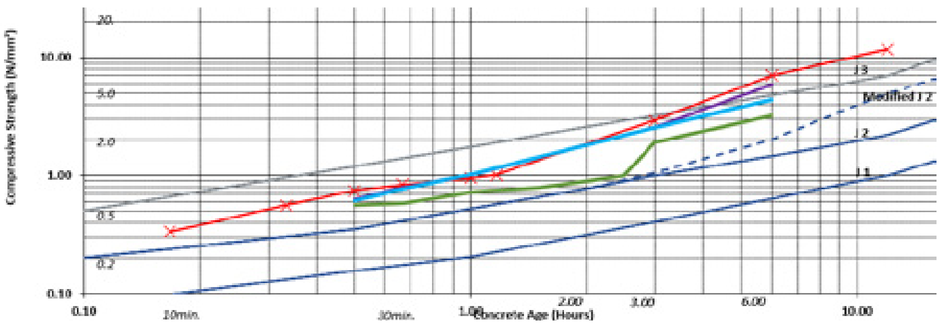
- As per most SCL works in the first few advances the face was sealed, but due to stability and reduce heat generation this requirement was removed.
- Shotcrete application was carried out in multiple passes to ensure proper adhesion.
- In unfavorable ground conditions, shotcrete application was restricted to 50mm in the crown and shoulders initially, allowing for initial adhesion before topping up to full thickness as work advanced.

Shotcrete was batched with hot water temperatures ranging between 30°C and 60°C, targeting 25°C. A conservative approach was adopted to establish the correlation between Early Age Strength (EAS) development and shotcrete on frozen ground. It was determined that achieving a minimum strength of 10MPa on the panel within 24 hours would be necessary for personnel entry, contrasting with the typical requirements when spraying on non-frozen ground.


*Figure 8 – Construction stages during CP SCL construction*

To explore this correlation, the CP team conducted in-situ tests on freshly sprayed invert SCL and on SCL panels simultaneously. Consequently, a specific construction sequence for top heading and invert was required. Although this adjustment impacted production time, the anticipated benefits in the long term justified the decision. Impact was minimised by constructing two top headings (Figure 6 LHS – advance 2 and 3), and then an invert (Figure 6 LHS – advance 4). This means that the 24hour personal entry period would be underway while we were construction the second top heading, and by the time we were ready to construct the invert the 24hours would nearly be fulfilled. In-situ testing was performed using a penetrometer up to 1 hour, followed by stud testing at intervals of 2, 3, 6, 10, and 12 hours. These tests were conducted at three locations within the invert: left, center, and right. Similar tests were conducted on a panel to establish a comparison between panel and in-situ results. The data collected was plotted on a graph to illustrate the relationship between spraying on frozen ground and the achieved strength over time. This determination established that 1MPa was sufficient for machine access, while a strength of 2MPa was required for personnel entry on the panel. The CP team intends to proceed with conducting additional in-situ data testing at the remaining AGF CPs, aiming to establish baseline data for the UK market, which is currently unavailable.

*Figure 9 – EAS data development from insitu testing*



*Interface management*

Simultaneous with CP construction, various activities occur within the Upline and Downline tunnels. These include operations involving the TBM Multiservice Vehicles (MSV), TBM Passenger Service Vehicles (PSV), construction of tunnel invert concrete, subcontractor logistics and additional tasks such as extending tunnel conveyor belts and cables. The CP team implemented lean processes, including the Kanban methodology, to manage and optimise workflows. This approach facilitated the identification of bottlenecks, minimisation of interruptions, recognition of interfaces, and efficiency improvement.

*Figure 10 – MSV travelling at a CP platform (LHS), SCL muck away (RHS)*

## 5. ENGINEERING CONTROL

The approach to the construction of the cross passages using AGF was that of a fully engineered design. Therefore, a rigorous approach to engineering control was essential to the successful implementation of the design. This rigor started with the sizing and competence of the engineering team. This team sat alongside the delivery team fulfilling an independent role in the review of the various data streams emanating from the works. This team managed the interface between the various design elements and the construction team to ensure that the designer's intent was captured in the works. Independent supervision for the sprayed concrete works was also provided in accordance with industry best practice and the Joint Code of Practice; this function was managed by the engineering team. The engineering team also fulfilled a design management function to track the design life cycle of each cross passage.

*Engineering review process*

The primary mechanism for the engineering control of the works is the Required Excavation Support Sheet (RESS) process. In summary, the RESS process is a 2-part control, comprising the distillation of the design into the RESS sheet. This is backed up by a comprehensive daily review of all pertinent data in the shift review group, (SRG) Any required escalations due to trigger value breaches for example are managed by convening a Management Action Team, (MAT), meeting in line with the project processes and industry best practice. The SRG is backed up by a weekly review of the SRG outputs. This weekly review is termed the Contract Technical Committee, (CTC). The CTC is attended by senior members of the Contractors and Designers site teams.

*Performance monitoring / overview*

Monitoring of the works to verify performance and design can be grouped into 3 categories, all of which were reviewed within the SRG: (1). Surface Monitoring; (2). In tunnel Monitoring; (3). Temporary Works Monitoring (Freeze body development).

1. Surface Monitoring: The impact and the development of the freeze body was assessed by monitoring points fixed to assets or set up as a transect across the location of the cross passage. Predicted heave was assessed to be between 10 – 20mm and this was verified in the recorded datal see Figure 11. Note the reversal of the heave upon thawing.

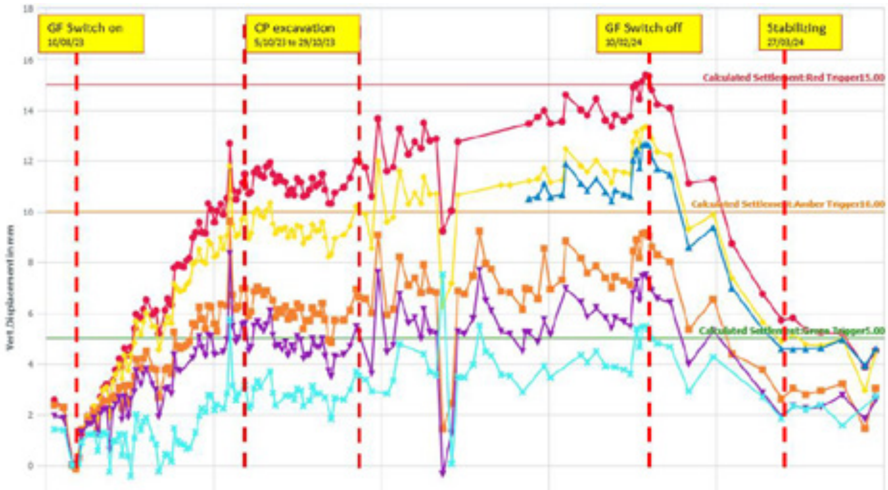



*Figure 11 – AGF impact on CP settlement data*

2. In Tunnel Monitoring: convergence arrays were installed on the tunnel lining adjacent to the cross passage. The deflection of the lining was noted as the freeze body developed but would stabilise when excavation commenced. Similarly, convergence monitoring was also undertaken on the SCL. Negligible convergence was noted, likely due to the stability provided by the surrounding ice body.

3. Temporary Works Monitoring: Temperature pipes with sensors spaced at fixed intervals, were used to visualise the freeze body. As the freeze body developed the temperature at each sensor decreased Warmer temperatures were noted at the sensors adjacent to the tunnel.. The data was used to establish when full closure of the freeze ring was achieved.

## 6. CONCLUSION AND RECOMMENDATIONS

The concurrent activities during CP construction demanded efficient logistics and workflow management. The implementation of lean processes, including Kanban methodology, optimised workflows, identified bottlenecks, and enhanced overall efficiency. Effective communication was vital for coordinating the interfaces, especially considering the exclusion zones at CP locations during spraying operations. Through trials and in-situ use at AGF CPs, the team identified the need to accelerate the concrete with a high-performing set accelerator to create early reactions with the cement and generate heat to initiate setting and continue to support EAS development. The Normet Tamshot 110AF was determined to be the best choice for this purpose. Due to the 20 CPs located at up to 500m intervals along an 8km route, an increased concrete open life of the mix was necessary to account for travel time (1.5HR) from the surface to each CP. Liquid Hydration control admixture (HCA) was added to extend the mix's open life to 5 - 6 hours.

The ongoing construction of the CPs has proved successful to date. There has been a clear willingness among the CP team and supporting functions to embrace the concept of lessons learned and optimise construction sequences through an open and transparent culture. Particularly for AGF CPs, this commitment was demonstrated through various actions, such as transitioning from Hybrid to Full Freezing CPs, interface management, and the production of innovative EAS development data aimed at benefiting the wider UK AGF/SCL market.

A conservative SCL thickness was assumed until the AGF / SCL interaction was fully understood. However, greater efficiency in design was achieved with a reduction of the lining thickness from 250mm to 100mm, along with the corresponding benefits. The flexibility employed in the design approach coupled with the level of engineering control applied through the RESS process enabled the tunnelling teams to react to the conditions encountered and maintain engineering safety.

In particular, the accumulation of AGF & SCL experience among engineers and operatives within the team, which was initially limited, has become a significant asset. This newfound expertise will be invaluable for future UK projects, offering a solid understanding of industry best practices related to AGF works.

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EFFICIENCY IN ACCELERATOR DISPERSAL IN MECHANIZED SHOTCRETE APPLICATIONS

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MAIN THEME: Equipment Technology

SUBTHEMES: Low-carbon solutions & Application, quality control, documentation

**ABSTRACT**

Due to the numerous inputs in shotcrete operations, determining a method for savings and efficiencies that provides numerous benefits throughout the value chain can allow for viability across a wider range of use cases. Mechanized shotcrete is essential for a wide range of construction activities, with the leading use cases being civil tunneling, slope stabilization, & mining.

The operational costs associated with shotcrete can become significant due to the wide range of requirements involved, including batched material components, admixture chemicals, re-entry or set times, rebound, etc. Therefore, finding solutions to provide savings & efficiencies while improving the final quality of the process is crucial to improving operations and making the process more technically & economically viable to an expanded range of use cases with confidence. Additionally, solutions that allow for a reduction in the carbon footprint associated with shotcrete as well as improving operations make the prospect enormously more valuable. This paper discusses how a system designed to reduce the amount of accelerant chemical consumed by using an efficient accelerant dispersal method can provide improved strength development characteristics, while also providing savings & efficiencies throughout the process. Key areas addressed include how atomization of aluminum-sulphate based accelerator impacts initial early-age strength development. While allowing for a reduction in dosage or consumption of accelerator, increased strength development at reduced time intervals that can allow for decreased re-entry times, and effectively optimizing shotcrete mix designs.

By utilizing a passive solution that administers numerous benefits that impact initial and final strengths, as well operational efficiencies, and cost improvements, that adds value throughout the process, is a true leap forward for the shotcrete industry.

## INTRODUCTION

The use of accelerator for shotcrete activities often does not attract the attention it deserves, and the scrutiny required to realize its further-reaching impacts on in-cycle shotcrete. The prospect of reducing accelerator consumption for mechanized shotcrete operations is enticing as it has the potential to provide significant downstream benefits that can impact wider-reaching parts of underground operations than on initial reflection. Further to this point, reducing the dosage of accelerator is a secondary benefit to the real transformation being studied, optimizing the usage of your accelerator. Making the real point of how more can be achieved with less. Material cost for accelerator is significant by itself, with its cost contributing to the greater overall operational costs of shotcrete operations underground. Beyond this, by using the accelerator more effectively, more opportunities arise where refining the complete process and its different inputs become more feasible. Reducing accelerator usage and consumption for the equivalent shot volume of material has the potential to allow for optimized mix designs, ‘greener’ shotcrete, the reduction of re-entry times during strength development, and material usage optimization, in addition to the cost reduction alone of saving on chemical.

## EFFICIENT ACCELERATOR DISPERSAL

The key to optimizing the use of accelerator for shotcrete, and therefore reducing overall consumption, is how the chemical is dispersed at the nozzle. Effective and efficient dispersal for integration into the shotcrete is at the core of what needs to be accomplished to achieve the desired outcomes. Accelerator is needed to speed up the hydration of the Portland cement components of the material (Wang et al. 2021). To understand how the traditional process can be improved and refined it is critical to first grasp how the process works and what occurs at each stage, from pump to nozzle.

**Accelerator Chemical in Shotcrete**

It is critical to kind in mind the cost of accelerator and how quickly its operational cost can accumulate. With prices ranging around $3500 USD per 1000 litre tote, over time the excessive usage of chemical can have a significant impact on the bottom line of those utilizing shotcrete. Providing a method for reducing this cost while maintaining and improving shot material quality is an encouraging proposition for all stakeholders.

Traditionally, liquified accelerator is pumped to the nozzle in its pure form and only becomes dosed into the wet material by use of a peristaltic pump moving the liquid to a 'stream converter' that takes the independent flows of liquid accelerator and high-pressure high-volume air and introduces it into the shotcrete simultaneously. This process has inherent flaws as it relies on the turbulent flow of the shotcrete material itself and air to adequately distribute the chemical throughout the material at the stream converter in its flowing liquid state. With no additional mechanism to blend the three components before exiting the nozzle. To provide a more efficient manner of utilizing the accelerator, an additional process is required to formulate a more effective accelerant-air mixture prior to reaching the stream converter. With a traditional setup, one way to comprehend how the accelerator & high-velocity air, mixes with the shotcrete could be thought of as 'injecting & coating' the shotcrete. To provide a more effective and efficient method, a full atomized gaseous mixture should be obtained.

**Atomization of Chemical**

Atomization of the chemical requires an additional device to the traditional system that provides this effect. Various methods exist for the atomization of fluid into the flow of air that provides this phenomenon. By atomizing the accelerator, the smaller particles in the air mixture provide the desired effect of superior and thorough molecular bonding to the cementitious material. An airblast atomizer provides a method of atomizing accelerator as it is more conducive to varying viscosities of liquids. According to Lefebvre et al. (2017), drop sizes produced by airblast nozzles tend to be less sensitive to variations in liquid viscosity. This can allow for more effective bonding with the cementitious particles in the shotcrete at a molecular level.

By decreasing the size of accelerator particles into the gaseous state versus the liquid state, a superior mixing effect can be obtained. "The airblast concept lends itself to a wide variety of design configurations. However, in all cases the basic objective is the same, namely, to deploy the available air in the most effective manner to achieve the best possible level of atomization" (Lefebvre et al. 2017). To obtain the mixture in such a state, the liquid accelerator chemical must be atomized to allow it to become more comprehensively distributed in the gaseous mixture, using high-velocity air (Ashgriz 2011). In comparison to a liquid being simply introduced into the shotcrete material.

To incorporate these principles and adapt hem for a mechanized shotcrete application that achieves the objective of a more efficiently and effectively disbursed accelerator, research and development have been undertaken to explore and develop systems utilizing different methods for atomization to promote better distribution of chemical droplets that enhance underground mechanized shotcrete applications. Improving processes and creating efficiencies are all part of delivering an improved final product that brings an assortment of downstream benefits.

## EARLY-AGE STRENGTH DEVELOPMENT OF ACCELERATOR ATOMIZED SHOTCRETE

Ensuring that required material properties are obtained is essential when reducing the dosage of accelerator into shotcrete. A reduction in dosed chemical without verification of the desired material properties is a dangerous proposition. When observing the widely accepted 'J-curve' representations for specifications for initial strength development, in accordance with EN 14487-1, it is known and observed that the difference between each represents the relationship between time and material strength. The three J-curves are represented by the data shown in Table 1 below for shotcrete early-age strength development. Where, J3 represents rapid strength development in a brief period and would be the desired outcome for mining applications where set times to 2-4 MPa are preferred to be as short as possible within reason to maximize in-cycle shotcrete processes. Curve J2 is better suited to represent what would be desired in the civil construction industry, where quick set times are less imperative.

*Table 1 - Shotcrete J-curve Representation for Early-age Compressive Strength Development*

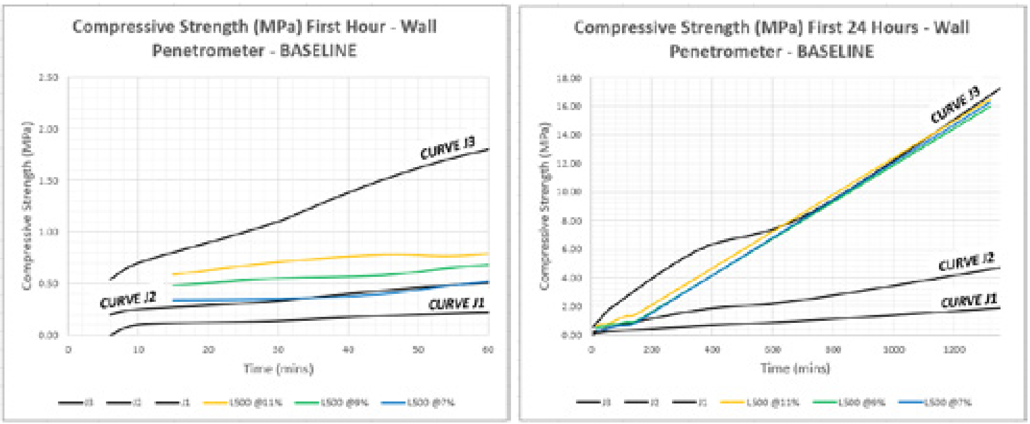
| Time/Age (Minutes) | J1 (MPa) | J2 (MPa) | J3 (MPa) |
|---|---|---|---|
| 6 | 0.00 | 0.20 | 0.54 |
| 10 | 0.10 | 0.25 | 0.70 |
| 30 | 0.14 | 0.33 | 1.10 |
| 60 | 0.22 | 0.51 | 1.80 |
| 360 | 0.60 | 1.75 | 6.00 |
| 720 | 1.00 | 2.50 | 8.50 |
| 1440 | 2.00 | 5.00 | 18.50 |

**Baseline Strength Development for Common Shotcrete Mix**

Before conducting any shotcrete testing for shotcrete sprayed using a system for accelerator atomization, baseline results are needed upon which to draw predictions for enhanced behaviour induced by the more effective hydration of the cementitious components of the shotcrete. All data taken from the field was Sika MS-W1UG shotcrete material for wet-process applications using Sika Sigunit L-500 AFI liquid alkali-free shotcrete accelerator. Results obtained from field data from testing using a conventional mechanized equipment setup suggest that accelerator dosage ranges between 7-11% to achieve 2 MPa in 3.5-4 hours. Observed field data is seen Table 2 and plotted in Figures 1 & 2 below.

*Table 2 – Field Test Results for MS-W1UG using L-500 Accelerator*

| Time/Age (Minutes) | 11% (MPa) | 9% (MPa) | 7% (MPa) |
|---|---|---|---|
| 15 | 0.59 | 0.48 | 0.34 |
| 30 | 0.71 | 0.55 | 0.35 |
| 45 | 0.78 | 0.58 | 0.40 |
| 60 | 0.79 | 0.68 | 0.52 |
| 90 | 1.09 | 0.76 | 0.66 |
| 120 | 1.32 | 0.91 | 0.67 |
| 150 | 1.44 | 0.96 | 0.90 |
| 1320 | 16.5 | 16.0 | 16.3 |




*Figures 1 & 2 – Field Data for Compressive Strength of MS-W1UG product using Sigunit L-500 Accelerator*

As seen in Figure 1 above, early strength development within the first hour is limited. Where development accelerates later within the first 24 hours to reach 14 MPa within 20 hours. Furthermore, it is observed that even an increase in dosage from 7 to 11% accelerator has a limited effect on accelerating the early strength development of shotcrete. This is seen by the closely correlating curves for each of the three test trials. Early strength development for these trials most closely resembled a result that aligns with curve

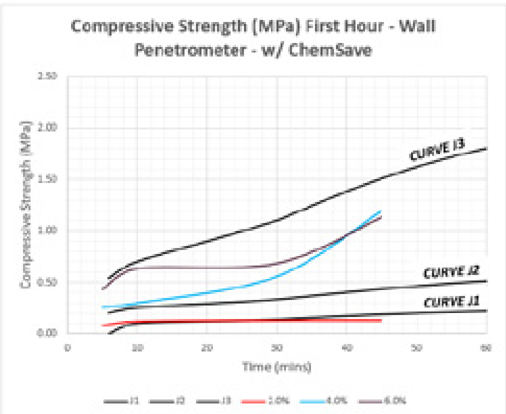
J2, meaning that early-age strength development can be improved to reach the 2 MPa threshold far quicker, potentially with reduced accelerator dosages.

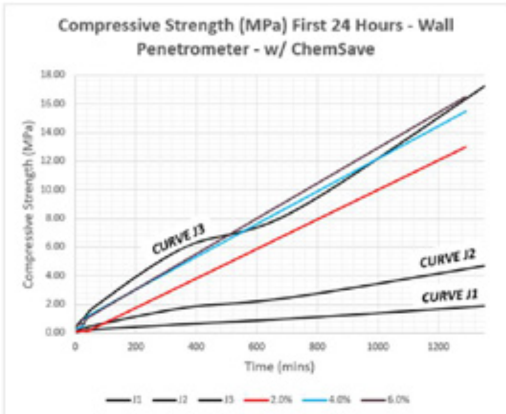
**Testing & Verification of Early-Age Strength Development**

To verify first hour early-age strength development properties of shot material using an accelerator atomization system, early-age strength testing was undertaken on sprayed concrete from a MacLean SS5 shotcrete sprayer equipped with the ChemSave accelerator efficiency and atomization system. Testing was conducted by MacLean Engineering in partnership with Sika Canada using a Mecmesin needle-style penetrometer for first-hour strengths for tests expected under 1.5 MPa. The results observed provided impressive data that outlined how reducing accelerator dosage while maintaining impressive strength generation is feasible. The materials upon which testing was conducted consisted of Sika MS-W1UG shotcrete material for wet-process applications using Sika Sigunit L-500 AFI liquid alkali-free shotcrete accelerator. Testing conducted for early-age strengths was conducted without the use of any synthetic fibres. As seen in Figure 3, it is observed that the first hour early-age strength achieved results trending towards 1.5 MPa within the first 60 minutes for dosages at 4-6%. Where dosage at 2% does not achieve significant strength in the first hour. In comparison to the curves observed in Figure 3, material shot at 4 and 6% experience significantly more strength development than the material shot at much higher dosage levels without the use of accelerator atomization. Where the data for tests at 4 and 6% in Figure 3 below are trending towards 1.5-2 MPa within 1 hour, which is a compelling increase in strength development considering the elimination of variables by using the same mix design, materials. and accelerator are controlled and considered to be the same quality and from the same manufacturing process. Proving that with no variability between the materials, besides the utilization of an accelerator atomization system at decreased dosage levels, can provide impressive, reduced re-entry time for in-cycle operations.

*Table 3 – ChemSave Trial Test Results for MS-W1UG using L-500 Accelerator*

| Time/Age (Minutes) | 6% (MPa) | 4% (MPa) | 2% (MPa) |
|---|---|---|---|
| 5 | 0.43 | 0.26 | 0.08 |
| 10 | 0.63 | 0.30 | 0.12 |
| 30 | 0.68 | 0.56 | 0.13 |
| 45 | 1.13 | 1.20 | 0.13 |
| 1290 | 16.5 | 15.5 | 13 |




*Figures 3 & 4 – Compressive Strength of Sika MS-W1UG product using Sigunit L-500 Accelerator at varying dosages using ChemSave system*

Tests conducted beyond the initial first hour of strength development, and the 1.5 MPa threshold, were conducted with the use of a HILTI DX 460 nail gun with a 72mm nail. This test provided a result for the 24-hour strength development. Where we can observe the trends in comparison to the J2 and J3 curves that identify the rate and magnitude of strength development in this region. When considering 24-hour compressive strength development, it is observed in Figure 4 that all data trends towards following the J3 curve. Highlighting that even at incredibly low dosages, appropriate strength development for a 24-hour period is observed utilizing an accelerator atomization system. Furthermore, these results show that an accelerator system utilizing atomization as a method for more thoroughly mixing the chemical into the shot material can yield results in line with the representation for strength development by curves J3 and J2. Overall, the testing conducted provided a result that points towards an initial conclusion that an accelerator atomization system can provide results for early first-hour strength development that are consistently within the bounds of all three curves, and first 24-hour strength all trends towards the targets set by J3, while drastically reducing dosage levels of accelerator in comparison to baseline data.

## STRENGTH DEVELOPMENT OF ACCELERATOR ATOMIZED FIBRE-REINFORCED SHOTCRETE FOR PERMANENT LININGS

Shotcrete designed for permanent linings with a composition of synthetic fibres, commonly known as 'fibrecrete,' was tested using the ChemSave accelerator atomization system to determine strength development properties over a period of 30 days. Testing intervals ranged from as early as 8 hours up to 28-day strengths to provide data on strength development for mix designs using MS-W1UG product with 5kg and 7kg per cubic metre of SikaForce Fibre-48, respectively. These tests were conducted to provide insight into the impact on strength development of utilizing an accelerator atomization system for shotcrete designed for permanent linings. All tests conducted were utilizing the ChemSave system to determine the feasibility and performance of the system when utilized for materials designed for permanent linings.

### Early-age Strength Development Utilizing End-beam Test

First 24-hour strengths were determined with the use of the end-beam test common to North America and Australia. Tests were conducted at 8- and 24-hour intervals to test for compressive strength development at dosages of 3 and 5% accelerator content per kilogram of cementitious material per cubic metre. Variations also included tests with differing fibre content at 5 and 7kg of fibres respectively, both tested at the varying dosages of 3 and 5%. Results from this test are crucial to determining the strength development versus time relationship for these fibrous mix designs for permanent sprayed linings to assist with determining re-entry times for in-cycle shotcrete. Mix design, accelerator content, and re-entry time are all key factors in optimizing operational performance and cost for sprayed concrete.

It was determined that for shotcrete sprayed at a dosage of 5% accelerator, results between 3.5-4.5 MPa were achievable within 8 hours. With later age strengths ranging between 17-25 MPa, depending on fibre content. As seen in Figure 4, for mixes with higher fibre content, lower early strength is achieved while later strengths are significantly higher. These results are in line with expectations for the behaviour of these materials and provide data points to compare when plotted against the same sprayed material at varying accelerator dosages.

*Table 4 – ChemSave End Beam & Core Test Results for MS-W1UG with Fibre48 using L-500 Accelerator*

| Panel/Test | End-Beams 8h | End-Beams 24h | 3D Cores | 7D Cores | 28D Cores |
|---|---|---|---|---|---|
| 7kg/m3 at 3% | 3.3 | 23.9 | 31.1 | 43.5 | 59.5 |
| 7kg/m3 at 5% | 3.6 | 24.9 | 24.8 | 33.3 | 58.3 |
| 5kg/m3 at 3% | 6.2 | 12.8 | N/A | 27.2 | 41.9 |
| 5kg/m3 at 5% | 4.4 | 16.5 | N/A | 34.6 | 50.2 |

**All test results UoM measured in MPa*

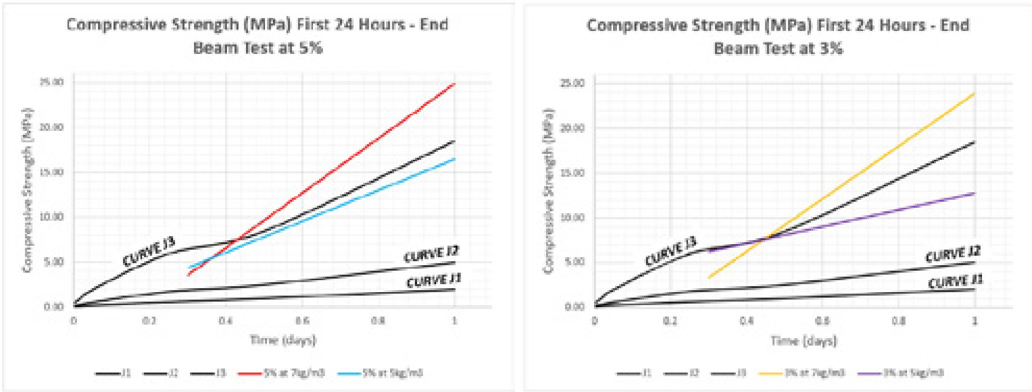



*Figure 4 & 5 – End-beam Test Result for First 24-Hour Compressive Strength for Fibrecrete Material at 5% Accelerator*

For shotcrete sprayed at a dosage of 3% accelerator, results between 3-6 MPa were achievable within 8 hours. While later age strengths ranged between 13-24 MPa, depending on fibre content. As seen in Figure 5, the same relationship between mixes at varying concentrations of fibrous content at varying time intervals applies. The results from the reduced accelerator dosage test provided encouraging information that provides evidence that reducing accelerator dosages with no changes to mix design can achieve comparable strength development levels when using a method of accelerator atomization. This is since the more thorough mixing of the accelerator into the shotcrete particles is occurring, and effective hydration of the cementitious materials is occurring more efficiently, providing excellent numbers for strength development. The results above provide significant insight into more use cases of an accelerator atomization system and how it may affect the performance of the final sprayed material, and how it can be used to achieve required strength targets while providing the ability to reduce operating costs on accelerator. For permanent linings, the first 24-hour strength development is crucial, and the end-beam test provides a thorough, accurate result upon which to draw further conclusions and provide insight for direction on further research.

**Later Age Strength Development Core Testing**

To be able to provide a full understanding of the strength development of a specimen of shotcrete, further testing is required in addition to the tests discussed above. Determining observed later-age strength development characteristics was conducted using core samples tested at intervals of 3, 7, and 28 days. For all core samples considered, panels were shot on the wall of the substrate and were continuous onwards from the end-beam specimens to provide as similar a shot product as possible. The primary focus of the core sample tests was to provide data on the utility of an accelerator atomization system for achieving the required final material strengths within 28 days. Figure 5 highlights the results of all four examined variations of core samples and plots them against each other over a 28-day period to identify the durations for achieving prominent levels of compressive strength.

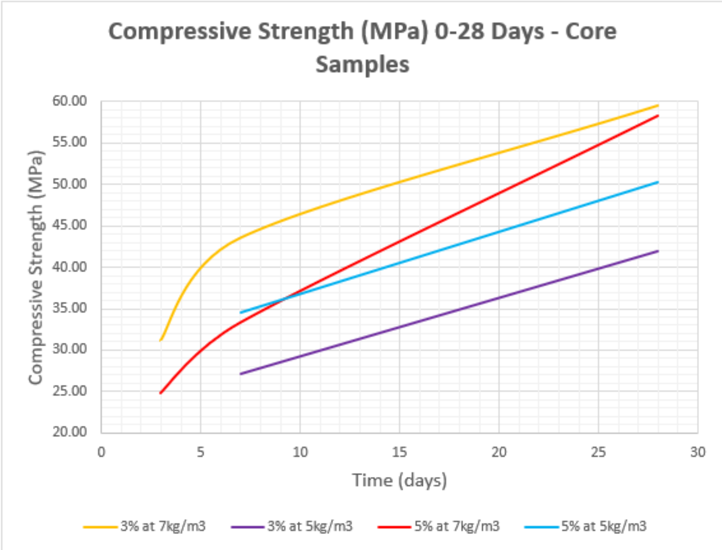



*Figure 6 – Core Sample Test Results for 28-Day Compressive Strength Development of a Fibrecrete Material*

As seen above, the results achieved were in line with initial expectations, except for the result for the material shot at 3% with 7kg/m$^3$ of synthetic fibres. All materials shot achieved a maximum compressive strength of 40 MPa ranging up to 60 MPa within 28 days. These results would satisfy many of the requirements for most projects utilizing a similar mix design. However, for the results that achieved the highest maximum strength development, we observe that some of these specimens are of a shotcrete that is over-engineered for its application, where a redesign may be undertaken to modify the composition and weights of the subcomponents of the mix design to optimize for performance, cost, and carbon intensity. Ultimately leading to a 'greener' material, but also one designed specifically for required performance and does not overcompensate to ensure needed strength development targets are achieved.

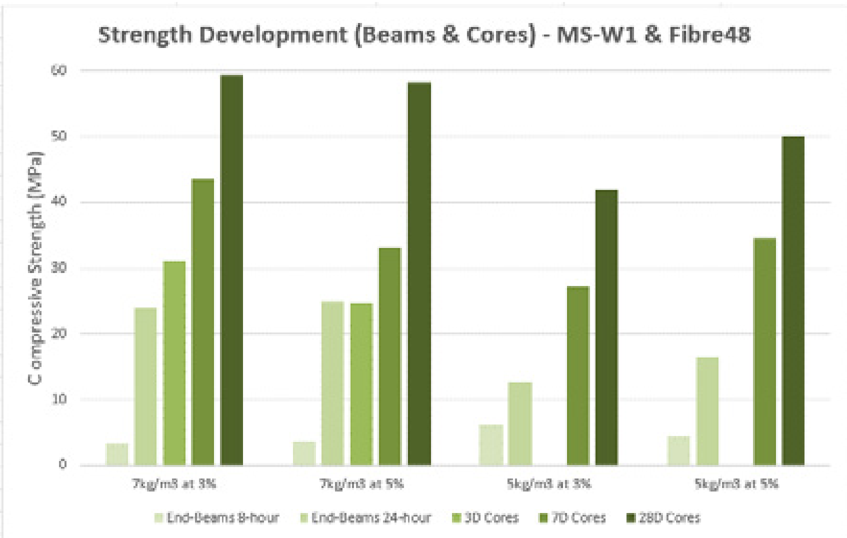



*Figure 7 – Strength Development Results for End-beam & Core Test Samples*

Figure 7 above shows the results of the end-beam and core test results overlayed to illustrate the strength development of each sampling variation. It is observed that all test samples achieved excellent results, even the samples that were sprayed at an accelerator dosage of 3%. Which had a maximum compressive strength of almost 60 MPa when batched with 7kg/m$^3$ of fibres, and over 40 MPa with 5kg/m$^3$ of fibres. A result that provides evidence that the accelerator atomization system was able to hydrate the mix more effectively and therefore achieve a result higher than what would typically be expected for the final 28-day strength of the same design. Where peak compressive strength normally is tested for around 50 MPa, when shot using a conventional mechanized shotcrete piece of equipment.

## OPTIMIZING MIX DESIGN

Mix design optimization is the process of looking into the parameters of the mix in question, ensuring that proper requirements are followed to ensure the fully engineered quality of the product. With the optimization of cementitious properties through accelerator atomization, a "leaner" product can be achieved without sacrificing end strength prerequisites, supported by the trial test results conducted on a standard shotcrete mix. This can also support the drive to achieve more stringent ESG initiatives and aid with carbon reduction strategies or potential qualification for governmental carbon credit schemes. Efficiently dosing and dispersing accelerant into wet shotcrete at the nozzle, is one of the biggest steps that can easily be taken to fully optimize the shotcrete mix downstream. Efficient use of accelerator can aid in ensuring that final strengths are not degraded by "cooking" the shotcrete, having a high heat of hydration over a longer period.

### Greener Shotcrete

Optimization of mixes increases the amount of savings over the long term, and even the short term, reducing the need for excessive quantities of cement and additional admixtures. Reduction of cement ensures that mixes can be made to engineer design and remove excessive safety nets for final strength development, also ensure that the final product does not become too brittle over time, especially if fibre reinforcement is required. Greener shotcrete, a shotcrete that can follow more stringent carbon reduction guidelines by reducing the amount of Portland cement and effectively reducing the need for additional on-site storage, additional logistical and transportation efforts. Efficient reductions in accelerator utilization have a clear correlation to cement reduction and improved bonding, which in turn has slight effects on rebound reduction. The efforts to optimize mix design for strength development targets when using methods for accelerator atomization, along with the drive to achieve a 'greener' shotcrete, can be combined towards a holistic effort to design and build a shotcrete specialized for specific applications.

As described by Aldrian et al. (2022), cement has a significant impact on the total $CO_2$ intensity of shotcrete, as high as 91%, due to the excessive amounts of chemically bound $CO_2$ released during the burning process used to produce clinker. Therefore, ways that reduce the usage of clinker-heavy cement in the final mix design is the key path towards cleaner concrete. The methods described by Aldrian et al. (2022) to use clinker-efficient composite cement is one way to modify how clinker-based cements can be modified to reduce carbon intensity. However, by also introducing a method of more effectively using the cement component of your existing mix designs through accelerator atomization, opens the door to further optimizations and allows for a rethink of how carbon-reduced shotcrete can be approached.

## REDUCING RE-ENTRY TIMES FOR IN-CYCLE SHOTCRETE

Among the most valuable commodities for underground operations is time. For shotcrete operations, the applied product must have sufficient time after being applied for strength development. This is even more critical when working with unsupported ground, as is common with many underground operations utilizing shotcrete. Allowing the material to set and undergo strength development demands that sufficient time pass before re-entering the space supported by the young material. This delay prevents further work from taking place while the material is given time to set. Finding ways to reduce this waiting period is imperative to increasing the overall productivity of in-cycle shotcrete.

Considering the impacts of more efficient accelerator dispersal into the shotcrete, and therefore enhanced material properties, allows the mix design to be formulated in such a way that improvements can be found to provide better early strength development, as mentioned earlier. This is done concurrently with ensuring the final compressive strength is acceptable. Providing more rapid early-age strength development without sacrificing older-age compressive strength allows for a reduction in the re-entry time (time required to allow for sufficient strength development) required. Meaning a high rate of productivity can be achieved with the same resources.

## CONCLUSIONS

The possibility of reducing the consumption of accelerators for mechanized shotcrete holds the potential to create extraordinary amounts of value. The ability to reduce operating costs due to lower consumption of chemical alone is significant, but when downstream effects are taken into consideration the advantages are compelling. By providing a method of more consistently and effectively adding accelerator to the shotcrete, the ability to make more optimized mix designs and 'greener' shotcrete is gained in a greater capacity than it would without. Downstream of that effect is the ability to create mix designs that can allow for faster strength development that still satisfies all other required material parameters.

The core component to achieving these objectives is to alter the way the accelerator is introduced into the material and to encourage thorough mixing. Atomization of the chemical provides a way of combining the processes of mixing shotcrete with an accelerator and physically shooting the material with high-velocity air. This allows for the implementation of the 'Do More with Less" philosophy. With the global shift towards a greener economy and finding new efficiencies in the industry, underground operators utilizing shotcrete have the potential to further corporate goals while improving operational processes.

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# LOW-CARBON, SHRINKAGE-REDUCED, SPRAYED CONCRETE; A PILOT STUDY

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**ABSTRACT**

Low-carbon concrete for pouring or pumping has become an industrial standard, and willingness to pay the extra cost remains the limit to further reduction of the carbon footprint from the concrete industry. A corresponding development seems not to have taken place for sprayed concrete.

Sprayed concrete meets additional technical requirements, compared to concrete for pouring or pumping. Sprayability comprises the ability to be pumped at high speed through a nozzle while being added accelerating admixture dispersed in a stream of compressed air, and then crashed towards a wall without losing homogeneity. Following the flash-setting caused by the accelerator, the concrete must obtain a rapid hardening development.

Shrinkage causes cracking and consequently reduces the durability of concrete in harsh environments. Sprayed concrete is often used to secure rock walls, where it might be exposed to mineral-rich water which might be extra harmful to the durability of both the concrete and the reinforcement.

This paper describes a full-scale pilot study with an attempt to reduce the $CO_2$ footprint of sprayed concrete, through partial substitution of the Portland clinker (OPC). To counteract shrinkage cracking, a shrinkage-reducing agent was added. The reference concrete, which was the base for the reduction of OPC, was used for rock wall securement in an actual large road-building project nearby.

All concrete batches turned out to be highly sprayable and clinging stable to the rock wall. The content of Portland Clinker was reduced from 445kg/m$^3$ to 305, heavily reducing the $CO_2$ emission. A strength development sufficient for rock securing was not achieved. However, the sprayed low-carbon concrete seems applicable for fireproofing. Even after 3 years of measurements, the shrinkage-reduced concrete shows a 38% reduction of shrinkage compared to the reference concrete.

**Keywords**: Low carbon, Shrinkage-reducing admixture (SRA), Materials development

## INTRODUCTION

During the later years, there has been an increased focus on reducing $CO_2$ emissions from the construction of large infrastructure projects like highways and hydropower plants. Sprayed concrete for rock support is an important part of these, e.g. in tunnel construction. Sprayed concrete is also used for fireproofing tunnels. However, carbon reduction of sprayed concrete does not seem to have had as strong a focus as traditional cast concrete.

There are 3 main approaches frequently used by ready-mix producers, to reduce $CO_2$ emissions from concrete mix designs. First, by reducing the amount of cement in the mix design. This is desirable both from an economical view as cement is an expensive constituent, and from an environmental point of view as cement is the main contributor to $CO_2$ emissions. The water demand of the mix is a key indicator for the concrete producer. Reduction of the water demand is achieved through optimization of the particle size distribution (PSD) of the aggregate, and the utilization of superplasticisers (SP). A reduction of water allows a reduction of cement, to keep the water/binder (w/b) ratio at a fixed level.

The second approach is to partly substitute Portland clinker (PC) with supplementary cementitious materials (SCM). SCMs are often pozzolanic, thus not reacting with water but with Portlandite ($Ca(OH)_2$) which is a by-product of the PC hydration. The pozzolanic reaction is consequently secondary to the PC hydration, thus slowing down the development of both strength and heat in hardening concrete. Reduction of PC lowers the heat of hydration and lowers $CO_2$ emission. As most used SCMs are by-products from other industrial processes, this substitution might also constitute one step in the transition of the concrete industry towards a circular economy.

Thirdly, using coarsely ground cement types rather than fine, reduce the water demand in the mix design. Once again, a reduction in water demand contributes to lowering $CO_2$ emissions from the concrete through the consecutive reduction in cement content. Due to the less surface area of the coarse cement being available for immediate hydration, this also leads to lower heat development.

All three measures mentioned above to reduce $CO_2$ emissions also lead to slower strength development. Setting and hardening development include several challenges to sprayed concrete. First, a layer of fresh concrete needs a certain cohesion to cling to the substrate immediately after spraying. Rebounce and fall-out affect productivity and constitute HSE harm. The setting phase of cement hydration is normally used to secure the necessary cohesion. To accelerate the setting of sprayed concrete, a "shotcrete accelerator" is added to the fresh concrete in a stream of compressed air, as it leaves the spraying equipment through the nozzle. This addition is an integrated part of the spraying operation. The shotcrete accelerator is a "set accelerator" which activates the setting process within seconds. This part of the spraying operation is one explanation of why experimental works on shotcrete lack relevance for industrial use, if not including the full-scale spraying procedure.

When spraying concrete for rock support, early strength development is essential to ensure that the concrete remains where it was placed and secure loose parts of the rock substrate towards fall-out. If the necessary compressive strength development is not achieved, the risk of collapse increases, yielding heavy consequences for HSE. Standard requirement for hardening development is that the compressive strength should be 1 $N/mm^2$/hour for the first six hours. Requirements for shotcrete used for fireproofing are not equally strict, as this procedure is executed secondary to securing the rock wall.

To reduce the carbon footprint of sprayed concrete, it is necessary to include low-carbon solutions. Low-carbon concrete includes substituting PC with SCMs. Indications are that the addition of the set accelerator during spraying is not sufficiently counteracting the retardation caused by the PC substitution, regarding strength development. Introducing a "hardening accelerator" (HA) during the production of the concrete might contribute to increasing the strength development, thus counteracting the retarded hardening development caused by the cement substitution. A literature search with meagre results indicates that knowledge of how HAs affect the shotcrete process in industrial settings seems to be scarce. This is the basis for our Research question number 1, which is stated in the section below.

A second issue with sprayed concrete that seems to be poorly investigated on an industrial scale, is that shrinkage can cause cracking in hardening concrete. Reducing shrinkage might be a measure to increase durability. Requirements or guidelines for restricting crack development in sprayed concrete seem not to have been adopted, at least within the CEN system. The Norwegian Concrete Association have given guidelines for shrinkage reduction in slabs made of ordinary concrete, through publication no 15 [1]. Three classes of floor qualities are described, depending on the maximum desired size of shrinkage and crack width. Necessary measures to achieve each class are stated. Even if the application area is different, this might serve as guidelines for evaluating and advising on how to reduce cracks in sprayed concrete.

One measure for successfully achieving the best class according to [1], is the utilization of two layers of plastic sheets underneath the concrete to separate the slab from the ground below. This is to allow sliding movements, thus avoiding the creation of cracks due to retaining during shrinkage. Sprayed concrete on rugged rock surfaces needs to cling to the substrate, thus not allowing the concrete to move relative to the base. A reasonable assumption to make is that application of all other measures required to obtain the best flooring class according to [1] is necessary but still not sufficient to obtain corresponding results in sprayed concrete.

Another measure might be restricting the water content, exceeding what is required for deciding the durability class in ordinary concrete types. High water content is one of several factors that contribute to increased shrinkage. Including a certain level of water is necessary to make the concrete sprayable. The addition of shotcrete accelerator during the spraying process increases the water content further due to the water content of the accelerator. When designing the mix composition, the concrete producer accounts for this additional added water, when delivering ready-mix that will consent to the quality requirements after spraying. However, the spraying process leaves a different product than traditional placement methods for concrete. Additionally, when constructing e.g. a large slab, the contractor might implement several actions to restrict or handle cracks, that are not possible to implement when spraying on rock walls and ceilings.

A last measure mentioned here that might be applicable, is the use of admixtures. Chemical admixtures for reducing shrinkage have been commercially available for years, generically referred to as SRA (Shrinkage-Reducing Agent). Knowledge of how these additives influence the properties of sprayed concrete seems however to be scarce. This is the basis for our Research question no 2 stated in the section below.

## RESEARCH QUESTIONS AND INDUSTRIAL IMPACTS

Based on the above-enlightened shortcomings, the following issues have been addressed:

**Research question 1:** How does the application of hardening accelerating admixture (HA) influence the strength development of sprayed concrete, when used to compensate for the retarding caused by measures commonly applied to reduce $CO_2$-emission in standard concrete?

**Research question 2:** How does the application of a shrinkage-reducing agent (SRA) affect the shrinkage of sprayed concrete?

Finding good answers to these questions might contribute to reducing the $CO_2$ emissions of sprayed concrete both directly through the mix design of fresh concrete and indirectly through increasing the lifespan of structures.

## METHODS AND MATERIALS

The research approach was an experimental full-scale pilot study. The reference mix design was the same as applied for sprayed concrete in the 500 million Euro highway project E39 Kristiansand-Mandal, and the same industrial equipment for mixing, transportation and spraying was utilised for all experiments. Both the ready-mix concrete producer and the shotcrete operating company are experienced in these types of projects.

3 different mix designs were tested, details given in Table 1. To produce test specimens, each mix was produced in batches of 2 $m^3$. The plant has a 3.5 $m^3$ twin shaft mixer, 4 powder silos for binders, 6 aggregate silos and 8 admixture tanks. The shotcrete accelerator was not added at the producer's facility but rather during spraying, according to the standard industrial procedure. The w/b-ratio was kept at 0.415 for all mixes, allowing for an addition of up to 38kg/$m^3$ of shotcrete accelerator, still being within durability class M45 according to [2].

The reference mix was produced twice. It was produced, transported, controlled for fresh properties, and then sprayed. The start-up of any spraying procedure, including calibration of the addition of set accelerator, is challenging and depends on the skills of experienced operators. After calibration, the accelerator consumption was measured at 30 – 35 kg/$m^3$, which is normal for sprayed concrete for rock support – at least in Norway. After this initial testing, utilisation of the reference mix were repeated to ensure repeatability.

*Table 1 - Concrete mix designs*

| | **Reference** | **Low Carbon (LC)** | **LC with SRA and HA (LC-SRA-HA)** |
|---|---|---|---|
| Cement (CEM II/A-V 42.5 R) | 445 | 305 | 305 |
| Silica Fume (activity factor =2) | 19 | 18 | 18 |
| Fly Ash (activity factor = 1) | 0 | 125 | 125 |
| Water | 200 | 193 | 193 |
| 0/8 | 1699 | 1699 | 1699 |
| Superplasticiser | 5 | 4 | 4 |
| Air Entraining Admixture | 1 | 1 | 1 |
| Shrinkage Reducing Admixture | 0 | 0 | 4 |
| Hardening Accelerator | 0 | 0 | 10 |

**Compressive strength**

Immediately after spraying, the concrete temperature was measured as illustrated in Figure 1 left side. The early compressive strength was tested in three different procedures. The early strength was tested for the first 3 hours in special procedures developed for sprayed concrete. After 1 hour, the compressive strength is measured directly on sprayed concrete utilizing a needle penetrometer, as shown in Figure 1 midst. This method is valid for compressive strength measurements up to 1.2 N/$mm^2$. For compressive strength $\geq$ 2 N/$mm^2$, a Hilti gun is used to shoot nails into the hardening sprayed concrete and then pulling them out. The equipment is shown in Figure 1 right part. The penetration depth and force required to pull nails out is then used to calculate the compressive strength.

Later compressive strength was measured on 100 mm x 100 mm x 100 mm test cubes after 1, 7 and 28 days, according to [2]. The test cubes were not sprayed but cast, following standard procedures for quality control.

*Figure 1 - Left: Temperature measurements on sprayed concrete. Middle: Example of needle penetrometer. Right: Hilti gun for the Hilti gun method.*

**Shrinkage measurement**

All specimens for testing shrinkage were sprayed to include the extra water added through the shotcrete accelerator, as indicated in Figure 2. The mid picture shows the positioning of the form before filling. The rapid set and hardening of sprayed concrete make it relevant to test at a very early stage. As no standards are made for this purpose, procedures for testing standard concrete were adjusted for application on sprayed concrete. The shrinkage was measured already 6 hours after spraying. This differs from the procedures where the first testing is performed at 24 hours, described in the NB 15 [1] and the Swedish standard SS-137512 “Concrete testing - Hardened concrete - Shrinkage” [3]. This Swedish standard is often used in Norway. However, neither of these publications is made for testing sprayed concrete.

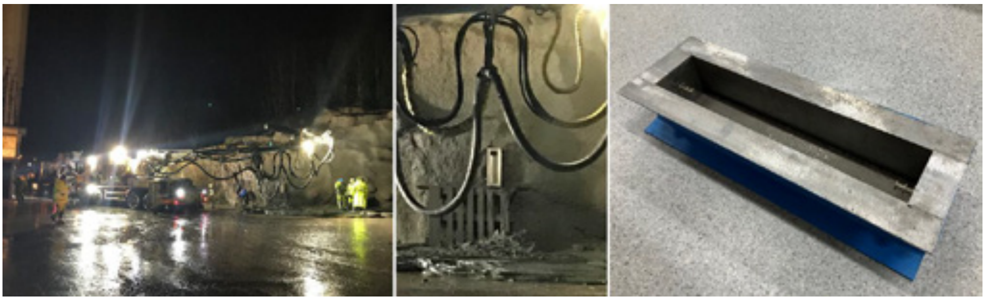

*Figure 2 – Left: Full-scale test of sprayed concrete Middle: spraying of specimens for shrinkage measurements. Right: Mould for shrinkage measurements.*

The specimens for testing shrinkage have dimensions 100 x 100 x 500 mm and include a steel peg at each end for measurement reference. These pegs are visual inside each end of the empty form shown to the far right in Figure 2. The shrinkage measurements are performed as follows: A reference rod in steel is measured, the specimen is measured, followed by the reference rod measurement repeated to ensure that the measurement is correct.

## RESULTS AND DISCUSSION

### Compressive strength

The initial temperature of the concrete before spraying was 20 degrees and the slump was 220 mm. The concrete temperature was measured immediately after application to the rock substrate. The essential strength development happens during the first 6 hours after spraying. A normal requirement when spraying for rock support is to achieve a strength development of 1 N/mm$^2$ per hour, starting at 1 N/mm$^2$ after 1 hour and ending at 6 N/mm$^2$ after 6 hours.

For the reference mix, a 6°C temperature increase was measured immediately after spraying. Such an immediate increase in temperature is considered a confirmation that the shotcrete accelerator is successfully included. Compressive strength of 1 N/mm$^2$ was measured after 1 hour and 5 minutes.

For the low-carbon concrete without the addition of a hardening accelerator (LC, ref Table 1), an immediate temperature increase of 3°C and a compressive strength of 0.5 N/mm$^2$ after approximately 1 hour was measured. However, there was no further development in compressive strength for the next three hours.

For the low-carbon concrete with the shrinkage-reducing agent and hardening accelerator (LC-SRA-HA, ref Table 1), a 3°C temperature increase was measured immediately. However, no compressive strength was measurable for the first three hours. All results on compressive strength are shown in Table 2.

The reference mixes achieved the best strength development at an early stage. The compressive strength after 1 day also shows the reference mix achieving the highest compressive strength. At 7 and 28 days the performance of LC and LC-SRA-HA are more equal to the reference mix.

Table 2 - Mass of specimens and compressive strength development for all mixes.

| | 1 hour | 1 day | | 7 days | | 28 days | |
|---|---|---|---|---|---|---|---|
| | Sprayed | Cast | | Cast | | Cast | |
| Mix | N/mm$^2$ | kg | N/mm$^2$ | kg | N/mm$^2$ | kg | N/mm$^2$ |
| Reference Test 1 | 1 | 2,25 | 24,6 | 2,29 | 39,7 | 2,27 | 55,6 |
| Reference Test 2 | 1 | 2,32 | 31,7 | 2,34 | 51,9 | 2,32 | 64,9 |
| LC | 0.5 | 2,28 | 17,5 | 2,31 | 38 | 2,29 | 57 |
| LC-SRA-HA | 0 | 2,32 | 19,3 | 2,31 | 39 | 2,34 | 57 |

A first reflection on the results is the difference in strength between reference mixes 1 and 2, which are supposed to be approximately equal. Air content is however influencing the compressive strength. Variations in air content might be due to several factors, among others the mixing procedure, the influence of fly ash in the cement (CEM II/A-V) and the rest time and speed of the drum on the concrete truck. Small variations amongst factors like these might have influenced the test specimens. Variations in weight might be used as an indicator. A normal assumption is that 1% of increased air content leads to a 5% reduction in compressive strength. Applying this rule of thumb on the test results for reference mixes 1 and 2, seems to explain the variations quite accurately for all cube tests.

Results from the ready-mix producer's quality control (QA) when producing concrete for rock support can be misleading. The QA is executed on cast cubes. As the produced water/binder ratio is lower than normal for a C35/45 to allow for the additional water content in the shotcrete accelerator, compressive strength from the producer's quality control is normally several N/mm$^2$ higher than actual values achieved at the construction site.

The lack of accelerated compressive strength development in the sprayed samples which is shown in the far left column in Table 2 indicates that the mix design for low-carbon concrete was not sufficient for rock support applications, despite the addition of a HA. Compressive strength results after 28 days were quite close to those of the reference mix, clearly exceeding the requirements.

These experiments do not contribute with satisfactory results to argue for utilizing low-carbon sprayed concrete for rock support. However, the requirements for strength development in sprayed concrete for fireproofing are lower. Thus, the results might support industrial applications for reducing $CO_2$ emissions from sprayed low-carbon concrete used for fireproofing.

**Shrinkage**

The maximum nominal size of aggregates in ordinary concrete is 22 mm (D22). Sprayed concrete has a maximum nominal size of 8 mm (D8). This reduction in $D_{max}$ triggers an increased water demand, as the accumulated surface of the aggregate particles is increased. The difference in water demand at the same consistency and w/b ratio, can be up to 40 litres of water per m$^3$, depending on the shape of the particles which is characterised by measurable parameters like flakiness and particle size distribution (PSD). This describes the water demand during mixing before the addition of the shotcrete accelerator. The total water content of a sprayed concrete of the applied quality is approximately 210 litres/m$^3$, depending on the amount and type of accelerator. High water content increases the shrinkage of concrete. Traditionally, shrinkage of sprayed concrete is not being focused on in Norway. However, the durability of concrete is dependent on homogeneous concrete without cracks.

Being able to spray concrete in the relatively small and narrow formwork was achieved through a well designed mix composition of concrete and the skills of the operator. In Figure 3 far right, the surface of the end of a test specimen can be seen. There are no visual signs of blockage or other failures. The two photos from the right in Figure 3 show details explaining the steel peg embedded at the end of the specimen in the far right photo.

*Figure 3 – Left: mould. Middle; closeup of steel peg. Right: close-up of sprayed specimen.*

The first measurement of shrinkage was performed 6 hours after the forms were sprayed with concrete. There is a major gap in measurements between 23 days and 1123 days, due to the lockdown during the Corona period. However, the quality of all results seems consistent and good across this period. It is well known that most of the shrinkage happens during the early maturing process, which is supported by these results.

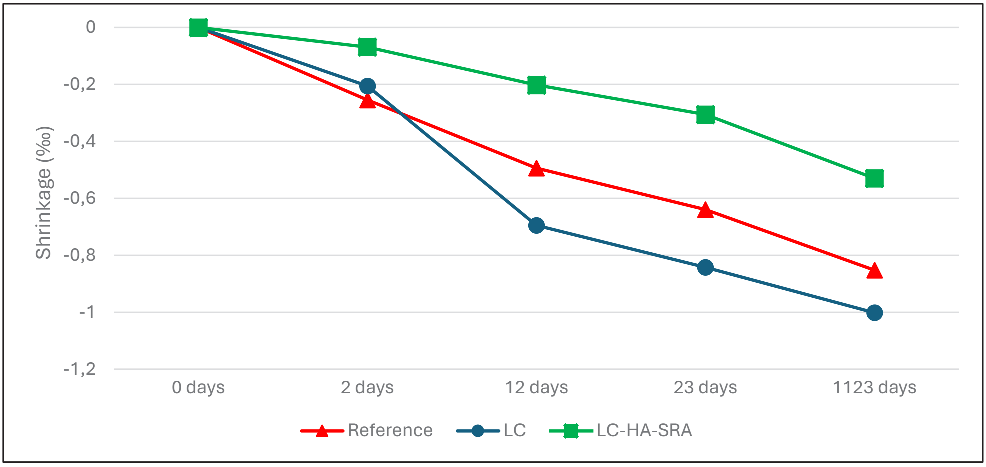


*Figure 4 – Development of shrinkage over time.*

After 2 days, there is a significant difference between the mix with SRA and those without. The shrinkage for LC-HA-SRA is only 0.07 ‰, whereas the LC is at 0.21 ‰ and the reference 0.26 ‰. At such an early stage, the strength – both compressive and tensile strength, is low and the concrete would be more vulnerable to cracking if shrinkage is high. The tendency of reduced shrinkage continues at 12 days, where LC-HA-SRA has a shrinkage of 0.20 ‰, which is what LC experienced after 2 days. At 12 days the reference has a shrinkage of 0.49 ‰ and the LC has 0.69 ‰. There is a considerable change in shrinkage for LC from 2 to 12 days. The measurements are an average of two specimens, and the variation is neglectable.

In Table 3 below, some of the requirements for classes 1 and 3 from [1] are referred to, compared to corresponding values for sprayed concrete. The SRA seems to perform well in sprayed concrete, as the shrinkage after 1123 days (approximately 3 years and 1 month) equals those of the reference mix after only 12 days. The last measurement shows a reduction of shrinkage when comparing LC concrete to the LC-SRA-HA concrete with an SRA addition of 47%. LC-SRA-HA compared to the reference mix which shows a reduction of shrinkage of 38%. The shrinkage for LC-SRA-HA is below 0.55 ‰, which is within the classification of flooring class 1, see Table 1. For both LC and Reference mix the shrinkage is above 0.75 ‰.

Table 3 - Requirements for flooring classes 1 and 3 according to [1] achieved after 1 year, compared to values for sprayed concrete measured after three years.

| | Class 1 | Class 3 | LC-SRA-HA |
|---|---|---|---|
| Shrinkage | ≤ 0,55 ‰ | ≤ 0,75 ‰ | 0.53 ‰ |
| Crack width | ≤ 0,3 mm | ≤ 1,0 mm | - |
| Maximum water content * | 160 kg/m$^3$ | 202 kg/m$^3$ | 210 kg/m$^3$ |

** Water content for sprayed concrete made with CEM II/A-V 42.5 R, which is the most common cement for sprayed concrete in Norway.*

The dosage of SRA used in this project is 4 kg/m3 or 0.85% of binder. This is a low dosage of SRA, according to the technical data sheet, which recommends dosages from 0.70 % to 2.0 %. SRA are often based on glycols like polypropylene glycol, and neopentyletylen glycol. The addition of glycols aims to reduce surface tension, which can potentially influence the effect of air entraining admixtures. Both admixtures are affecting surface tension of water in a fresh concrete mix. It is well known in the industry that entrained air reduces pumping pressure, which is beneficial for sprayed concrete. However, as the shrinkage reducing agent does not influence the binder, this should not affect the spraying nor hardening process. This is in accordance with the experience from spraying concrete with SRA performed in this project. However, this is a parameter which should be investigated further.

## CARBON EMISSION REDUCTION

The reduction in $CO_2$ emissions from the Reference mix to the LC mix is approximately 75 kg $CO_2$ pr $m^3$ concrete, resulting in a total emission of 199 kg/$m^3$ as mixed at the producer's facility. As the hardening accelerator is introduced, the total $CO_2$ emission is increased 209 kg/$m^3$ due to the CO2 emissions from 10 kg of HA. This concrete satisfies the requirements for "Low Carbon concrete Class A" according to NB 37 [4], where the limit for standard C35/45 is 210 kg $CO_2$ pr $m^3$ for stages A1-A3.

The guidelines NB 37 "Low Carbon Concrete" [4] was first published in June 2015, and updated in November 2019. The tests performed in this paper refer to the version of November 2019. The publication was later updated in May 2020 and March 2024, reflecting the rapid development on lowering carbon emissions from the concrete industry. In the May 2020 revision, it is stated that it is not valid for underwater concrete, lightweight concrete, or sprayed concrete. This was changed in the 2024 revision, where $CO_2$ emission $\leq$ 280 kg/m3 is classified as low-carbon sprayed concrete. That allows for the emission of 70 kg/$m^3$ more $CO_2$ than what was applied in these pilot experiments.

## CONCLUSION

The focus of the full-scale pilot study described in this paper was to contribute to utilising sprayed low-carbon concrete in road construction. It is concluded that:

1. The assumption that adding a hardening accelerator (HA) to ready-mix concrete might contribute sufficiently to counteract the retardation of low-carbon concrete, was not strengthened.

2. An addition of shrinkage shrinkage-reducing agent (SRA) to the ready-mix concrete did not negatively impact the behaviour of the concrete during spraying.

3. Even a relatively low dosage of SRA added to a ready-mixed low-carbon concrete contributed to reducing the shrinkages after spraying from 0.85‰ for the reference concrete that was used in a major E39 road construction project, to 0.53‰ for the experimental low-carbon concrete made for these tests. The content of Portland Clinker was reduced from 445 kg to 305 kg. When SRA was not included, the shrinkage of the low-carbon concrete was increased to 1.0‰.

4. Shrinkage reduction is an effective measure to increase the durability of concrete. In lack of any formal requirements made for sprayed concrete, the shrinkage behaviour of was compared to Norwegian requirements for avoiding shrinkage cracks in traditionally cast concrete slabs. Values achieved after three years for the sprayed low-carbon concrete were well within the one-year requirements for even the strictest class according to [1].

ACKNOWLEDGEMENT

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# THE DEVELOPMENT OF ACCELERATORS FOR SPRAYED CONCRETE

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**Abstract**

Accelerators for wet-mixed sprayed concrete can be divided into two main groups: Alkaline (high pH) and non-alkaline (low pH) accelerators. In most markets today the alkaline accelerators are considered problematic due to handling of caustic material, risk of promotion of alkali-aggregate reaction, and decrease in final concrete strength. Therefore, non-alkaline accelerators – often termed alkali-free accelerators – were developed to solve some of the problems caused by the alkaline accelerators. The typical non-alkaline accelerator on the market today is a weakly acidic solution or dispersion of aluminium sulphate blended with some additional ingredients. At high dosage aluminium sulphate causes rapid setting of cement.

Over the last 30 years or so the formulations of these non-alkaline accelerators have been modified and adapted to meet market requirements like performance and material costs. At the same time, it has been suggested, based on laboratory test results, that the high content of aluminium sulphate in non-alkaline accelerators may promote an internal form of sulphate attack in the concrete, but so far, no conclusive results have been found concerning the durability of sprayed concrete in the field. This uncertainty should call for the development of next generation of accelerators for sprayed concrete, maybe sulphate-free? Today's new binders with supplementary cementitious materials also call for further development of accelerators.

This paper presents a short state-of-the-art overview of the development of accelerators for wet-mixed sprayed concrete and provides some ideas about where to go next in terms of product development based on future challenges. A few examples from product development activities are presented.

## INTRODUCTION

The development of sprayed concrete technology – often referred to as 'shotcrete' – started more than 100 years ago in North America. In the 1920s the technology began to spread to other parts of the world. In those days it was purely a dry-mix process. Later, in the mid-1950s almost a revolution occurred with the development of the wet-mix sprayed concrete process. Since then, the wet-mix process started to find significant application [1].

In the wet-mix sprayed concrete process an accelerator – normally in liquid form – is added at the nozzle of the spray equipment, providing rapid setting and high early strength of the sprayed concrete [2,3,4]. Accelerator dosages are typically in the range of 6 to 8 % by weight of cement, sometimes even higher. These accelerators can be divided into two main groups:

- Alkaline accelerators - Traditional/conventional since 1960s, high pH
- Non-alkaline accelerators - New/modern accelerators since 1990s, low pH

The alkaline shotcrete accelerators are normally one-component materials, i.e. an alkaline compound dissolved in water as the sole active ingredient, while non-alkaline accelerators are blends of several ingredients dissolved in water. The non-alkaline accelerators are often referred to as 'alkali-free' accelerators.

Without the accelerating chemical added at the nozzle, sprayed concrete applications would not have gained its widespread use. The accelerators have played a key role in the development of sprayed concrete technology.

## PAST AND PRESENT ACCELERATORS

### Alkaline accelerators

There are three types of alkaline accelerators for wet-mix sprayed concrete. They consist of alkali salts dissolved in water, see Table 1. Alkali carbonates (e.g. sodium carbonate) have also been used to some extent, but almost entirely for the dry-mix process [5,6].

*Table 1 – Alkaline accelerators for sprayed concrete*

| Type of accelerator | Chemical composition | pH |
|---|---|---|
| Sodium silicate ('water glass') | $Na_2O \cdot (SiO_2)_n$, $n \approx 3.3$ | $\approx 11.5$ |
| Alkali aluminate | $NaAl(OH)_4$ or $KAl(OH)_4$ | $\geq 14.0$ |
| Alkali hydroxide | NaOH or KOH | $\geq 14.0$ |

*Sodium silicate*

Sodium silicate reacts with calcium in the cement paste to form solid calcium silicate, thus providing rapid setting [3,4]:

$$\text{Na-silicate (soluble)} + Ca^{2+} \text{(dissolved in fresh concrete)} \rightarrow \text{Ca-silicate (insoluble)} + Na^{+}$$

The precipitated calcium silicate will then form a hydrated calcium silicate gel (C-S-H), quite similar to the gel formed by the hydration of $C_3S/C_2S$ clinkers in the cement.

There are many grades of sodium silicate solutions which are categorised by their $SiO_2/Na_2O$ molar ratios. The molar ratio in a typical accelerator for sprayed concrete is approx. 3.3. This gives a relatively low alkali content compared to alkali aluminates, and with a pH value around 11.5. These high molar ratio solutions are classified as non-caustic. Sodium silicate still has a widespread use in sprayed concrete and is characterised by [3,5]:

- Compatible with all types of cement
- Less caustic than aluminates (pH lower than that of fresh concrete)
- Quite low early strengths
- Decrease in the E-modulus with time
- Reduced final strength at high dosages
- Decrease in the water proofing characteristics due to the extraction of lime when the concrete is subjected to a continuous exposure to moisture
- High viscosity at low temperature which requires heating of the material during shotcrete operations

Due to some of these attributes, especially the last 4 items listed above, the use of water glass as shotcrete accelerator was restricted in Germany and Austria more than 20 years ago [3].

*Alkali aluminate*

Aluminate ions ($Al(OH)_4^-$) react with calcium ($Ca^{2+}$) and sulphate ($SO_4^{2-}$) present in the liquid phase of the cement paste, forming ettringite (AFt) and AFm phases. Then, further $C_3A$ hydration proceeds with limited sulphate content and C–A–H phases might also be formed [7,8].

Unfortunately, the pH of alkali aluminate solutions has to be very high to keep the aluminate ion, ($Al(OH)_4^-$) in solution. This is sometimes achieved by addition of extra alkali hydroxide (e.g. NaOH), leading to a very caustic solution with pH up to 14.5 (!), and an alkali content as high as 15-20 % $Na_2O$-equivalents. Lowering the pH can cause precipitation of solid $Al(OH)_3$. The chemical reaction (equilibrium):

$$Al(OH)_4^-{}_{(solution)} \leftrightarrow Al(OH)_{3(solid)} + OH^-{}_{(solution)}$$

shows the effect of pH in aluminate solutions. When adding $OH^-$ (as NaOH) to the mix, meaning increasing pH, the reaction is forced to the left and the amount of soluble aluminate ($Al(OH)_4^-$) increases. Therefore, adding extra NaOH will increase the shelf life of alkali aluminate accelerators.

In some parts of the world, and particularly in Northern parts of Europe, the chemistry and performance of alkali aluminate accelerators are considered problematic due to:

- Highly caustic material that requires caution in handling (can burn skin)
- High amount of alkalis which may promote damaging alkali-aggregate reaction (AAR) in the concrete
- Decrease in final strength of the concrete

In the 1990s these problems, especially the AAR concern and the health hazard issue, initiated the development of non-alkaline, or alkali-free, accelerators.

**Non-alkaline accelerators**

Even though the non-alkaline accelerators were originally developed to solve some of the problems caused by the alkaline accelerators, the large market share of these accelerators today is mainly caused by the capability of these accelerators to improve setting and hardening properties, rather than their lack of 'alkali problems'.

The European standard EN 934-5 defines the term 'non-alkaline' as a chemical substance with an alkali content less than 1.0% ($Na_2O$-equivalent (%) = $Na_2O$ (%) + 0.658 $K_2O$ (%) $\leq$ 1.0). Today's liquid non-alkaline accelerators are, without exceptions, weakly acidic solutions of aluminium salts, sometimes in slurry form.

The exact chemical composition of liquid non-alkaline accelerators are closely guarded trade secrets. However, the main ingredient is, with hardly any exceptions, aluminium sulphate [4,6,9], which means they are all weakly acidic (pH=2-3) due to the hydrolysis caused by the dissolved aluminium salt. The dissolved $Al^{3+}$ ions have a strong effect on water molecules. These three-valent ions can split the water molecules – a phenomenon called hydrolysis – and form acidic $H^+$ (simplified):

$$Al^{3+} + H_2O = Al(OH)^{2+} + H^+$$

In order to neutralize this acidification and increase the pH one might try to add $OH^-$, but this will increase the risk of $Al(OH)_3$ precipitation:

$$Al(OH)^{2+} + 2\,OH^- \rightarrow Al(OH)_{3(solid)}$$

Therefore, an alkali-free accelerator can only remain dissolved in a solution state when weakly acidic. If pH is increased the accelerator will turn into a slurry state (suspension of solid particles in the liquid).

An alkali-free accelerator added to a fresh concrete will cause flash setting of the cement paste due to the chemical reaction between aluminium sulphate and lime ($Ca(OH)_2$) originating from the Portland clinker [10]. This reaction forms ettringite [9]:

$$Al_2(SO_4)_3 + 6\,Ca(OH)_2 + 32\,H_2O = \underset{\textit{Ettringite}}{3CaO{\cdot}Al_2O_3{\cdot}3CaSO_4{\cdot}32H_2O} + Al(OH)_3$$

The rapid setting is caused by the ettringite [9,11]. The formed aluminium hydroxide, $Al(OH)_3$, is also known to have an accelerating effect on cement hydration [9], and may therefore be added as a separate component in the accelerator [12]. The role of ettringite formation and the effect of the aluminium/sulphate ratio have been studied in detail [13,14].

In addition to the main component, aluminium sulphate, the alkali-free accelerators typically consist of 2-4 additional components. In a famous textbook on cement chemistry published more than 25 years ago, it is said that also ferric sulphate, triethanolamine and sodium fluoride are being used as ingredients in shotcrete accelerators [10]. When checking patent literature and safety data sheets from suppliers of alkali-free accelerators over the last 20 years or so, one can summarise the situation as follows:

- All contain aluminium salts
- Almost all (probably all) contain aluminium sulphate
- Many contain aluminium hydroxide
- Many contain amine (diethanolamine or triethanolamine)
- Some contain fluoride
- Some contain organic acids (e.g. formic, oxalic) and/or inorganic acid (e.g. phosphorous)
- Some contain glycerol
- A few contain magnesium salts
- Many suspension/dispersion types contain a clay based thickening agent to improve the stability of the liquid

A lot of effort has been put into understanding and optimising existing product chemistry based on aluminium sulphate and the ingredients listed above, rather than developing new ‘accelerating chemistry’ for sprayed concrete. One might say that R&D in a way is ’trapped’ in the aluminium sulphate chemistry. This chemistry has been tweaked and fine-tuned, but not really changed that much over the last 20 years.

New chemical thinking that enables the development of the next generation of accelerators for sprayed concrete might be triggered by two challenges:

- Potential durability issues related to the alkali-free accelerators
    - Sulphate attack
- New binders entering the market
    - Supplementary cementitious materials (pozzolanas)
    - Alkali-activated binders/geopolymers without Portland clinker

## NEXT STEP IN ACCELERATOR DEVELOPMENT

### Alkali-free and sulphate-free accelerators

Back in the 1990s the non-alkaline aluminium sulphate-based accelerators were developed partly due to the risk of alkaline accelerators promoting damaging alkali-aggregate reactions. A decade or so later, it has been questioned if the high content of aluminium sulphate in alkali-free accelerators can promote an internal form of sulphate attack in the sprayed concrete [15,16,17,18]. These studies were all based on lab tests, and no alarming results have been published for sprayed concrete in the field.

During recent years this accelerator/sulphate issue has been further discussed [19,20,21], but no clear advice has been found in terms of where to go next. However, when looking into the future perspectives, Wang et al concluded that "*The development of sulfur- and alkali-free accelerators is also an important direction*" [20]. Based on this, one should encourage the development of sulphate-free, or at least low-sulphate accelerators for sprayed concrete.

Attempts have already been made to develop sulphate-free accelerators. In 2003 one patent suggested the use of water-soluble aluminium fluoride as main component [22]. However, this involves handling of toxic hydrofluoric acid during manufacturing. Such sulphate-free products are not among the commercial accelerators found on the market today.

Another attempt has been to blend chemical compounds based on amines, phosphor, magnesium, and nano-alumina to make an alkali- and sulphate-free accelerator [23].

It is expected that further product development along these lines will continue.

### Accelerators for new binders

Another future challenge will be to develop accelerators for blended cements with a high amount of supplementary materials (pozzolanas). These binders tend to be less reactive than pure Ordinary Portland Cement (OPC). This calls for new accelerators capable of speeding up the pozzolanic reaction rates and possibly other reaction mechanisms.

An accelerator for pozzolanic reactions should be able to increase the solubility rate of the glassy phase of these materials (typically fly ash), i.e. the amorphous silica and alumina [24,25]. One way to do this is to increase the pH of the binder so that the glassy phase dissolves quicker. Adding sodium sulphate to the binder will form gypsum and sodium hydroxide when mixed with water and thus increase pH [25,26]:

$$Na_2SO_4 + Ca(OH)_2 + 2\,H_2O \rightarrow CaSO_4{\cdot}2H_2O_{(s)} + 2\,NaOH$$

However, this would not comply with attempts to develop alkali- and sulphate-free accelerators. Another attempt has been to use an organic compound capable of forming dissolved complexes with amorphous silica [26]. Some positive effect was found in blended cements with low OPC replacement (fly ash), but at high fly ash levels the effect was negligible.

Other attempts to speed up the setting and hardening of sprayed concrete containing blended cements have been to add accelerating materials in the concrete mix at the batching plant:

- A ‘dormant’ accelerating admixture (strength enhancer) as part of the consistence control admixture. The dormant accelerator is then activated by the spray accelerator in the nozzle during spraying [27].
- Nanoparticles of calcium-silicate-hydrate (C-S-H) acting as nucleation seeds during cement hydration [28].

Such developments are highly welcomed, but there is still a need to develop new accelerators for sprayed concrete to meet the challenges associated with durability issues and new binders.

**Product development attempts**

*Alkali- and sulphate-free accelerator*

An attempt to develop a completely new accelerator for future sprayed concrete applications have been made. The development aimed at making an accelerator free of sulphates and alkalis, and preferably a solution type (not dispersion/slurry) with long shelf life. The first simple version had the following characteristics:

- Colourless to pale yellow transparent aqueous solution:
    - pH = 2.5
    - Density = 1.52 g/cm$^3$
    - Viscosity = 500 cP (quite high)
- Composed of inorganic chemicals:
    - Alkali-free
    - Sulphate-free
    - Fluoride-free
    - Amine-free
    - No organic acids or compounds

The performance of this accelerator was compared with a standard commercial alkali-free accelerator, dispersion type with density 1,43 g/cm$^3$ and pH 2.5. When tested in laboratory (by hand mixing) at 7% dosage by weight of cement, the set times with the new accelerator were noticeable longer than for the standard accelerator (see Table 2). On the other side, the early age compressive strength after two hours was higher with the new accelerator. Unfortunately, the strength development slowed down after a few hours, but was close to the alkali-free accelerator (AFA) level after a few days (see Table 3).

*Table 2 - Set times in CEM II paste (7% accelerator dosage)*

| Accelerator type | Initial set time (min:sec) | Final set time (min:sec) |
|---|---|---|
| Alkali-free accelerator | 1:40 | 3:45 |
| New accelerator | 2:25 | 6:00 |

*Table 3 - Relative compressive strength in CEM II mortar (7% New accelerator dosage)*

| Curing time | 2 hrs | 24 hrs | 2 days | 3 days | 7 days |
|---|---|---|---|---|---|
| % of AFA strength | 122 | 44 | 67 | 93 | 96 |

Obviously, further development is needed to shorten the set times and increase the strength development in the period between a few hours up to 2-3 days. The key question is how to design the new accelerator to achieve acceptable strength after one day without losing the high strength after first couple of hours. However, the actual performance of this accelerator, as for all accelerators, can only be evaluated properly by carrying out concrete spray trials.

*Accelerator for alkali-activated binders/geopolymers*

Binders without Portland clinker, e.g. alkali-activated aluminosilicates and geopolymers, have been investigated heavily over the last years, and are utilized primarily for precast concrete [29,30]. These 'eco-friendly and green' binders are made by mixing waste materials, typically fly ash, with highly alkaline materials including concentrated aqueous solutions of sodium hydroxide and sodium silicate. Normally, curing at elevated temperatures is needed to gain acceptable early age strength [31]. A screening of existing accelerating admixtures for cast concrete did not show any significant accelerating effect on these binders [32].

The potential use of such binders for sprayed concrete is a new fascinating challenge. Obviously, this will trigger the development of a new kind of accelerators. One attempt has been made to accelerate a geopolymer fly ash binder containing only non-caustic sodium silicate as the alkali source. The highly caustic sodium hydroxide (normal part of geopolymer binders) was not used. A simple grout was mixed by blending three materials (approx. 1/3 of each by weight):

- Filler (finely ground crystalline silica)
- Fly ash (type F)
- Sodium silicate (36% solution, $SiO_2$:$Na_2O$ molar ratio 3.71)

This grout was sprayed at ambient temperature with the addition of a liquid accelerator at the nozzle of the sprayer. The accelerator was based on non-hazardous liquid organic compounds. At an accelerator dosage of 5-6% by weight of fly ash this grout set fast, like normal sprayed concrete, and the 2 hours compressive strength was about 1.8 MPa. However, the strength development after a few hours was low [33].

From an environmental and durability point of view, these new binders are promising for future concrete in general. Sprayed concrete is another matter which requires a step change in the development of accelerators. This should be encouraged for future product development of sprayed concrete.

## CONCLUSIONS

- Traditional accelerators for sprayed concrete, comprising sodium silicates and aluminates, have limitations linked to health hazards and/or performance properties (low final strength).
- The typical accelerator on the market today is non-alkaline, or alkali-free, and is based on aluminium sulphate with additional chemicals.
- Accelerator development over the last 20-25 years has been dedicated to fine-tuning and optimising alkali-free accelerators rather than developing new accelerator chemistry.
- A few attempts to develop alkali-free and sulphate-free accelerators have been done, but none has yet been commercialised.
- New Portland-free binders for sprayed concrete will trigger a step change in the development of new accelerators.

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**NORWEGIAN CONCRETE ASSOCIATION'S PUBLICATION NO.7 - SPRAYED CONCRETE FOR ROCK SUPPORT. NEWS FROM THE 2022-EDITION**

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**Abstract**

A revised edition of the Norwegian Concrete Association's publication no.7: Sprayed concrete for rock support (NB7) was published in December 2022. NB7 is a guideline giving best practice for production, execution and quality control of wet-mix fibre reinforced sprayed concrete [1]. The European standards are the starting point [2][3], but with adjustments to Norwegian experience and tradition. For larger tunnel projects NB7 works as a normative document since road, railway and water supply projects to a large extent call up NB7 in contracts. NB7 has the following four chapters: (1) specification, (2) test methods, (3) guidance on design of rock support and (4) guidance on specification.

The previous revision of NB7 was in 2011. At that time there was among others great focus, and substantial revision, on the panel test method and -procedure – this was presented at the sixth Sprayed Concrete Symposium in Tromsø, September 2011. This time there has been more a general technical and editorial update, but some adjustments have still appreciable character with consequences for concrete producer, contractor, and client. For instance, the revision committee spend some time on discussing, and revising, the routines related to the initial start-up fibre dosage in a project, and conformity/nonconformity levels for energy absorption capacity results from panel tests taken in a project. Furthermore, some control frequencies have been adjusted, and casting of samples for compressive strength on-site is no longer necessary (only drilled cores from hardened sprayed concrete). The paper givesmore details on the 2022-edition.

## INTRODUCTION

The Norwegian Concrete Association's publication no.7: *Sprayed concrete for rock support* (NB7) was first published in 1993 and has since then been revised several times. Until the first European standard concerning sprayed concrete was published in 2005, NB7 was regarded the Norwegian "standard" for the use of sprayed concrete as rock support. While the European standards include requirements for both wet- and dry mix methods, and for different types of applications: support and reinforcement of rock and soils; free-standing structures; and repairs and upgrading of existing structures, NB7 is limited to requirements concerning the use of wet mixed sprayed concrete for rock support.

NB7 is a supplement to the European standards and should always be used alongside these. Regardless, NB7 is often referred to in contracts. NB7 is written and revised by a committee with members from across the industry and is regarded as best practice in Norway.

NB7:2022 replaces NB7:2011.

## STRUCTURE

NB7 consists of four chapters. Chapters 1 and 2 are both relevant as descriptions in contracts, whilst chapters 3 and 4 provides guidance.

Chapter 1 *Specification of sprayed concrete for rock support* specifies the minimum requirements for the specification in contracts, as well as what additional requirements can be included. The specification is based on requirements provided by both the designer and the contractor. The chapter provides requirements for constituent materials and material properties of sprayed concrete; for production and application; for quality control and documentation; for measures in case of non-conformity; and finally measuring rules as a basis for payment.

Chapter 2 *Test methods* includes description of methods linked to required quality control and documentation given in chapter 1. Especially, the method for determination of the energy absorption capacity for fibre reinforced sprayed concrete is thoroughly described; this includes equipment, production of test panels, storage, test procedure and calculation of results. Other methods, particularly applicable for sprayed concrete, is measurement of final thickness and documentation of fibre content and fibre distribution.

Chapter 3 *Guideline for designing sprayed concrete for rock support – assessments of engineering geology and rock mechanics* provides advice on decisions about rock support and especially the use of sprayed concrete. In Norway, sprayed concrete used as temporary support must be suitable as part of the permanent support. Permanent rock support is selected based on experience, by evaluation of rock mass quality. Type of support is then established according to the Q-system, e.g., thickness of the sprayed concrete, bolting and use of reinforced ribs.

Chapter 4 *Guideline for specification of sprayed concrete* is directly linked to chapter 1 and gives supplementary explanation and guidance on most topics. Chapter 4 follows chapter 1 on chapter level two, e.g., chapters 1.3 and 4.3 are both named *Materials properties*. Chapter 4 can also, to some extent, be used as textbook material.

## SIGNIFICANT CHANGES

Although the changes this time was not as substantial compared to the 2011-revision of NB7, there are some changes that impacts the use of NB7, changes that affect both the contractor and the concrete supplier, and indirectly, the client. In the following, the most significant changes are summarized.

### General changes

Changes have been made to provide a clearer distinction on what is requirements and what is guideline. Whereas chapter 1 in earlier editions of NB7 not so systematically consisted of both requirements and text of more informative character, chapter 1 is now more customized to be used as basis for specification in contracts and indicative text have been moved to chapters 3 and 4. Remaining explanatory text in chapter 1 is regarded directly necessary for the understanding of the requirements. Furthermore, requirements and tolerances related to testing and documentation of material properties are given in chapter 1, while the actual test methods are described in chapter 2. Both these chapters are therefore relevant in contracts.

One objective of the revision was to improve readability and by that make the content more accessible for the reader. This was resolved by directly linking indicative text to the given subjects with subchapters in chapter 1 having parallel subchapters in chapter 4, as mentioned above. In this way, it is much easier to find explanations and guidance if needed.

Other changes of more general character were referring to standards, instead of repeating requirements; that guidance regarding the design of sprayed concrete for rock support was given a separate main chapter (chapter 3); and also, that the content from an appendix published in 2015 is now integrated into the 2022-edition.

**Technical changes**

When it comes to technical changes, there were two particular subjects that were thoroughly discussed in the committee and that led to changes in the publication. These were the understanding of the routines regarding choice of initial fibre type and fibre dosage with subsequent understanding of what is considered conformity/nonconformity, and requirements regarding quality control of fibre content and fibre distribution. The latter implies less testing. In addition, there are changes regarding testing of compressive strength, as well as some minor technical changes.

*Initial fibre type and fibre dosage*

During the revision, it became clear that there were differing interpretations of the rules for selecting fibre type and fibre dosage at startup, as well as what constituted deviations in documentation during project execution. During the discussion, it became clear that the description in the previous edition of NB7 was unclear and a direct cause to the disagreement between contractors and clients.

In both the previous and current editions of NB7, there are two ways described for selecting the initial fibre type and corresponding dosage: either based on previously obtained test results or through pre-testing in the actual project. The disagreement was related to the choice of fibre type and fibre dosage based on previously obtained results. In the previous edition of NB7, margins were provided for the achieved energy absorption capacity, where the magnitude of the margin was depending on the number of test results. The contractor interpreted this to mean that if the requirements, including the margins, were met, it entailed a pre-approval of the chosen fibre type and fibre dosage. Consequently, if the first testing in the project yielded a lower energy absorption capacity than specified, it was not considered a deviation that, in addition to an adjustment of fibre dosage, required assessment of potential consequences with subsequent deviation handling. From the client's perspective, this interpretation posed an unacceptable risk, over which they had no control, considering the safety of the construction. In some cases, it could also raise suspicions that the contractor selectively chose previous test results to their advantage.

Although it could be discussed just how accurate the requirements regarding energy absorption capacity are, considering that choice of rock support is based on empirical data, changes were made in NB7 to ensure equal understanding. The contractor still has the opportunity to choose initial fibre type and fibre dosage based on previously obtained test results, and this method is overall preferred over pre-testing in the actual project. The difference now is that the contractor "only" must substantiate that the chosen fibre type and corresponding dosage will entail that the requirements for energy absorption capacity is satisfied. The previous required margins are now given as recommended values in the guidance in chapter 4. Since the choice of fibre type and fibre dosage lies with the contractors, the risk of a possible nonconformity when testing during project execution is also transferred to them. Experience with these changes must be assessed next time NB7 shall be revised.

*Quality control of fibre content and fibre distribution*

The subject of requirements regarding quality control of fibre content and fibre distribution was also thoroughly discussed in the committee, but with much less agitation. Whereas the previous edition of NB7 required testing and documentation of fibre content and fibre distribution independent of how the fibre is added to the concrete mix due to recurring challenges with fibre distribution in projects, now, initial testing and conformity control refer entirely to NS-EN 206+NA. If the fibre is added to the concrete mix directly in the mixer at the concrete factory, no testing is required regardless of whether it is added automatically or manually. The only documentation needed is conforming that required quantity is added. If the fibre is added to the concrete mix in the concrete truck, the fibre content and fibre distribution in the fresh basic mix must be verified according to NS-EN 206+NA.

However, when the concrete mix is used for production of test panels for determination of energy absorption capacity, the fibre content and fibre content distribution must be determined on samples from the same truck, regardless of the method of fibre addition. This control is part of the contractor's identity control.

*Minor technical changes*

For the contractor, it is no longer required to perform testing of compressive strength on cast cubes. Control of compressive strength (and density) at the site is linked to cored cylinders from finished sprayed concrete. For Execution class 3, the control frequency has been increased.

Other changes of minor extent include clearer requirements regarding control of bond strength, control of thickness, and clarifications and corrections regarding curing measures. In addition, chapter 4 now includes guidance on two subjects that are not part of the specification in chapter 1, subjects still useful to inform about. The first gives good advice regarding execution of rock support in existing/older tunnels, and the second gives a brief overview of the use of sprayed concrete for fire protection of flammable insulation used for water- and frost protection, where mainly the execution deviates from the execution of sprayed concrete for rock support.

A new chapter about particularly large and very small projects have been added (chapter 4, guideline). It discusses that in special cases it can be reasonable to consider a reduced control frequency for energy absorption capacity. Any such decision must be agreed upon among the involved parties in the given project.

### Other topics of discussion

Sustainability is an important subject, and production of cement and concrete contributes to a large extent to greenhouse gas emissions, this also applies to sprayed concrete. NB7 does not have its own rules regarding low carbon sprayed concrete, but the subject is briefly mentioned in the guidelines in chapter 4. The Norwegian Concrete Association has a separate publication on low carbon concrete, publication no. 37 [4] (available in English autumn 2024). Although the low carbon classes given for structural concrete in NB37 is not directly applicable for sprayed concrete, the same means to reduce greenhouse gas emissions can be applied for sprayed concrete. NB7 refers to NB37 in chapter 4.

Sprayed concrete with reduced $CO_2$-emission have been tested out in full scale trials by means of using around 30% fly-ash of binder [5]. It was shown, through some mix adjustments, that it was sprayable, it obtained satisfactory early strength on the short run and both final strength and energy absorption capacity later.

## SUMMARY

A new edition of the Norwegian Concrete Association's publication no.7: *Sprayed concrete for rock support* (NB7) was published in December 2022, and replaced the 2011-edition. Overall changes have improved readability, and especially a much clearer distinction between requirements and corresponding guidance has enhanced the use of the publication. Changes of more technical nature includes, among others, clarification regarding selection of initial fibre type and fibre content, and unified understanding of is considered conformity/nonconformity during testing in projects, changes in requirements regarding documentation of fibre content and fibre distribution, and there have also been relaxation regarding the contractors' documentation of compressive strength. Although requirements regarding sustainability have not been directly implemented in NB7, the subject was widely discussed during the revision.

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## ECONOMICAL STEEL FIBRE REINFORCED SPRAYED CONCRETE WITH LOW $CO_2$-FOOTPRINT

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**Abstract**

In addition to mechanical properties, durability, and economy, sprayed concrete mixes are increasingly focusing on ecology. To fulfil all requirements, it is important that the materials used are well matched to each other and also have the lowest possible $CO_2$-footprint.

The main focus of this article is to provide assistance in selecting the most technically suitable steel fibres for the respective sprayed concrete mix in order to achieve the required fresh and hardened concrete properties in a targeted and economical manner for the application. For this purpose, a parameter study is presented and carried out based on test results in which different fibre types - variation of length, diameter, and steel tensile strength - are used to select the optimum fibres. According to the available findings, this selection can reduce the fibre content by up to 45 %.

Furthermore, comparative calculations are used to show how high the $CO_2$-savings-potential is. On the one hand, the effect of reducing the dosing quantity is shown. On the other hand, an overview of the range of $CO_2$-footprints of standard fibres from different manufacturers with comparable dimensions and steel tensile strengths is provided.

Finally, case studies of real construction projects are presented to realistically estimate the savings potential. The information in this article shows that the best possible selection of a technically suitable steel fibre can make a significant contribution to reducing the environmental impact of sprayed concrete mixes and, at the same time, enable economic benefits.

Keywords: Sprayed concrete, steel fibres, Global Warming Potential, $CO_2$-saving

## 1 INTRODUCTION

The reduction of $CO_2$-emissions is becoming increasingly important worldwide. Norway has a pioneering role in this area, partly due to its history. Under the leadership of Norwegian Prime Minister Gro Harlem Brundlandt, the "Brundlandt Commission" developed the guiding principle "Development that meets the needs of the present without compromising the ability of future generations to meet their own needs" on behalf of the United Nations in 1987 [1].

This was the starting point for the work in Norway on sustainability, which is also being actively promoted by Norwegian industry in particular [1].

Consequently, the focus is also on sustainability in the field of tunnelling. This article takes a closer look at sustainability, particularly the $CO_2$-emissions (Global Warming Potential - GWP) of (steel fibre-reinforced) sprayed concrete and highlights the $CO_2$-savings potential.

There are two major levers for reducing the $CO_2$-footprint:

- Reduction of the concrete volume
- Use of raw materials with the lowest possible GWP

The following chapter will first show why the use of steel fibres for permanent sprayed concrete tunnel linings can help to reduce the concrete volume.

## 2 PERMANENT SPRAYED CONCRETE LININGS

### 2.1 General information

The use of steel fibre-reinforced sprayed concrete for the permanent lining of tunnels makes it possible to reduce the volume of concrete [2]. A comparison of Figures 1 and 2 and Table 1 shows that the Permanent Sprayed Concrete Lining(s) (PSCL) enables an overall thinner solution than a conventional solution as Double Shell Lining (DSL).

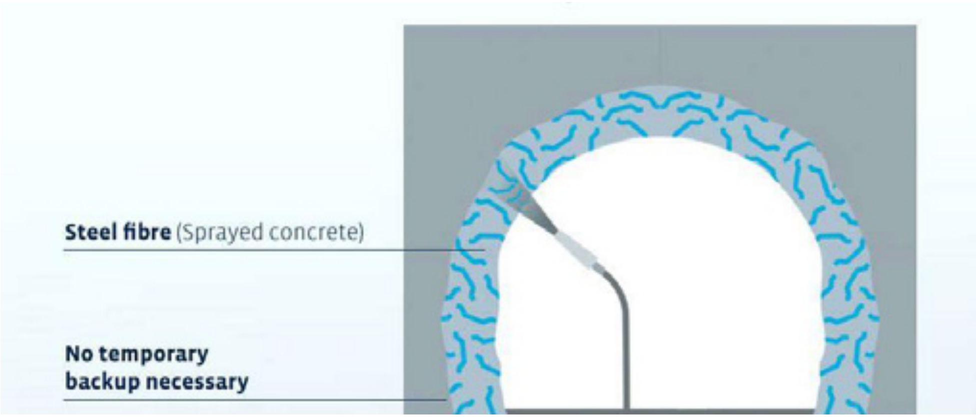


*Figure 1 - Solution with steel fibre reinforced sprayed concrete [3]*

Due to the ever-advancing development of the technology used, such as spraying robots, as well as constantly improving control and design methods, standards and guidelines, sprayed concrete is being used more and more frequently as a permanent lining method. In particular, the use of steel fibre-reinforced (see Figure 1) for (permanent) protection (final tunnel lining) is therefore increasing worldwide.

In Norway, the use of synthetic fibres has become rare and is not permitted for some applications for environmental reasons, as the fjords have been heavily polluted by floating plastic fibres [4]. The use of steel fibre-reinforced sprayed concrete offers numerous advantages:

1. Lower maintenance costs thanks to reduced cracking.
2. Concrete surface is much less sensitive to potential damage.
3. Faster completion of the tunnel through time- and cost-saving construction processes.
4. Simple handling: Only one step in the reinforcement process as the fibres are applied together with the concrete.
5. There is no need for temporary support because conventional reinforcement isn't installed, and the sprayed concrete can be applied directly as a finished layer.
6. Significantly increased service life thanks to the greater durability of fibre concrete.
7. High fire resistance when Polypropylene (PP) micro-fibre concrete is used.

Conventional lining (see Figure 2) has several disadvantages compared to the sprayed concrete solution:

1. The surface of the concrete is unprotected and susceptible to damage resulting from the greater spacing with classic reinforcement.
2. Installation of the steel bar reinforcement is considerably more time-consuming as the reinforcement is applied in a separate work step.
3. Very complex reinforcement situation with an irregular surface and holes.
4. Complex and close-mesh reinforcement can easily result in "spray shadows" (voids), which in turn can quickly lead to water ingress and further cracks.

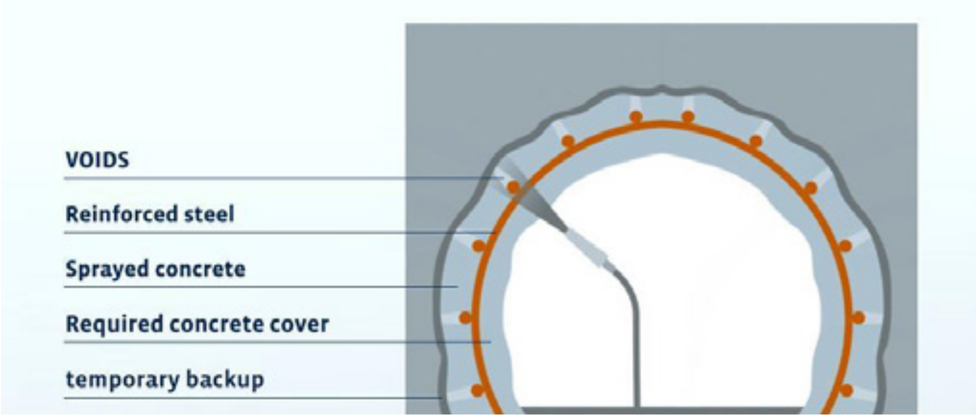


*Figure 2 - Solution with conventional reinforced sprayed concrete [3]*

The explanations described above are also discussed in detail in the ITA Report "Permanent Sprayed Concrete Linings" [2] and excerpted in the following chapter 2.2.

## 2.2 GLOBAL WARMING POTENTIAL OF PSCL

To obtain a valid basis of information for the life cycle assessment of a product, an Environmental Product Declaration (EPD) is often required to disclose the $CO_2$-balance of building products and thereby promote sustainable construction.

An EPD is a comprehensive, independently verified, and registered product passport. It contains life cycle information, characteristics of the life cycle analysis and test results for a detailed assessment of building materials and construction products. They are based on the international standard ISO 14025 [5]. Regarding the construction industry, EPDs are based on the EN 15804 standard [6] for construction products, services, and processes.

An EPD is ideal for communicating the environmental performance of building products and thus promoting sustainable construction. In the requirements for sustainable products, the focus today is primarily on the Global Warming Potential (GWP).

In the further course of the article, the GWP is illustrated using EPD.

Table 1 below shows the assumptions in [2]:

Table 1 - Key parameters for the comparison between PSCL and DSL carbon footprints [2]

| Item | Hard rock PSCL | Hard Rock DSL |
|---|---|---|
| Primary lining concrete thickness (mm) | 80 | 80 |
| Primary fibre/bar (kg/m³) | 40 | 40 |
| | Steel fibre | Steel fibre |
| Membrane | 40 | 0 |
| | SAWM | PVC Sheet |
| Secondary lining concrete thickness (mm) | 80 | 300 |
| Secondary fibre/bar (kg/m³) | 40 | 97 |
| | Steel fibre | Steel bar |

In [2], a sprayable waterproofing membrane (SAWM), which requires a regulating layer (a total thickness of 40 mm is assumed), is selected for waterproofing the PSCL option, and a conventional PVC waterproofing membrane is selected for the DSL options.

Sprayed concrete formulations usually have a higher cement content and therefore a higher GWP per cubic meter, but this is significantly reduced due to the low concrete volume (see chapter 2.1), resulting in a lower GWP overall (see Figure 3). The content of steel fibres is generally also significantly lower than the total amount of conventional reinforcement made of steel bars (see Table 1 and Figure 3).

The calculations show that a PSCL variant can result in GWP savings of approx. 20 to 50 % compared to the DSL variant (see Figure 3).

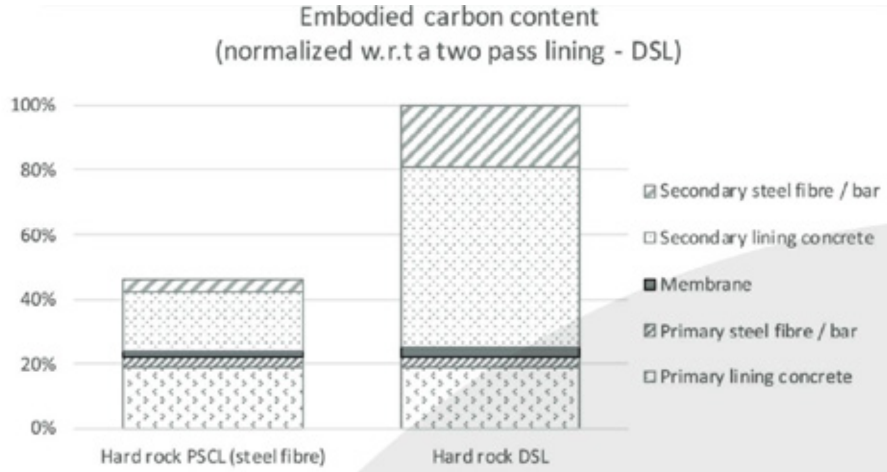


*Figure 3 - Embodied carbon for hard rock tunnel linings (normalized w.r.t a two pass lining - DSL) [2]*

In summary, it can be said that significant quantities of concrete can be saved, particularly with steel fibre-reinforced sprayed concrete.

## 3 $CO_2$ -SAVING AND RESPONSIBILITY

The main emitter of $CO_2$ from concrete is cement. The higher the proportion of clinker, the higher the $CO_2$-emissions. In the publication “Sustainability in Norwegian Tunnel” [1] reducing the proportion of clinker is identified as one option.

Unfortunately, practice has shown that the mixes have not changed significantly over the last decade. The reason for this is not a reluctance on the part of concrete manufacturers, construction companies or clients to use other cements, but rather the existing requirements for sprayed concrete. Both the required early strength and occupational safety still largely prevent the use of alternative cements.

This means that $CO_2$-savings can be achieved "only" through the other raw materials used, in addition to those already achieved through the steel fibre-reinforced construction method.

With our partner Mapei AS, we have therefore been endeavoring for over 10 years to find the most efficient sprayed concrete possible to reduce the dosing quantity as much as possible. In addition to a direct reduction in the $CO_2$-footprint of the formulation, $CO_2$-emissions during transport should also not be ignored, as significantly fewer journeys are required for the construction work.

The fibre producer takes the issue of responsibility very seriously, which is reflected in four areas:

- Consistent actions,
- fair and humane treatment,
- sustainability and
- reliability

As a manufacturer, we consume energy and impact the environment around us. However, it is important to us that we keep this impact as low as possible: since 2015 we only purchased 100 % green electricity from renewable energy, maintain short, low-emission delivery distances (90 % of the raw material is delivered by ship in an environmentally friendly way), ensure a high recycling rate of our products, and promote sustainable mobility and energy among our employees and in the company.

In the following, the development and the $CO_2$-saving-potential is shown based on real tunnels.

## 4 CASE STUDY

The case study compares two Norwegian tunnelling projects. Firstly, the Ryfast tunnel, which was built between 2015 and 2018, and the Rogfast tunnel, which has been under construction since 2023.

While the normal-strength fibre DE 35/0.55 N with a length of 35 mm and a diameter of 0.55 mm with a tensile strength of 1,250 $N/mm^2$ was used for the Ryfast tunnel, a 40 mm long fibre with a diameter of 0.55 m and a tensile strength of 1,800 $N/mm^2$ was used for the Rogfast tunnel.

Until around 2019, a fibre with a length of 35 mm and a diameter of 0.55 and a tensile strength in the range of approx. 1,200 to 1,300 N was commonly used worldwide for permanently applied sprayed concrete and is still frequently used today.

The main parameters of the two tunnels are summarized in Table 2.

Table 2: Parameter Ryfast and Rogfast tunnel

| Parameter | Unit | Ryfast tunnel | Rogfast tunnel |
|---|---|---|---|
| Total length | m | 28,600 | 51,000 |
| Sprayed thickness | | 0.10 | 0.10 |
| Sprayed concrete | $m^3/m$ | 2.27 | 2.40 |
| GWP concrete - A1 to A4[1)] | kg $CO_2/m^3$ | 277.929 | 283.030 |
| Fibre | - | DE 35/0.55 N | DE 40/0.55 M |
| Fibre dosage[2)] | $kg/m^3$ | 28.00 | 21.75 |

1) Without fibres
2) ~ 75 % E700 and ~ 25 % E1,000

The concrete used for both tunnels had to fulfil the requirements of strength class B35 and durability class M40. As is generally known (see chapter 2.2), the cement content is somewhat higher in sprayed concrete mixes with steel fibres and is around 448 $kg/m^3$ in this mix, which accounts for the majority of the GWP. The difference in GWP between the two tunnels per cubic meter of concrete is due to the slightly longer transport distances for Rogfast tunnels.

The detailed composition of the formulation is shown in the Table 3.

The steel fibre-reinforced sprayed concrete used in both tunnels had to meet the requirements for energy absorption class E700 for around 75 % of the areas and energy absorption class E1000 in accordance with EN 14887-1 [7] for the remaining approx. 25 %.

This results in an average dosage of 28 kg/m³ of DE 35/0.55 N for the Ryfast tunnel and 21.75 kg/m³ of DE 40/0.55 M for the Rogfast tunnel.

Table 3: Mix design for 1 m³ sprayed concrete B35 M40 D8 Spr 220 mm - E700 and E1000 [8]

| Raw Material | Unit | Content |
|---|---|---|
| Air entrainer | kg/m³ | 1.00 |
| Supplementary Cementitious Materials (SCM) | | 49.76 |
| Cement | | 447.85 |
| Superplasticizer | | 4.98 |
| Aggregates | | 1,573.00 |
| Water | | 188.00 |

By using the DE 40/0.55 M instead of the DE 35/0.55 N, a reduction in dosage of around 22 % was achieved in the Rogfast tunnel - with the same performance (see Figure 4, left).

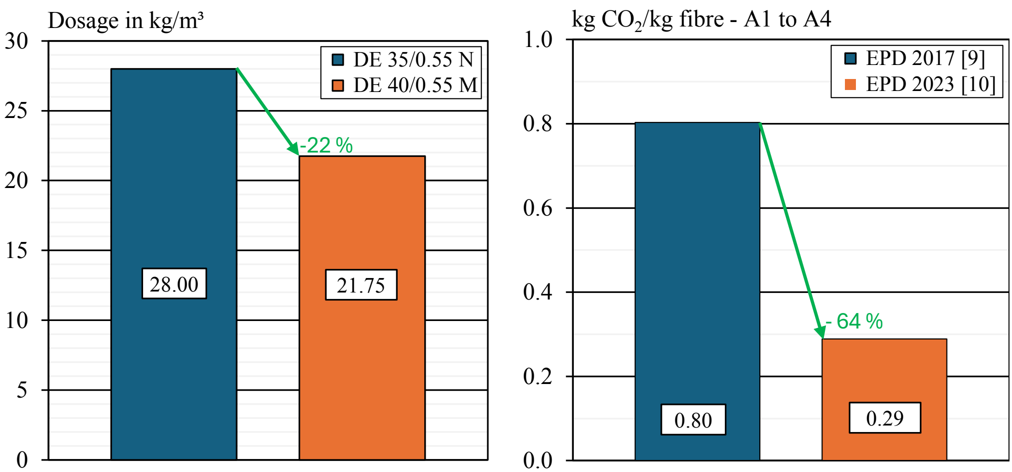


*Figure 4 – Reducing the dosage (left) and reducing the GWP [9, 10] (right)*

In addition to reducing the dosage, the GWP of the steel fibre has also been significantly reduced since 2017. While the value for the production stage (A1 to A3) in 2017 [9] was still around 0.771 kg $CO_2$ /kg fibre, this was significantly reduced to 0.257 kg $CO_2$/kg fibre by 2023 [10]. The share of transport A4 was simplified (basis [10]) to 0.032 kg $CO_2$/kg fibre. This value is used in the following to simplify the categorization of the results for the known range of GWP of comparable fibres and can be classified as very conservative for this.

Thanks to the progressive requirements and our customers and partners in Norway KrampeHarex has developed to the leading manufacturer for high performing steel fibres with the lowest GWP [10]. This was a long way which we continue mindful and persistent.

KrampeHarex has been sourcing 100 % of its electricity from renewable energy sources since 2015. The installed photovoltaic system produces more than 1.5 GW per year, which allows us to reduce our energy purchase by 15 %.

We also have established an energy management system according to ISO 50001 [11] what forces us to increasing efficiency constantly.

As a result of the measures described, the GWP was reduced by approx. 64 % (see Figure 4, right).
Table 4: Comparison of the GWP of different fibre types and possible $CO_2$-savings

| Fibre typ | Dosage | GWP A1 to A4 | | $CO_2$-saving related to the reference |
|---|---|---|---|---|
| | kg/m³ | kg $CO_2$/kg fibre | kg $CO_2$/m³ | % |
| DE 35/0.55 N - reference | 28.00 | 0.80 [9] | 22.48 | - |
| DE 40/0.55 M | 21.75 | 0.29 [10] | 6.29 | 72.04 |
| Comparable fibre „low“ | 21.75 | 0.83 | 18.10 | 19.52 |
| | 7.56 | | 6.29 | 72.04 |
| Comparable fibre „high“ | 21.75 | 2.03 | 44.20 | - 96.57 |
| | 3.09 | | 6.29 | 72.04 |

Table 4 shows the GWP (A1 to A4) for the two fibres DE 35/0.55 N and DE 40/0.55 M as well as the range of known comparable fibres. Comparable fibre "low" represents the lower limit with a value of 0.83 kg $CO_2$ /kg fibres and comparable fibre "high" represents the upper limit with a value of 2.03 kg $CO_2$ /kg fibres. As explained above, a value of 0.032 kg $CO_2$/kg fibres was used for A4 for all four fibres, which is probably far too low for the comparable fibres.

In addition to the determined GWP values (A1 to A4) per cubic meter of concrete, the $CO_2$-savings-potential was also calculated in relation to the reference (DE 35/0.55 N, 28 kg/m³, EPD 2017 [9]). For the two limit values, a dosage of 21.75 kg/m³ was applied, as for DE 40/0.55 N. On the other hand, the content was calculated at which the same GWP is achieved as for DE 40/0.55 M with a dosage of 21.75 kg/m³.

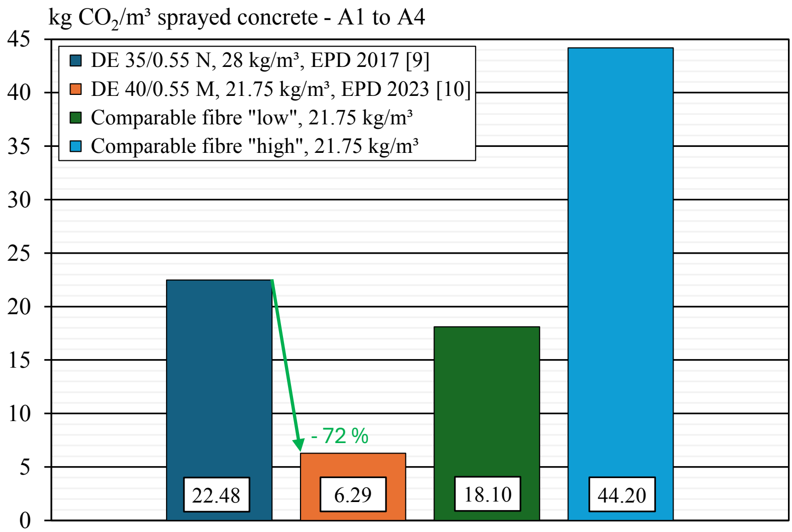


*Figure 5 – Comparison of the GWP – A1 to A4 - in kg $CO_2$/m³ sprayed concrete of different fibres*

Figure 5 shows that the low dosage of DE 40/0.55 M (21.75 kg/m³) in combination with the improved EPD results in a reduction in GWP of around 72 % for the fibres in the Rogfast tunnel compared to the Ryfast tunnel with DE 35/0.55 N and a dosage of 28 kg/m³. The comparison with the comparable fibres - dosage 21.75 kg/m³ - shows that with a low $CO_2$-footprint, there would only be a savings potential of just under 20 % and with a high $CO_2$-footprint almost double the $CO_2$-emissions compared to the Ryfast tunnel.

Furthermore, a comparison is made of the maximum dosing quantity that is possible to comply with the 6.29 kg $CO_2$/m³ as with the DE 40/0.55 M in the Rogfast tunnel. The results are shown in Figure 6. With the DE 40/0.55 M, 288 % (comparable fibre "low") or 709 % (comparable fibre "high") more can be dosed than with the comparable fibres. Conversely, only 7.56 kg/m³ (comparable fibre "low") or 3.09 kg/m³ (comparable fibre "high") could be used to comply with the same $CO_2$-footprint.

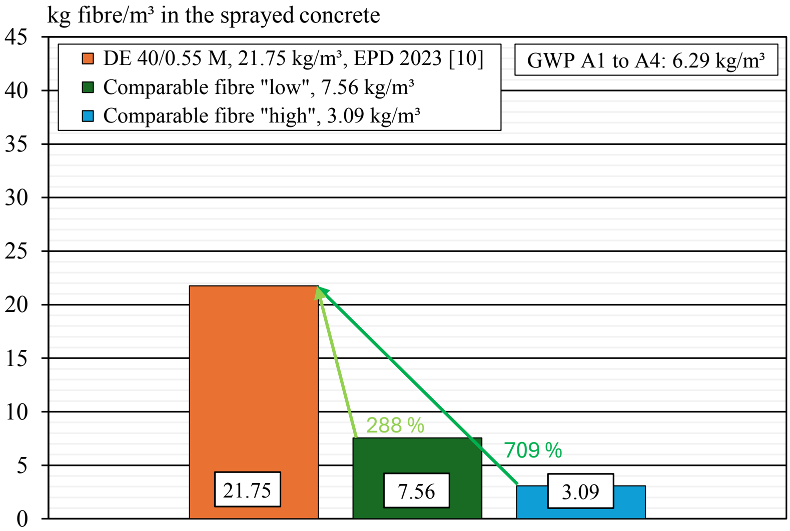


*Figure 6 – Maximum dosage for reaching the same GWP A1 to A4 of 6.29 kg/m³ sprayed concrete*

Chapters 2 and 3 have already explained that the use of steel fibre-reinforced sprayed concrete and a reduction in the proportion of clinker can make a considerable contribution to significantly reducing the GWP. According to [2], potential savings of 20 to 50 % are possible.

In this case study, the focus was "only" on the savings potential through the selection of the right fibres. Table 5 therefore only considers the effects of the fibre. The use of DE 40/0.55 M in the Rogfast tunnel resulted in $CO_2$-savings of approx. 3.7 % on the entire concrete, which corresponds to around 1,373 tons.

Table 5: GWP Tunnel with fibres

| Parameter | Unit | Ryfast tunnel | Rogfast tunnel |
|---|---|---|---|
| Total length | m | 28,600 | 51,000 |
| Sprayed thickness | | 0.10 | 0.10 |
| Sprayed concrete | m³/m | 2.27 | 2.40 |
| GWP concrete - A1 to A4[1)] | kg $CO_2$/m³ | 277.93 | 283.03 |
| Fibre | - | DE 35/0.55 N | DE 40/0.55 M |
| Fibre dosage[2)] | kg/m³ | 28.00 | 21.75 |
| GWP fibre | kg $CO_2$/m³ | 22.48 | 6.29 |
| GWP concrete + fibres | | 300.41 | 289.32 |
| $CO_2$-savings | % | - | 3.7 |
| | kg | | 1,373,385.60 |

1) Without fibres
2) ~ 75 % E700 and ~ 25 % E1,000

## 5 CONCLUSION AND OUTLOOK

Sustainability is becoming increasingly important in the construction industry and therefore also in tunnelling. Norway is playing a pioneering role here.

The use of steel fibre-reinforced sprayed concrete for permanent tunnel linings can make a major contribution here, as significantly less concrete is used, and less excavated material must be removed from the tunnel.

As the concrete formulas for sprayed concrete in Norway are currently not yet being changed significantly due to the requirements for early strength and occupational safety, a good selection of a high-performance fibre with a low $CO_2$-footprint can contribute to sustainability, especially in the raw materials.

As part of this article, the Ryfast and Rogfast tunnels were analyzed in a case study. It was shown that with the DE 40/0.55 M it was possible to reduce the dosage by approx. 22 % while maintaining the same performance requirements. In addition, the $CO_2$-footprint of the fibre has been significantly reduced in recent years thanks to a consistent selection of raw materials, production using green electricity and comparatively low-$CO_2$-emission transport by ship and rail to Norway, resulting in a reduction in GWP of around 72 %. Compared to comparable fibres, up to 700 % more fibres can be added with the same GWP.

It is known from other projects that potential savings of up to 45 % can be achieved in terms of fibre dosage with the DE 40/0.55 M, so that the $CO_2$-footprint can be further reduced in relative and absolute terms.

In addition to the $CO_2$-savings, the significantly lower dosages also result in considerable reductions in reinforcement costs, which will further promote the use of steel fibres in sprayed concrete construction in the coming years.

We would like to thank Ølen Betong AS and Mapei AS for their support with this article!

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OPTIMIZING TUNNEL CONSTRUCTION:
*THE IMPACT OF REAL-TIME DATA THROUGH DRIV AND ES-SHOTCRETE ON MANAGEMENT, DOCUMENTATION AND ACCURACY*

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## ABSTRACT

This paper presents DRIV, a full-fledged digital reporting system developed by Skanska Digital, specifically designed to enhance the documentation process in tunnel construction. DRIV incorporates a specialized "ES Shotcrete" module which leverages real-time data from concrete suppliers, significantly improving the accuracy and efficiency of shotcrete documentation. By integrating these data-streams, we are able to reduce human error and expedite administrative work, while enabling real-time project evaluation for management. The system has been implemented effectively, demonstrating improvements in documentation speed, error reduction and overall project insight. Field workers report notable improvements in productivity and ease of use. The success of this system underscores the potential of integrating supplier data in construction technology, and the benefits of digital reporting for improved efficiency, decision-making and economic predictability.

## INTROUCTION

Tunnels are critical to modern infrastructure, facilitating urban development and connectivity through water distribution, sewage, transportation, and utilities. The global urbanization-boom has increased both the number and complexity of tunneling projects, resulting in a growing demand for sustainable infrastructure with accompanying detailed and high-quality documentation. Traditionally, contractors and builders have been reliant on paper-based processes which are error-prone and inefficient.

The implementation of digital technology offers a transformative opportunity, enabling streamlined documentation and an enhanced project management methods through increased accuracy and real-time insights. Skanska Digital's DRIV with its ES-Shotcrete module that specifically improves the documentation of shotcrete demonstrates this advancement. By replacing paper-based methods with digital tools, we not only address the inefficiencies that go along with paper-based systems, but we also enable real-time data sharing and analysis, revolutionizing tunnel construction management, execution and documentation.

This paper explores the implementation of this system in tunnel construction and demonstrates how it significantly improves efficiency and accuracy. It also looks at the broader implications of digitization in construction, offering insights into potential future innovations.

## BACKGROUD ES-MODULE

The ever-evolving regulatory framework coupled with the complexity of modern tunneling projects has led to a dramatic increase in the amount, level and detail when putting together documentation for a tunneling project. This has in turn resulted in a significant burden for the project administration.

Traditional paper-based reports are not only time-consuming to manage, but they are a significant time burden for field workers, which led to being a source of irritation, which further led to frequent careless human-errors when field workers were rushing to complete mandatory paperwork. As previously mentioned, error correction is a tedious process. Every source of error that is eliminated leads to massive time savings.

For a contractor specializing in shotcrete placement, the purchase of concrete represents a substantial portion of operational expenses. One of the primary financial risks associated with the delivery of shotcrete include the possibility of having to “dump” concrete (return to plant). If the concrete has to be returned, it is essential that we can document the reason it was returned, be it the quality of the concrete, a malfunction in our machinery, etc. Concrete suppliers also charge for delays in delivery, so it is crucial that we are able to document if the delay is due to the concrete being outside of spec (hard to pump), if it’s a site-access issue, or an issue in relation to our equipment.

All of these charges show up on an invoice from the concrete supplier, which we have to audit to ensure that we’re not charged for an expense that doesn’t belong to us. Paper reports meant that these auditing sessions were extremely time consuming.

An example of common challenges when auditing a concrete invoice: Usually, concrete suppliers split invoices in regard to a specific site delivery address, which means we can get multiple invoices for the same stretch of time, per site delivery address. If every delivery goes 100% according to plan, auditing invoices is usually somewhat straightforward. However, as any project can attest to, things never go 100% according to plan. If there was trouble at one site-location, concrete would be sent to another location. The re-routing of concrete isn’t caught by the manufactures invoicing system, which leads to a time-consuming process of spreading out every shotcrete report and corresponding concrete slip, alongside invoices for comparison. Despite thorough examinations, missing information frequently led to an inability to contest unjustified additional charges.

Furthermore, addressing inquiries about specific deliveries involved a labor-intensive search through extensive paperwork, a process mirrored by our contractors. This situation emphasized the urgent need for digital transformation in reporting and monitoring practices in our field, to not only enhance efficiency but also effectiveness in project management and financial control.

### Global pandemic – Push for API Access

While API access to concrete suppliers was a longstanding goal, the outbreak of Covid-19 made the implementation of an API solution urgent. The Norwegian Health Authorities published guidelines, in which they suggested keeping all physical contact to a minimum. The physical exchange of concrete delivery slips posed a significant risk of transmission, in which an API solution could solve.

Previously, there was an obvious hesitation to develop an API solution within the concrete branch, with most awaiting a universal API standard. However, the urgency that was spawned by Covid-19 led to a renewed willingness to find immediate solutions.

Ølen Betong was our immediate focus, as they delivered concrete to our biggest project at the time. Within a month of our request, Ølen had published a fully functional API for us. Having a major concrete supplier with an API solution was pivotal in convincing the rest of the suppliers in Norway to publish solutions. As per writing this paper, all three major concrete suppliers in Norway, Ølen Betong, Heidelberg Materials and Unicon, have published fully functional APIs for us.

## DRIV – From Raw Data to Rock-Solid Insight

Tunnel project management involves analyzing and consolidating data from multiple sources. This process can be time-consuming and prone to errors. Once data is analyzed and consolidated, project engineers and managers must invest additional time in assembling a comprehensive report for the builder.

To streamline this process, Skanska developed DRIV, an in-house software created by Skanska Digital in collaboration with their tunneling department. DRIV offers a comprehensive overview of tunneling work, customized for each user's roles and responsibilities. Built on Appfarm, a no-code platform, (Appfarm, 2024), this solution aims to enhance efficiency and accuracy in tunnel project management.

The basis for the solution is to capture all data regarding tunnel production via APIs (Tyson, 2022) (Application Programming Interface) and manual input. All work processes are digitized, and both client and contractor can monitor the progress in real-time with updated construction plans and quantities produced. The solution consists of 4 main modules, Figure 1 shows the dataflow in DRIV.

*The modules:*
1. SETUP is the starting point for all projects. In this module, the projects are established, process codes from the contract are inputted and populated, tunnels, tunnel faces, and site areas. This module manages user rights, project-specific information, and connection to the shotcrete supplier.

2. FIELD is where large amounts of data are collected. The field module is the interface for blue-collar workers and has both offline and online support. This makes it easy to document the work while being in the tunnels with variable wifi coverage.

3. REPORT is the primary user interface for supervisors and production managers. All data registered in the FIELD module is transferred to the REPORT module. This is where the supervisors check and

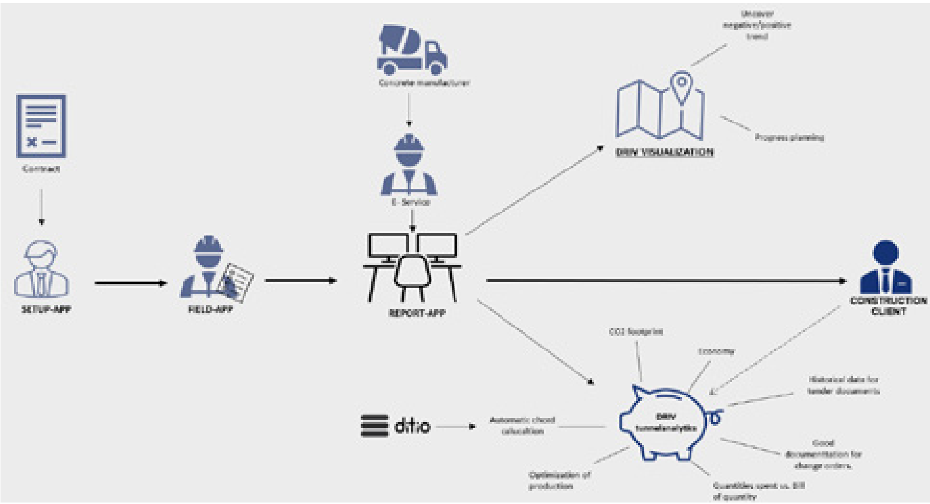


**Figure 1. Dataflow in DRIV from SETUP to construction client**

approve the data, including spray reports from spray operators. The spray report contains accompanying notes from the batching plant, retrieved through an API. When the spray report is approved by the contractor, the concrete operator sees that the report is approved, ensuring a one-to-one match. This allows the concrete operator to invoice based on data that has already been approved by the contractor daily.

The production manager creates quantity reports in this module, listing all produced quantities within a defined timeframe. These reports are then digitally transferred to the client.

4. INSIGHT consists of several different dashboards and apps, gathering for internal and external users allowing a complete overview of progress and activities. The solutions are also connected to execution plans and geolocated data (GIS data). A progress map is rendered based on the FIELD data merged with Skanska's GIS solution. This progress map is of great value to project management for task such as forward planning and monitoring negative and positive trends. With the help of this INSIGHT map, it is possible to see if the individual tunnel face or project is ahead or behind plan in terms of days.

DRIV addresses the challenges of fragmented reporting head-on. It seamlessly consolidates data streams from various sources, providing a comprehensive and single-source dashboard for project progress. It simplifies data-analysis, allows the effortless generation of finalized reports, and provides the insights necessary to be a proactive leader.

Beyond reporting; DRIV goes a step further and streamlines invoicing. By directly integrating contract details and process codes, the solution can pass quality and quantity details directly to the correct process on an invoice, streamlining invoice work and ensuring accuracy.

## Methodology Introduction: The Journey Towards Digitalization

The digitalization process began with a focus on enhancing shotcrete reporting before advancing to develop DRIV, which is a comprehensive "total" system for tunnel reporting. The transition from traditional methods began with a pilot of the "off the shelf" system known as Formworks, however we quickly discovered that Formworks was far too limited for our use. A pivotal visit from Skanska Digital introduced us to the potential of the Microsoft Power Platform, marking the beginning of a successful shift from paper to digital reports.

This digitalization project coincided with the Nordøyvegen and New Skarvbergtunnelen projects, providing a unique opportunity to develop and refine our solution parallel to conventional paper reporting. Initially, the project concentrated on internal documentation, storing digital reports in our database without integrating them into a broader tunneling report system. This phase required field workers to create digital "twins" for their paper reports, a step that, despite its initial burden, was aimed at streamlining the reporting process.

The incredible flexibility of PowerApps was crucial to the success of our digitization, with Skanska Digital promptly and continuously implementing iterative changes based on field feedback. This responsiveness forged a strong sense of ownership and loyalty towards the digital reporting system amongst the field workers. Having field workers that are committed to a new system is an absolutely crucial part of a successful transformation. As the digital system evolved and proved to be more reliable than its paper counterpart, the decision was made to make a full transition to digital reporting. This transition was effectively realized at the start of the "E8 Forberedende arbeider" project.

When DRIV was published, our system was further enhanced by integrating shotcrete data directly from our database into DRIV, with a two-way data exchange between the systems. This integration facilitated seamless data exchange and improved report reliability, marking a significant milestone in our digital transformation journey.

## Detailed Methodology: Systematic Approach and Implementation

Following the digitalization narrative, the methodology of developing and implementing DRIV and ES-Shotcrete involved a systematic engineering process that addressed the challenges of tunnel construction documentation and reporting.

We believe that the best solutions come from those closest to the work processes. For that reason, we adopted the methodology of employee-driven innovation when developing the system. By involving employees from across the organization we were able to tap into unique insights and knowledge, leading to the development of a robust and adaptable system. This approach also fostered a strong sense of ownership among employees, ensuring a successful implementation of the system.

### System Development Process

- **Requirements**: Conducted through interviews and documentation review to understand the nuances of documentation and reporting in tunnel construction.
- **Design Phase**: Utilized software engineering principles to create a blueprint for DRIV and ES-Shotcrete. This phase focused on user interface (UI) design, system architecture, and the integration of modules for shotcrete documentation and general construction reporting.
- **Collaborative design approach**: Employees from all levels of the organization were involved in brainstorming, testing, and were able to directly influence iterative changes to the system through a team of dedicated developers. Thus ensuring the solution serve the needs of everyone from ground-level to project owners.
- **Development and Testing**: Adopted agile methodologies for the iterative development of the systems. This approach allowed for incremental improvements and early detection of issues. Testing was conducted in simulated environments and on pilot projects to ensure reliability and user-friendliness.
- **Feedback Integration**: Feedback from testing phases was used to refine and optimize the systems, ensuring they met the practical needs of tunnel construction projects.

### Data Integration

Data integration was a critical component, ensuring that the systems could seamlessly collect, process, share and analyze data from various sources.

- **Data Sources Identification**: Mapped out all potential data sources, including sensors for monitoring construction progress, machinery operating data, and manual input from field personnel.
- **Integration Framework Development**: Developed a framework using APIs (Application Programming Interfaces) and ETL (Extract, Transform, Load) processes to facilitate the smooth integration of data from these diverse sources.
- **Data Normalization and Storage**: Implemented data normalization techniques to ensure consistency and accuracy. Data storage solutions were designed to handle large volumes of data efficiently while ensuring data security and privacy.

### Real-time Reporting Implementation

The implementation of real-time reporting capabilities was essential for timely decision-making and project management.

- **Real-time Data Capture and Processing**: Integrated IoT (Internet of Things) devices and mobile applications for real-time data capture. Developed algorithms for the immediate processing of this data to generate insights and alerts.

- **Dashboard Development**: Designed interactive dashboards that provide a comprehensive view of project metrics, including real-time progress, resource allocation, and safety indicators. These dashboards were made accessible to project managers, engineers, and stakeholders for immediate access to project status.
- **Alerts and Notifications System**: Implemented an automated alerts system that notifies relevant personnel about critical issues, deviations from the plan, or safety concerns, enabling swift action to mitigate risks.

## RESULTS

The implementation of DRIV and ES-Shotcrete in tunnel construction projects yielded significant improvements across multiple metrics, including but not limited to error reduction, documentation efficiency, project management, and economics. This section presents the findings from multiple projects that utilized these systems.

- **Error Reduction in Reports**: Pre-Implementation, reports were often riddled with errors that usually weren't identified for weeks. Post-Implementation, error rates plummeted, and the few errors that were made were usually identified within hours, thanks to real-time data accuracy and integrated verification processes.
- **Documentation and Efficiency**: Project teams reported a reduction in the time used reporting by at least 80% through automated data-capture and report generation. This also increased accuracy and reduced time spent on revisions. This automation exended to EPD documentation as well, enhancing compliance efficiency.
- **Project Management Enhancements**: Real-time analytics led to an increase in the speed of decision-making while at the same time enabling an optimization of resource allocation while minimizing downtime and cost overruns.
- **Economic Benefits**: The system resulted in substantial cost savings by reducing errors, improving resource efficiency and in sum shortening project timelines. The result was a strong ROI for Skanska, while also optimizing our projects for sub-contractor and suppliers due to the reduction in disruptions in activities and timeline predictability.

**Proactive reports/warning systems/dashboards based on ES Shotcrete data:**

Examples showcasing the use of shotcrete data, including recurring e-mails to field workers, concrete suppliers, and administration, summarizing concrete volumes, quality feedback, logistics notifications, and shotcrete reports to facilitate proactive adjustments and keep track of necessary actions.

- Recurring e-mails to field workers with a summary of the total volume of concrete at the time of the e-mail, which helped workers keep track of when samples/tests of the shotcrete should be taken.
- Recurring e-mails to the concrete suppliers and Skanska with comments about the concrete quality for the past day, thus enabling them to be proactive regarding tuning the mix.
- Recurring e-mails to the concrete suppliers with notifications when we had to wait for concrete delivery so that they could be proactive regarding logistics.
- Recurring e-mails with a list of all the previous days shotcrete reports including the actual report text for the administration in Entreprenørservice.
- PowerBI reports to calculate and expose incorrect arrival frequency of shotcrete loads.

- PowerBI reports to show trends in volumes and classes of shotcrete.

**Proactive reports/warning systems/dashboards based on DRIV data:**

Examples showcasing DRIV data, including data visualization for optimization,

- Automated Co2 reporting based on data from DRIV
- Real-time progress visualization with satellite overlay
- Trend insight by comparing multiple data-sources, enabling the identification of trends that may not be visible otherwise.
- Using historical data from DRIV for a competitive advantage when calculating new projects.

**Future reports**

- Supply chain optimization – A fully digitalized reporting system with real time, and accumulated usage of different materials can be shared with suppliers, enabling them to optimize logistics and reduce tension in the supply-chain
- Machine learning to suggest proper factors when calculating the requirement of different materials, from shotcrete to injection mass.
- Resource allocation optimization – Apply AI to optimize the allocation of resources, including personnel, machinery, and materials based on project needs and timelines.
- Compliance – Automate compliance tracking, with notifications when issues need to be addressed.

## DISCUSSION

The implementation of DRIV with ES-Shotcrete has given significant improvements in error reduction, documentation efficiency, project management practices, and a notable economic impact in our projects. This section takes a look at the significance of these findings, their implications for the construction industry, and considerations for future research.

**Significance and Innovation**
The system represents a significant leap forward in the integration of real-time reporting and data analytics in construction management. The system's ability to dramatically reduce errors and improve documentation efficiency is not just a technical achievement, but a shift in how our projects are managed and executed. The system underscores the critical role of data integration and analytics in achieving operational excellence and sets a new benchmark for accuracy and efficiency in construction projects.

**Industry Transformation**
The field of tunneling is amidst a digital revolution, with DRIV exemplifying the transformative potential of technology. By demonstrating substantial improvements in project management outcomes, these systems illustrate the tangible benefits of adopting digital tools in an industry traditionally characterized by manual processes and slow adoption of technology. The widespread adoption of such systems could lead to:

- **Standardization of Best Practices**: The success of these systems could encourage the development of industry-wide standards for data integration and real-time reporting.
- **Increased Competitiveness**: Firms that adopt these technologies may gain a competitive edge, compelling others to innovate and adopt similar technologies.
- **Sustainability Improvements**: Enhanced efficiency and reduced errors contribute to more sustainable construction practices by minimizing waste and optimizing resource use.

**Limitations and Future Research**
While Systems DRIV and ES Shotcrete have demonstrated significant benefits, it is important to acknowledge limitations and outline potential areas for future research:

- **Adoption Barriers**: The cost and complexity of implementing these systems may pose challenges for smaller firms, suggesting a need for scalable solutions. However, most tunneling contractors have enough economic muscle to be able to allocate adequate resources.
- **Long-Term Impact Assessment**: Future research should focus on the long-term impacts of these systems on project success and organizational performance.
- **Broader Applicability**: Investigate the applicability of Systems DRIV and ES Shotcrete in other construction domains, could provide insights into their versatility and adaptability.
- **Integration with Emerging Technologies**: Exploring the integration of these systems with emerging technologies like AI and IoT could further enhance their capabilities and benefits.
- **Data Sharing - Key to Innovation and Sustainable development**: For continued sustainable development, we are dependent on more open data from suppliers. This will enhance pattern recognition and drive innovation that benefits both the environment and the organization.

## CONCLUSION

This paper introduced the Systems DRIV and ES-Shotcrete, an innovative pair of systems designed to enhance tunnel construction management through advanced system development, data integration, and real-time reporting capabilities. The key post-implementation findings are the demonstrated and substantial reduction in errors, improved documentation efficiency, enhanced project management, and significant economic benefits. Thes results underscore the transformative potential of integrating sophisticated data analytics and real-time monitoring in tunneling projects.

Not only have the systems proven to be effective in eliminating inefficiencies and errors that often plague runnel construction projects but have also laid down the framework for a more predictive and precise approach to management. Their deployment highlights the role of technology in streamlining processes, enabling enhanced decision-making, and ultimately redefining the standards for project management within the industry.

The potential for these systems to change tunnel construction practices cannot be overstated. By showcasing a future where data-driven insights and real-time feedback can be the cornerstone of construction management, these systems advocate for a shift towards a more sustainable, efficient and economically predictable construction practice. Their success in tunnel construction projects serves as a compelling case for the broader application of such technologies across the construction industry.

In conclusion, the implications of the Systems DRIV and ES Shotcrete extend far beyond tunnel construction and reach into the broader industry. The systems represent a leap forward in the journey towards an integrated and technologically advanced construction industry, where efficiency, data integration, and economic predictability are not only thought of as ambitious goals, but as achievable realities. As the construction industry continues to relentlessly evolve, the adoption of digital systems signals a new era of efficiency and innovation, highlighting the critical role of technology in shaping the future of construction management. The positive implications of DRIV for the wider construction industry are profound, signaling a move towards a more informed, efficient, and sustainable approach to construction project management.

# NOTCHED BEAM AND PANEL TEST ALTERNATIVES FOR CONFORMANCE TESTING OF SPRAYED CONCRETE LININGS CONSISTENT WITH LIMIT STATE DESIGN

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**Abstract**: After recent revisions EN 14487, the European Standard for sprayed concrete, and EN 14488, the European Standard for testing sprayed concrete, which covers the determination of various fibre reinforced concrete flexural tensile strength parameters, now include a notched panel test in addition to the EN 14651 notched beam test, consistent with limit state design to *fib* Model Code 2010 and the recently revised EN 1992, Eurocode 2: Design of concrete structures. The notched panel methodology provides an alternative to the notched beam to obtain equivalent residual flexural tensile strengths with several potential advantages including better representation of the in-situ material with reduced variability. A previous spraying trial examining the prerequisite requirement that the alternate notched flexural tests lead to the same Model Code 2010 performance classification, confirmed equivalence for a single hooked end drawn wire fibre type in two sprayed concrete basic mixes. Additional spraying trials extending the work to include a third basic mix design and an alternate hooked end drawn wire fibre geometry are reported. The results for the original fibre type are consistent with the previous trial, confirming the reduced variability of the residual flexural strength parameters in the notched panel test and potentially leading to reduced margins when conformance is based on characteristic strengths, although this advantage may be offset by higher beam strengths. Results for the single basic mix and alternate fibre geometry combination are inconsistent with those from the original fibre. This observation is attributed to a combination of the variability in matrix strength and in-situ fibre content caused by several factors at the trial and illustrates the need for carefully controlled trial conditions. The work nonetheless confirms the potential of the alternate notched panel test methodology as an aid to more efficient sprayed concrete mix design.

**KEYWORDS:**
Fibre reinforced concrete; Residual flexural strength; Performance classification; Embodied carbon; Sustainability.

## INTRODUCTION AND BACKGROUND:

In 2011 EFNARC [1] introduced a notched panel test equivalent to the EN 14651 [2] notched beam method of deriving fibre reinforced concrete (FRC) flexural tensile strength parameters. A 2022 amendment to EN 14487 [3], the European Standard for sprayed concrete, introduced the alternate notched flexural test specimens, and the tests were themselves fully introduced in a 2023 amendment to part 3 of EN 14488 [4], the European Standard for testing sprayed concrete, which covers the determination of various flexural strength parameters.

The introduction of the FRC notched flexural tests to the European standards for sprayed concrete was significant in that it provided a conformance test methodology consistent with limit state design to *fib* model code 2010 (MC2010) [5]. Hitherto these standards had included an energy absorption plate test [6] and a bespoke unnotched beam test [4], both based on the prior EFNARC provisions [7]. While these tests had become well established in empirical design methodologies, neither could be used to derive the fundamental material parameters consistent with the models of FRC behaviour adopted in contemporary limit state design codes.

Historically mined tunnel linings were constructed using an empirically designed primary sprayed concrete lining (SCL), typically treated as temporary or sacrificial, with a permanent cast in place secondary lining. An integrated limit state design and testing approach is fundamental to the goal of realising design and carbon related efficiencies through the adoption of single pass permanent sprayed concrete linings (PSCL) [8]. In 2023 the MC2010 principles of FRC design were subsumed into the revised EN 1992, Eurocode 2: Design of concrete structures in the form of Annex L [9]. This was also significant in that it provides a generic codified methodology in which conformance testing is consistent with the material parameters used in limit state design.

The notched panel methodology is based on the same test specimen as the energy absorption plate test and provides a geometrically analogous alternative to the notched beam, thereby providing equivalent residual flexural tensile strengths. It is advantageous with respect to testing sprayed concrete when obtaining a suitable specimen otherwise involves sawing a beam from a larger test panel. The notched panel test better represents the in-situ material and has several other potential advantages including reduced variability.

A previous project applied Analysis of Variance (ANOVA) methods to the results of prior laboratory-based investigations of cast and sprayed concrete undertaken to support the equivalence of the alternate methodologies [10]. This showed statistically significant differences in the alternate flexural strength parameters, contrary to the prerequisite requirement for consistent MC2010 performance classification. In the subsequent investigation full-scale sprayed concrete pre-construction trials were conducted using three different Bekaert Dramix® steel fibres in both a contemporary CEM I and a CEM IIIB basic mix, which achieves an embodied carbon reduction of 60% [11,12].

In notched panel tests the steel fibres achieved a minimum MC2010 2b performance classification. In the case of the commonly used Dramix® 4D 65/35BG fibre, alternate notched panel and beam tests showed the anticipated higher beam $f_L$ strengths to have a statistically significantly difference. Nonetheless mean residual flexural tensile strength parameters were not significantly different and led to consistent MC2010 ductility classification. Notched panel $f_L$ determination is likely to be conservative and, given the less brittle failure, MC2010 crack control classification is likely to better represent the in-situ material performance.

The improved consistency in a full-scale trial supported the adoption of the alternate notched panel methodology, confirmed the potential to achieve the MC2010 3c performance classification with high performance steel fibres, and demonstrated the potential for more efficient fibre proportioning leading to further carbon savings. Nonetheless, the target C32/40 strength class was not consistently achieved in either mix and the further trials reported here were conducted to help confirm the 3c performance using a C40/50 strength class sprayed concrete. The trials included a comparison of notched panel and beam performance with both Dramix® 4D 65/35BG and 3D 80/30BGP fibres.

When reporting the prior C32/40 trial in-situ fibre content data had only been determined for the Dramix® 4D 65/35BG fibre in the alternate C32/40 concretes. Fibre content data for the other fibre types, including in the additional trial reported here, has since been obtained and is also considered.

**SPRAYING TRIALS:**

The principal aim of the spraying trial was to obtain notched panel and beam test data for Bekaert Dramix® 4D 65/35BG and 3D 80/30BGP fibre types in a nominally identical C40/50 CEM I basic mix. For the two combinations of fibre type and basic mix 6 No. 600 mm x 600 mm x 100 mm, 2 No. 1200 mm x 1200 mm x 150 mm and 1 No. 1200 mm x 600 mm x 150 mm test sample panels were sprayed for the respective preparation of notched panel specimens, notched beam specimens and cores for compressive strength and in-situ fibre content determination.

The C40/50 trial mix design used identical materials and proportions to the previous C32/40 CEM I concrete except for the admixture combination. For the C40/50 CEM I mix a combination of a super plasticising and a 10 kg/m$^3$ addition of a silica fume slurry admixtures was used to enhance the strength relative to the C32/40 CEM I concrete. An identical liquid accelerating admixture was used with the same target dosage of 8% by weight of cement, although this was inadvertently varied below this level to facilitate screeding of the test panels. The Dramix® 4D 65/35BG and 3D 80/30BGP hooked end drawn wire steel fibres were sprayed at a dosage of 40 kg/m$^3$ as in the previous trial.

The spraying trials took place in October 2023 and were otherwise conducted using identical arrangements as for the April/June 2023 C32/40 trial in a purpose-built mock tunnel section at the premises of Shotcrete Services Ltd (SSL) with similar ambient conditions. The same Meyco Suprema sprayed concrete pump with integral accelerating admixture dosing was used with a Wetkret3 spraying manipulator rather than the Oruga manipulator used in the C32/40 trial. Consistent with the previous trial basic mix batching, including fibre addition, was undertaken at the adjacent third-party quality assurance accredited premises of SSL sister company Wealden Concrete.

2 $m^3$ of material was batched for each combination, with fibres manually weighed and dosed directly into the truck mixer hopper. The basic mix was also sampled for cube compression testing and flow tests were conducted. Pumping delivery rates and accelerating admixture dosages were recorded, along with concrete basic mix and ambient temperatures. The hardened panels were covered in wet hessian, left to cure in the mock tunnel overnight and were either cored or rough sawn in the following days to produce the required test sample pieces.

Cube specimens were cured at the batch plant in accordance with standard quality procedures and the relevant panels were cored by SSL to produce the necessary core test sample pieces prior to shipping to Loughborough University where the compressive strength testing work was conducted. Core compression testing was undertaken at the university except for all 24 hr cores and the Dramix® 3D 80/30BGP fibre 3-day cores, which for logistical reasons were tested by an SSL appointed test house and included fibre content determinations. The 1 and 3–day cores were tested in the as received condition with subsequent cores tested in the saturated condition.

The bending test sample panels were shipped to Bekaert's laboratory in Belgium where 28–day notched panel and beam testing, including specimen preparation, was undertaken with the samples air cured in ambient laboratory conditions. 28–day sawn cube compression testing was also undertaken by Bekaert on samples cut from the notched beam test specimens immediately after testing.

**STRENGTH DEVELOPMENT:**

The basic mix cubes were tested in groups of three at 7 and 28 days, returning respective means of 53.5 MPa and 51.4 MPa for the Dramix® 4D 65/35BG concrete and 45.1 MPa and 52.4 MPa for the Dramix® 3D 80/30BGP concrete. These results suggest a minor difference in the strength development of the nominally identical batches without any accelerating admixture, although the aim of producing a C40/50 strength class basic mix sprayed concrete was achieved.

Strength development was assessed by core compression testing in accordance with BS EN 12390-3 [13]. 6 No. cores were tested at 1, 3, 7 and 28 days, with 3 No. cores cut both parallel and perpendicular to the direction of spraying. Cores were ground to achieve a 1:1 length:diameter ratio and density tests undertaken to BS EN 12390-7 [14]. The 1 and 3-day densities were determined in the as-received condition, with all other cores being stored under water after receipt and prior to testing, and density determined in the saturated condition. Nonetheless the 3 and 7-day specimens were received at the laboratory just in time and the prior temperature control was not recorded.

The statistical significance of the difference between the parallel and perpendicular cores was examined by ANOVA. For the 3-day cores in the Dramix® 3D 80/30BGP concrete the variance was so low that, despite similar means, this led to the conclusion that the groups of core results came from different populations. Nonetheless, by inspection confirmed by further ANOVA, there was no significant difference in the means of any of the other subgroups of cores at each age for either fibre type, which were subsequently treated as belonging to the same populations. The data presented in the results below, is therefore based on a statistical analysis of all 6 No. cores tested at each age.

In Figure 1 the core compression strength development with age is plotted along with the C32/40 CEM I and CEM IIIB strength development data from the previous trial. The figure confirms that the strength development of the sprayed material in the C40/50 concretes significantly exceeded that of the C32/30 concrete. It also shows a difference in the strength development of the nominally identical C40/50 concretes, with the Dramix® 3D 80/30BGP concrete having a higher 1-day strength but falling below the Dramix® 4D 65/35BG concrete strength at subsequent ages.

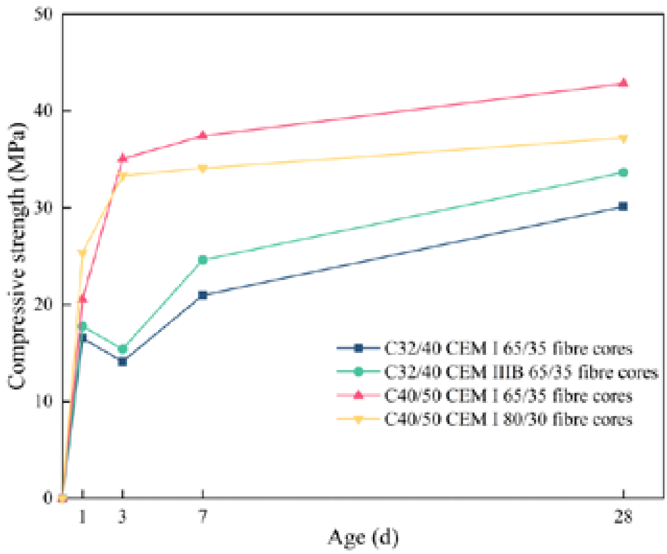


*Figure 1 C40/50 and C32/40 CEM I & CEM IIIB core compressive strength development*

The panels from which the core compression specimens were extracted were sprayed with respective accelerator dosages of 7.5% and 7.0% for the Dramix® 4D 65/35BG and 3D 80/30BGP concretes, although this is unlikely to account for the difference in strength development. The concrete temperature was slightly higher in the case of the Dramix® 3D 80/30BGP concrete, although the ambient temperature was lower. This combination may explain the more rapid early age strength development in the Dramix® 4D 65/35BG concrete followed by lower 3-day strength, although the effect of these differences on the 28-day strength of water cured cores would be expected to be limited.

The test panels from which the notched beams, and therefore the sawn cube samples, were taken were sprayed with respective accelerator dosages of 5% and 3% for the Dramix® 4D 65/35BG and 3D 80/30BGP concretes. The 28-day sawn cube tests returned respective mean results of 42.7 MPa and 45.6 MPa, corroborating the 28-day mean core strength of 42.8 MPa in the case of the Dramix® 4D 65/35BG concrete, although showing a higher value than the mean of 37.2 MPa for the Dramix® 3D 80/30BGP concrete.

The combined evidence with respect to any effect of accelerator dosage on strength development is therefore contradictory. This suggests that, as anticipated, there is no significant effect and the explanation for the difference in the relative 28-day strengths of in-situ cores and cubes is more likely to be inherent differences in the way that the test panels were sprayed and indicating some variation between the core and notched beam test panels.

The strength of the in-situ material samples is lower than that of the basic mix cubes, which is typical and explained by the observation once the heat of hydration has dissipated, and prior to water immersion, the ambient temperature curing regime is lower than that of the water cured cast cubes.

**FIBRE CONTENT AND REBOUND:**

Fibre contents were determined for all 18 No. core specimens tested by the SSL appointed test house comprising 6 No. and 12 No. specimens from the Dramix® 4D 65/35BG and 3D 80/30BGP concretes, respectively returning mean in-situ fibre contents of 21.0 kg/m$^3$ and 28.3 kg/m$^3$. The corresponding respective rebound losses are 48% and 29%. An ANOVA indicated a statistically significant difference in the in-situ fibre contents of the two fibre types with less than a 1% chance of an incorrect judgement. This data suggests there may be a difference in fibre rebound with the alternate fibre geometries in nominally identical sprayed concretes.

For the Dramix® 4D 65/35BG concrete the in-situ fibre content of 21.0 kg/m$^3$ compares with 24.4 kg/m$^3$ and 22.4 kg/m$^3$ for the C32/40 CEM I and CEM IIIB concretes in the previous trial when the corresponding respective fibre losses were 39% and 44%. While these values are broadly comparable, an ANOVA indicates the three in-situ fibre contents obtained for the Dramix® 4D 65/35BG fibre in the three different concretes are from different populations. Nonetheless, differences of this magnitude are unlikely to materially affect residual flexural tensile strength performance.

At the time of reporting the C32/40 spraying trial, the fibre contents discussed above had been determined from cores taken from a single test panel sprayed with each of the 6 No. steel fibre type and concrete combinations. Additional fibre content data was obtained by taking 4 No. cores from selected notched panel test specimens after testing. For each combination the panel with highest and lowest residual flexural tensile strength performance were selected for sampling to give a total of 8 No. core specimens from each combination, which was thereby likely to represent the full range of in-situ fibre contents.

For the Dramix® 4D 65/35BG fibre the in-situ fibre content determined from the C32/40 concrete notched panel specimens was lower than that determined from the corresponding core compression test panel specimens for both C32/40 concretes. The higher rebound in the notched panel specimens may result from their relative thicknesses of 100 mm and 150 mm respectively, since rebound would be expected to reduce as the effect of spraying directly on to the substrate becomes less significant.

For the Dramix® 3D 80/30BGP fibre the in-situ fibre content of 28.3 kg/m$^3$ determined from the core compression samples in the C40/50 trial compares with that of 22.3 kg/m$^3$ from the notched beam specimen sawn cubes in the prior C32/40 trial. This also suggests that rebound is higher in a thinner test panel. While these results are not directly comparable, having been sprayed in different trials and obtained from difference specimen types, a difference of this magnitude would be expected to influence flexural strength performance.

As discussed in the previous reporting [10] these rebound figures appear relatively high compared with those reported elsewhere, although this may simply reflect the relatively thin test panel depth. Nonetheless, while the absolute numbers are relatively high, rebound for the Dramix® 3D 80/30BGP fibre was consistently at least 10% lower than that for the Dramix® 4D 65/35BG fibre.

**FLEXURAL TENSILE STRENGTH:**
Notched panel tests were conducted in groups of six from each of the two C40/50 mix combinations. Eight notched beam test specimens were prepared from each of the relevant test panels for the two mixes. The result of one notched beam test on a Dramix® 4D 65/35BG concrete sample was discarded due to the presence of a crack prior to testing.

The mean, sample standard deviation and coefficient of variability (CoV) were calculated for each group of test results. Characteristic values for flexural tensile strength parameters were also calculated using a log normal distribution and performance classified in accordance with MC2010 consistent with the treatment of the data in the prior reporting [10]. Figures 2 and 3 respectively show the mean and characteristic flexural strength parameters for the Dramix® 4D 65/35BG and 3D 80/30BGP C40/50 concretes.

For the Dramix® 4D 65/35BG concrete (Figure 2) notched panel $f_L$ CoV was 16% and residual tensile flexural strength CoV in the range 12-14%. The corresponding respective values for the beam tests were 8% and in the range 21-24%. ANOVA techniques showed there to be no statistically significant difference between either $f_L$ or $f_{R1}$ respective mean strengths in panel and beam tests with a 5% risk of an incorrect judgement. For the remaining pairs of residual strength parameters there was a significant difference at the same 5% level of risk although, if the level of risk is reduced to 1%, the $f_{R2}$ and $f_{R3}$ mean parameters are not significantly different and the $f_{R4}$ means are only marginally judged to be so.

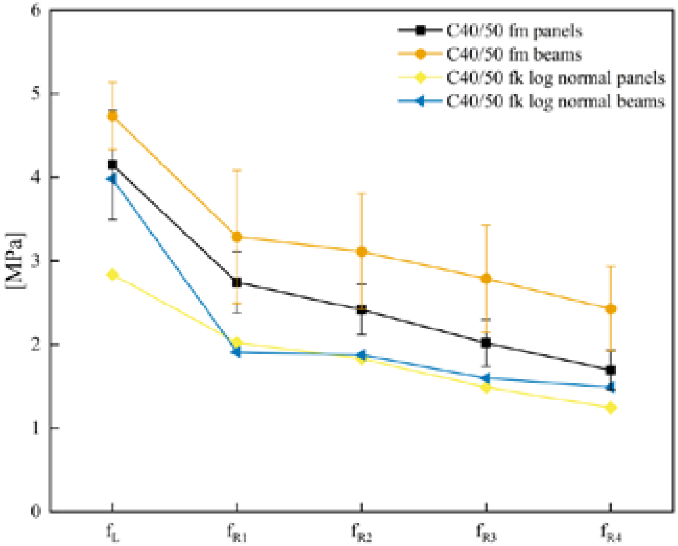


*Figure 2 - C40/50 CEM I Dramix® 4D 65/35BG fibre notched panel and beam mean and characteristic flexural tensile strengths*

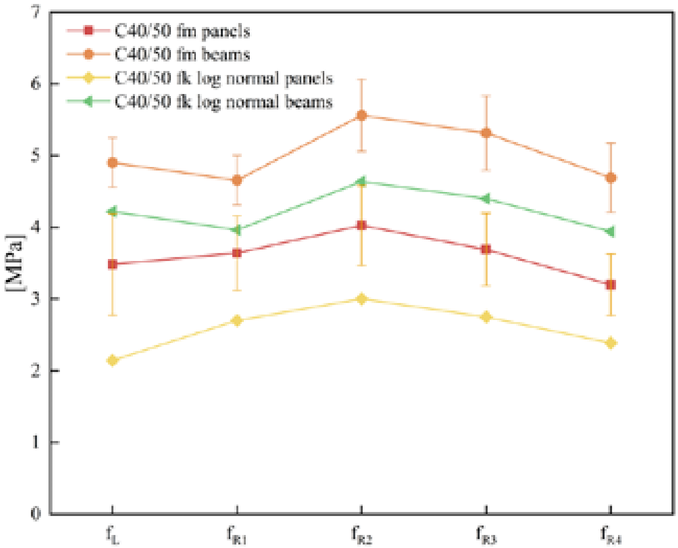


*Figure 3 - C40/50 CEM I Dramix® 3D 80/30BGP fibre notched panel and beam mean and characteristic flexural tensile strengths*

For the C40/50 Dramix® 4D 65/35BG concrete panels this leads to a MC2010 2b class, reflecting both strength and ($0.7 < f_{R3k}/f_{R1k} < 0.9$) ductility, and a $f_{R1k}/f_{Lk}$ crack control value of 0.71. In the corresponding beams the combination of higher mean $f_L$ and residual strength variability leads to a more brittle failure ($f_{R1k}/f_{Lk} = 0.48$). The $f_{R3k}/f_{R1k}$ ductility is slightly higher than that for the panels, thereby maintaining the b level ductility performance, although $f_{R1k}$ reduces to below 2MPa and with it the overall classification to 1.5b. The respective $f_{R1k}$ values fall either side of a residual flexural tensile strength class boundary although this difference in overall performance is insignificant.

For the C40/50 Dramix® 3D 80/30BGP concrete (Figure 3) notched panel $f_L$ CoV was 20% and residual tensile flexural strength CoV in the range 13-14%. The corresponding respective values for the beam tests were 7% and in the range 7-10%, therefore exhibiting a typical reduction in residual strength parameter variation with more ductile fibre concretes. Nonetheless, in this case there is no equivalence of any mean flexural strength parameter, with all five pairs of such values being statistically significantly different with less than a 1% chance of an incorrect judgement. As a result, there is no consistency in MC2010 performance classification with the panels and beams achieving the 2.5c and 3d class respectively. Based on the limited data available, a combination of the higher in-situ fibre content and anticipated better fibre performance compared with the Dramix® 4D 65/35BG fibre leads to relatively improved crack control and ductility performance. However, variations in the concrete matrix strength led to significant differences in strength related performance.

The Dramix® 4D 65/35BG C40/50 concrete trial results are therefore broadly consistent with the C32/40 trial results for the same fibre type where the higher beam test $f_L$ values were explained by the smaller fracture surface and, except for one pair of $f_{R4}$ values there was no statistically significant difference in residual strength parameters. In the prior C32/40 concrete testing the Dramix® 4D 65/35BG fibre also achieved a MC2010 2b performance class ($0.7 < f_{R3k}/f_{R1k} < 0.9$) and a minimum $f_{R1k}/f_{Lk}$ value of 0.95 in notched panel tests with both CEM I and CEM IIIB concretes. In the C32/40 CEM IIIB concrete $f_{R1k}$ also fell marginally below 2MPa leading to a 1.5c classification.

In Figures 4 -7 the mean and characteristic values of the test parameters are respectively shown for the Dramix® 4D 65/35BG and 3D 80/30BGP concretes. For the Dramix® 4D 65/35BG concretes Figures 4 and 5 confirm that, despite a variation in the strength of the concrete matrix illustrated by the variation in $f_L$ values, the post cracking residual strength is characterised consistently. They also illustrate a relatively brittle failure which, despite an increase in the cracking strength in the C40/50 concrete leads to similar post cracking strength performance. Nonetheless the b classification with respect to ductility is consistently met and c class is achieved in one example. This illustrates the potential to achieve the c level performance when fibre retention is at more typical higher levels.

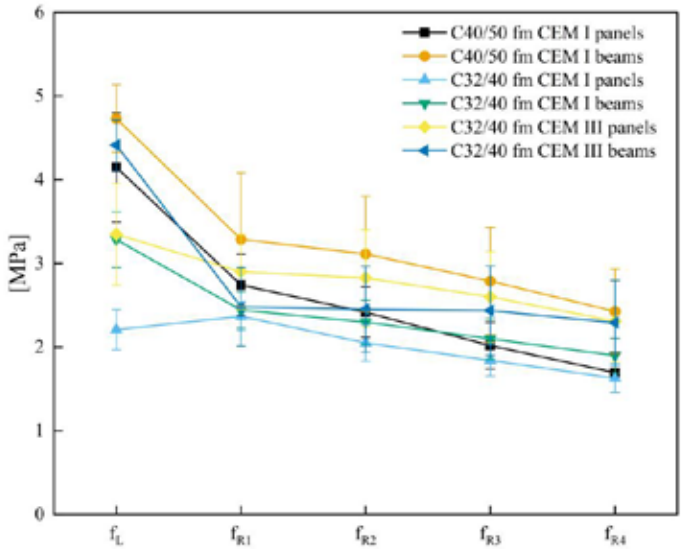


*Figure 4 - C40/50 and C32/40 CEM I & CEM IIIB Dramix® 4D 65/35BG fibre panel and beam mean flexural tensile strengths*

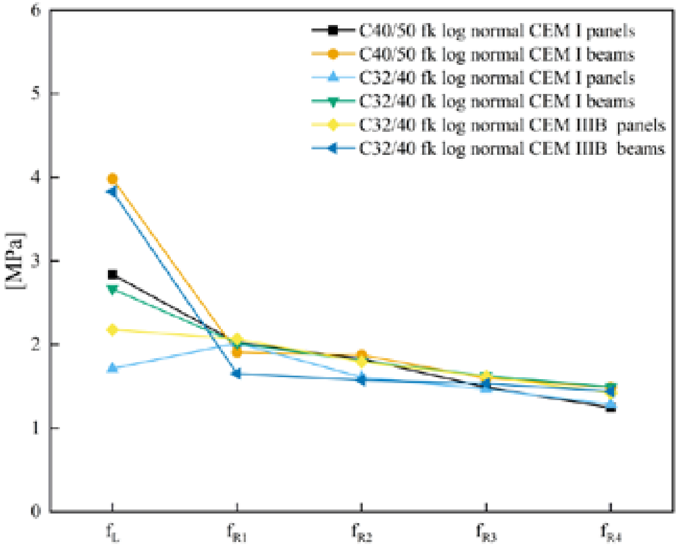


*Figure 5 - C40/50 and C32/40 CEM I & CEM IIIB Dramix® 4D 65/35BG fibre panel and beam characteristic flexural tensile strengths*

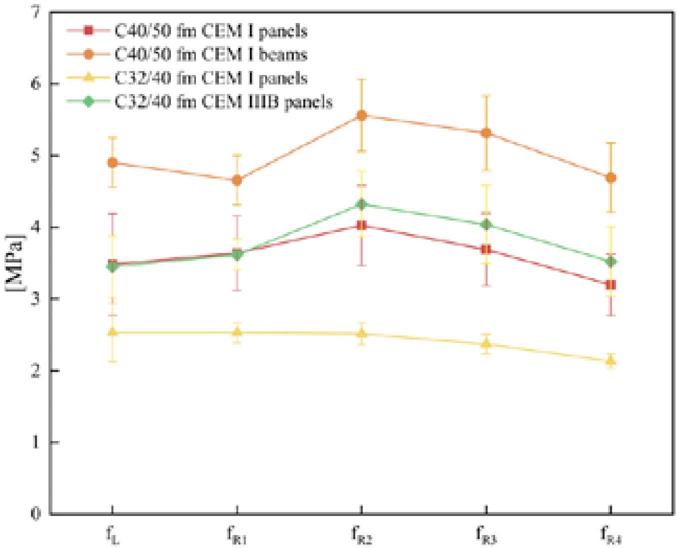


*Figure 6 - C40/50 and C32/40 CEM I & CEM IIIB Dramix® 3D 80/30BGP fibre panel and beam mean flexural tensile strengths*

For the Dramix® 4D 65/35BG there is no consistent trend in the relative panel and beam residual strengths across the three concretes. However, C40/50 concrete mean beam residual strengths are sufficiently higher than the corresponding panel values such that, while the beam values are typically more variable, the characteristic values are much the same. In this event the potential advantage of reduced panel residual strength variability in terms of smaller margins between mean and characteristic strengths is offset by the higher beam values. Nonetheless a consistently less brittle failure occurs in the panel test, and it remains the case that notched panel $f_L$ determination is likely to be conservative and the larger specimen better representative of overall in-situ material performance.

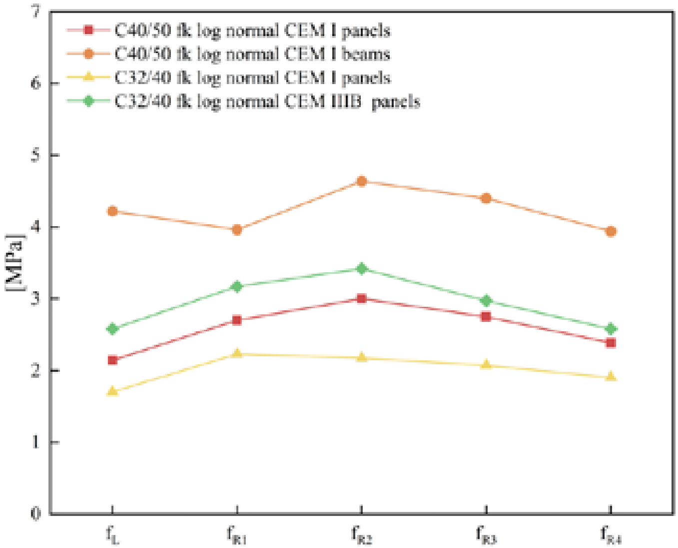


*Figure 5 - C40/50 and C32/40 CEM I & CEM IIIB Dramix® 3D 80/30BGP fibre panel and beam characteristic flexural tensile strengths*

In the preceding C32/40 trial the Dramix® 3D 80/30BGP fibre was used to spray panel test specimens only with no attempt to demonstrate equivalence in beam tests. As illustrated in Figures 6 and 7, the panels achieved MC2010 class 2c and 3c in the CEM I and CEM IIIB concretes respectively and are therefore broadly consistent with the 2.5c class achieved in the C40/50 CEM I concrete. Based on limited in-situ fibre content determination rebound losses appear to have been lower in the C40/50 trial, which may explain the 3d classification in the Dramix® 3D 80/30BGP fibre beam tests. Nonetheless, with all combinations of in-situ fibre content and improved fibre performance the failure is less brittle with $f_{R1k}/f_{Lk} > 1.0$ for all panels and a minimum c class ductility ($0.9 < f_{R3k}/f_{R1k} < 1.1$) is consistently achieved. Variations in concrete strengths lead to inconsistent strength related classifications, although there is clear potential to consistently meet a 3c classification.

**CONCLUSIONS:**

In the second in a series of two full-scale pre-construction trials, Dramix® 4D 65/35BG and 3D 80/30BGP hooked end drawn wire steel fibres were sprayed in a C40/50 basic mix with, across the two trials, the respective fibre concretes achieving minimum MC2010 2b and 2c performance classification in notched panel tests.

For the Dramix® 4D 65/35BG concrete there was no statistically significant difference in the mean $f_L$ or residual tensile flexural strength parameters leading to consistent performance and MC2010 classification. C40/50 Dramix® 4D 65/35BG concrete performance was also consistent with the prior C32/40 trial MC2010 classifications.

In the C40/50 Dramix® 4D 65/35BG concrete the beam residual flexural tensile strengths exceed the panel values to the extent that, despite typically greater variability, characteristic values are similar. Nonetheless a consistently less brittle failure occurs in the panel test, $f_L$ determination is likely to be conservative and the larger specimen better representative of in-situ material performance.

For the Dramix® 3D 80/30BGP concrete there was no equivalence in mean $f_L$ or residual tensile flexural strength parameters between notched panel and beam specimens, which was attributed to variations in fibre content and matrix strength. Notched panel performance was nonetheless broadly consistent across the three concretes.

The Dramix® 3D 80/30BGP fibre typically shows less brittle failure and greater ductility than the Dramix® 4D 65/35BG fibre, which is in part attributed to the anticipated improved performance of the higher aspect ratio fibre, although may also be due to relatively lower rebound with resulting higher in-situ fibre content.

Both fibre types show the potential to achieve MC2010 3c performance classification when concrete mixes and spraying conditions are such that more typical fibre retention is achieved.

**ACKNOWLEDGEMENTS:**
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# GREENHOUSE GAS EMISSION REDUCTION FROM INNOVATIVE SPRAYED CONCRETE

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**Abstract**

Around 55% of the total greenhouse gas (GHG) emissions over a tunnel's lifecycle are related to concrete structures. The lining is the major contributor to GHG emissions due to the demanding use of cement-based concrete. Therefore, the choice and design of tunnel linings is of outmost importance for climate change goals within tunnel construction. The aim of this investigation is to shed light on the potential for GHG emission reduction from the use of a new sprayed concrete technology, as an alternative to existing lining systems. This study was part of the SUPERCON (Sprayed sUstainable PErmanent Robotized CONcrete) research project, which aimed at improving the sprayed concrete technology and ultimately develop, if possible, the design of a new permanent, sustainable and waterproof sprayed concrete with a reduced GHG-emission compared to traditional sprayed concrete. This paper summarizes the results of the last concrete mix designs, emphasizing the environmental aspects from its production and the technical functionality during a full-scale trial as well as laboratory tests. Based on this comprehensive analysis, the mix design that performed as the most "optimal" was selected for further studies on GHG-emissions. A comparison of the total GHG emissions from the selected SUPERCON mix design with the production of traditional tunnel linings will be presented in this paper. The outcome of this comparison demonstrates an outstanding performance of a completely new approach to the sprayed concrete mix design and not least it shows a significant potential to reduce the climate footprint of tunnel linings.

## INTRODUCTION

The production of concrete is one of the biggest contributors of greenhouse gas (GHG) emissions during tunnelling [1][2]. Around 90% of the carbon footprint of concrete is from the production of ordinary cement, assuming clinker as is the only binder material [3]. Despite the different methods, inputs and geological conditions among countries, several studies have pointed out that the construction of tunnels is the most emission intensive phase during its life cycle, followed by maintenance and operation [4][5]. The construction of tunnel lining is the major contributor to carbon emissions due to the demanding use of cement-based concrete [6][7]. For conventional lining systems (i.e., made of cast in-situ or pre-cast concrete) about 55% of the total GHG emissions over tunnel's lifecycle are due to concrete consumption. Therefore, the choice and design of tunnel linings is of outmost importance for climate change. The increasing demand of the construction sector in cements with reduced clinker content and new technologies may face opposition from the construction sector due to a lack of performance data, best practices or reported long-term benefits from innovation. Thus, the aim of this paper is to shed light on the potential for GHG emissions reduction from the use of a new sprayed concrete technology, as an alternative to existing lining systems. This study was part of the SUPERCON (Sprayed sUstainable PErmanent Robotized CONcrete) research project. The aim of SUPERCON was to strengthen the

Norwegian methodology for supporting tunnels as well as to add knowledge on the design of a new permanent, sustainable and waterproof sprayed concrete. The focus was on qualifying sprayed concrete linings and increasing their use by offering a reduction of water transport and cement content. It allowed to optimize the consumption of resources and execution times (by removing the need for a separate waterproofing system), while keeping the required functional properties. New mixes were designed, tested and refined along the project. Several trials were carried in Norway involving laboratories, batching plants and construction sites.

This paper summarizes the results of the final mixes, emphasizing the environmental aspects from its production and the technical functionality during a full-scale trial. The final mixes were designed accounting for the lessons learned from previous tests. First, an estimation of GHG emission from the production of each mix was carried out. The reduction was calculated based on the Global Warming Potential ($CO_2$-eq) of the greenhouse gases involved. It means that the measurement is specifically targeting emissions with consideration for their impact on global warming. The mixes were categorized using a colour scale according to their carbon intensity. Second, the technical functionality of the mixes was examined looking at the workability of fresh concrete and hardened properties of the sprayed concrete. Based on the comprehensive analysis, the mix that performed as the most "optimal" was suggested.
Lastly, a comparison of the total GHG emissions from the production of alternative tunnel linings is presented, including the "optimal" solution developed by SUPERCON project. The outcomes not only demonstrated an outstanding performance of a completely new technology but also a significant potential to reduce the climate footprint of tunnel linings.

## METHOD

### Overview of the new sprayed concrete and field tests

The SUPERCON technology is based on the state-of-the-art of low water-to-binder ratio concrete mixes, and it is characterized by specially designed additives such as, shrinking reducing agent and hydration accelerator. The design of the explored mixes was based on the following approaches:

(i) Optimization of aggregate gradation: the 0-8 mm gradation provide more grain shape and morphology to compensate for variations in aggregates, resulting in a more robust solution for the designed matrix volume (438 l/m$^3$);

(ii) Use of Portland-composite cement (CEM II) with reduced clinker content, compared to conventional Portland cement (CEM I);

(iii) Combination of supplementary cementitious materials (SCMs) such as, limestone, fly ash, and microsilica;

(iv) Combination of additives such as, water reducing admixture (superplasticizer), shrinking reducing agent, hydration accelerator, and air-entraining agent;

(v) Use of dispersible co-polymer powder such as ethylene vinyl acetate (EVA) for improved crack properties; and

(vi) Use of discontinuous and isotropic reinforcement such as, steel fibres. The hooked ends of these fibres ensure a desired concrete ductility and post-crack strength.

Four mixes were analysed from the environmental and technical perspectives, involving a full-scale test at a railway tunnel in southern Norway. The transportation distance from the concrete production plant

to the construction site was around 90 km. The project consisted of one lane tunnel with 125 $m^2$ profile and 3 km length. Each mix was sprayed in an area of approximately 50 $m^2$ over the tunnel walls and arch. A total of 6 $m^3$ of concrete was consumed per mix, including the spraying on tunnel walls and panels.
For the environmental analysis, the GHG emissions from the production of each mixture were calculated based on the kg $CO_2$-eq per cubic meter of sprayed concrete. The analysis follows the ISO 14040 methodology, which is the basis for Environmental Product Declaration (EPD). The sources of GHG intensities were obtained from carbon footprint declarations provided by the product suppliers involved in the project, according to the certified agency "Environmental Product Declaration Norge". The life cycle information accounted for the production stage as the most significant contributor, including cradle-to-gate assessment of raw material extraction (A1), transport to the manufacturing site (A2) and product manufacturing (A3). Based on the estimations, the mixes were classified according to an alphabetical and colour scale, as shown in Table 1.

*Table 1 – Classification of mixes based on the Kg $CO_2$-eq per cubic meter of sprayed concrete*

| Mix A | Mix B | Mix C | Mix D |
|---|---|---|---|
| *Low* | *Medium-Low* | *Medium-High* | *High* |

The analysis of technical functionality includes the properties measured before and after spraying, in addition to other parameters observed during spraying. The slump retention and spread flow of fresh concrete are indicators of workability and consistency. Early strength was measured between 1 and 10 hours with the penetration needle method (Hilti). This method records the force needed to drive a needle 15 mm deep into the sprayed concrete. The strength development is assessed according to the early strength classes (classes J1, J2, J3) defined in EN 14487-1:2022. The compressive strength at 28 days was measured in core samples of sprayed concrete in accordance with EN 12390-3:2009. A subjective assessment of the properties during spraying was conducted based on field observations, video recordings, and experts' analysis. It mainly looked at the pumpability and sprayability of the tested mixes.

The mix that performed as the "optimal" - both in terms of reduced emissions, and desirable workability and hardening properties, was used as basis to compare the total GHG emissions from the production of alternative tunnel linings. Considering the type of excavation method and the water and frost protection, a total of six different linings were compared: (i) cast in-situ concrete; (ii) pre-cast concrete for Tunnel Machine Boring (TBM) excavation; (iii) pre-cast concrete for drill and blast excavation; (iv) combination of pre-cast concrete (walls) and sprayed concrete (arch); (v) sprayed concrete (including two different types of water & frost protection); and (vi) the "optimal" spayed concrete developed by SUPERCON project. The calculations were based in a scenario in which rock support (bolts and sprayed concrete) and secondary lining are needed. The functional unit is defined as one meter Norwegian road tunnel with a functional lifetime between 60 and 100 years (according to the service lifetime of each lining system). A typical Norwegian road tunnel refers to one tube and two lanes with a theoretical blast profile 66.62-$m^2$ cross section. The profile is T9,5 and the Annual Average Daily Traffic (AADT) is < 12000, according to the Public Road Administration`s guidance Vegnormal N500:2022. The system boundaries account for the consumption of materials for the water and frost protection as well as for the final lining.

## Results

### GHG emission estimation

Table 2 summarizes the materials and proportions of the designed mixes. The total GHG emissions per cubic meter of sprayed concrete is presented according to the classification illustrated in Table 1. In addition, the measured properties before (slump and spread flow) and after spraying (early and compressive strength) are also given. The water-to-binder ratio was fixed at 0,42 for all mixes.

*Table 2 – Materials, GHG emissions, and properties*

| Materials [kg/m³] | Mix A | Mix B | Mix C | Mix D |
|---|---|---|---|---|
| CEM II*[1] | 319 | 371 | 450 | 483 |
| Microsilica | 19 | 18 | 18 | 17 |
| Fly ash | 95 | 90 | 0 | 0 |
| Limestone | 161 | 80 | 90 | 32 |
| Superplasticizer | 4,4 | 3,8 | 3,9 | 3,7 |
| Shrinking reducing agent | 2,4 | 2,2 | 2,2 | 2,1 |
| Hydration accelerator | 12 | 11 | 11,2 | 10,6 |
| Air-entraining agent | 0,58 | 0,64 | 0,64 | 0,63 |
| EVA Polymer | 20,3 | 19 | 19 | 18 |
| Water | 179 | 197 | 204 | 216 |
| Sand | 1433 | 1426 | 1430 | 1430 |
| Steel fibre | 40 | 40 | 40 | 40 |
| **GHG emissions (Kg $CO_2$ eq./m³)** | **344** *Low* | **381** *Medium-Low* | **441** *Medium-High* | **465** *High* |
| **Properties before spraying (mm)** | | | | |
| Slump retention | 270 | 280 | 280 | 270 |
| Spread flow | 680 | 750 | 770 | 710 |
| **Properties after spraying - Strength [MPa]** | | | | |
| 1 hour | 0,6 | 0,8 | 0,9 | 0 |
| 3 hours | 3,5 | 1,98 | 2,33 | 0 |
| 6 hours | 8 | 6,2 | 8,1 | 5,8 |
| 9 hours | 11 | 10 | 11,6 | 10,8 |
| 28 days | 27,5 | 36,5 | 37,1 | 43,1 |

*[1] CEM II/A-V Aalborg Rapid FA

Figure 1 illustrates the proportion of materials of the four mixes. For all mixes, the partial replacement of cement was given by the addition of limestone and microsilica. In addition to that fly ash was added, but only to Mix A and B. The dosage of hydration accelerator by volume of concrete also allowed to reduce the cement content, by speeding up the hydration process.

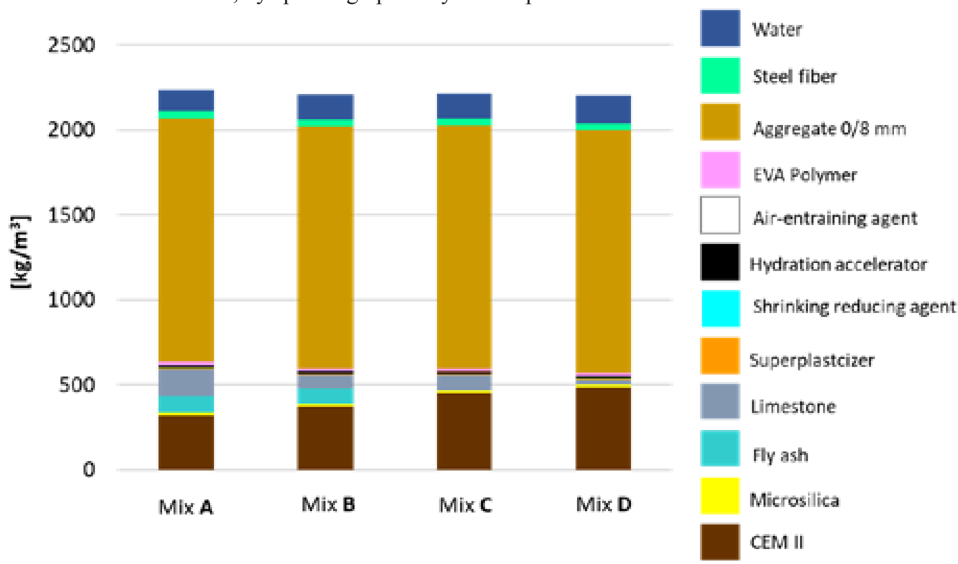



*Figure 1 – Proportion of materials for each mix*

Figure 2 shows the estimation of GHG emissions from the production of mixes according to the classification scale (Table 1). The total GHG emissions per cubic meter of sprayed concrete were: 344 Kg $CO_2$ eq. (Mix A: Low); 381 Kg $CO_2$ eq. (Mix B: Medium-Low); 441 Kg $CO_2$ eq. (Mix C: Medium-High) and 465 Kg $CO_2$ eq. (Mix D: High). The major contributors in all mixes are cement – with a share ranging from 74% to 83%, steel fibres (8% to 10%), EVA polymer (6% to 9%). The hydration accelerator contributes between 2% to 3% and fly ash 1%. The rest of the constituents add less than 0,5%.

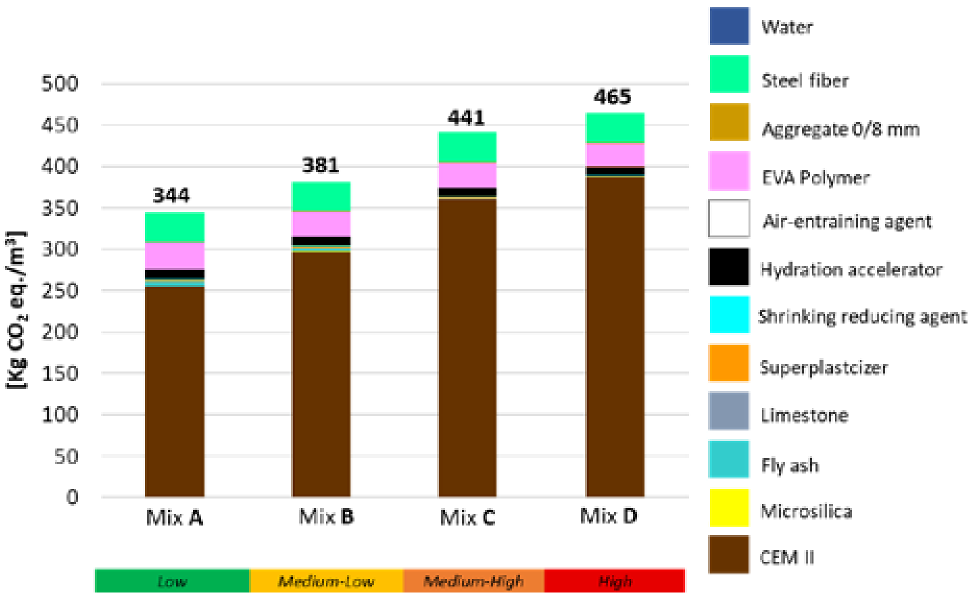



*Figure 2 – Absolute contribution of materials to each mix*

**Workability properties and observed parameters before and during spraying**

Figure 3 presents the properties of fresh concrete measured immediately before spraying. The slump retention was 270 mm for Mix A (low) and Mix D (high), while 280 mm for the medium categories (Mix B and C). Those values are significantly higher than the recommended parameters for wet mix sprayed concrete, established between 180 mm and 220 mm and acceptable up to 240 mm. However, one of SUPERCON's objectives was to achieve such effect - a very liquid concrete which is stable against separation and has a cohesive and "sticky" property. This also enabled a very good compaction on the rock wall during spraying.

The spread flow of Mix A (680 mm) was slightly similar to Mix D (710 mm) despite a significant difference in cement replacement proportions. Mix C had the highest spread (770 mm) followed by Mix B (750 mm). Both mixes share a similar proportion of materials, except of cement (371 kg/m$^3$ Mix B vs. 450 kg/m$^3$ Mix C) and fly ash content (90 kg/m$^3$ Mix B to 0 kg/m$^3$ Mix C).

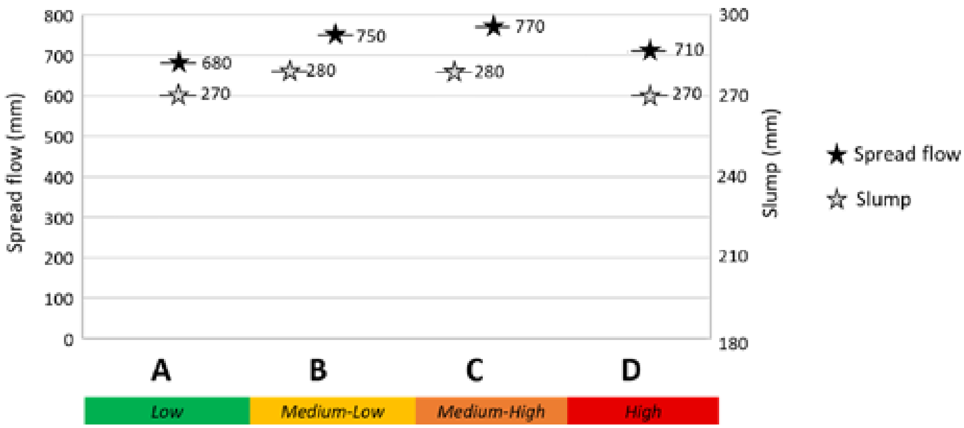
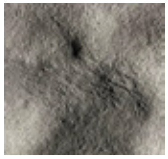



*Figure 3 – Properties measured onsite before spraying*

Besides the use of superplasticizer (which helps to achieve an optimal workability and pumpability), a combination of two other factors contributed to higher plasticity. One is the effect of EVA polymer, which is known for increasing the workability of fresh concrete (Ohama, 1995; Silva and Monteiro, 2016; Khan et al., 2019). It allows to decrease the water requirements and achieve the desirable consistency thus performing as a plasticizer in concrete.
The second factor is related to the addition of hydration accelerator at the arrival onsite due to the extended time frame between batching and spraying (2 hours). The hydration accelerator is suitable for developing fast-hydrating low carbon cements and cementitious materials. These materials have a longer setting behaviour and strength development than ordinary cement and therefore require precise control and measures to optimize construction processes. Before spraying, the hydration accelerator was poured into the mixing truck and revolved for 10 minutes. This approach offered more flexibility in controlling the early cement hydration and, consequently, their influence on setting and hardening of fresh concrete. An assumption is that the logistics strategy contributed to achieve the desired workability and pumpability.

Table 3 summarizes the observed parameters of concrete before and during spraying such as, workability (slump retention and spread flow), pumpability and sprayability. According to the subjective assessment, the four mixes were characterized by a reasonable viscosity/fluidity and cohesion given by a sufficient volume and quality of the paste and a suitable grading curve of aggregates thus resulting in an enduring homogeneous mixture. The mixes were able to pass smoothly through the nozzle. The observations onsite and the analysis of videos led to assume that Mix B and C were better and easily dispersed than Mix A and D, showing a more continuous flow and a more homogeneous blend with the accelerator before the concrete is sprayed and compacted on the substrate.

Table 3 – Observed parameters of concrete before and during spraying

| | Workability, Pumpability and Sprayability | | | | | |
|---|---|---|---|---|---|---|
| **A** |  | **270 mm** |  | **680 mm** |  |  |
| **B** |  | **280 mm** |  | **750 mm** |  |  |
| **C** |  | **280 mm** |  | **770 mm** |  |  |
| **D** |  | **270 mm** |  | **710 mm** |  |  |

Other favourable parameters from Mix B and C were qualitatively observed and examined, such as:

(i) Lower rebound generation and dust development than a standard sprayed concrete. In general, the spray application method may influence the amount of rebound and dust – i.e., depending on the spray rate used or compressed air balance. However, for this innovative sprayed concrete, the designed paste volume, amount for lubrication and optimization of the aggregate gradation contributed very well to the overall material loss and shotcrete quality;

(ii) Strong adhesion to the substrate, both in walls and arch of the tunnel. Although further test may be required, a preliminary deduction is a high load-bearing capacity of the shotcrete lining in combination with the substrate; and

(iii) Good self-compaction on substrate. An experienced nozzleman was crucial for this aspect, by keeping an accurate angle, a reasonable amount of compressed air as well as an optimal distance of the nozzle to the substrate.

## Strength development

Figure 4 presents the measured values of early strength during the first 9 hours, in addition to the compressive strength of cores from sprayed panels after 28 days.

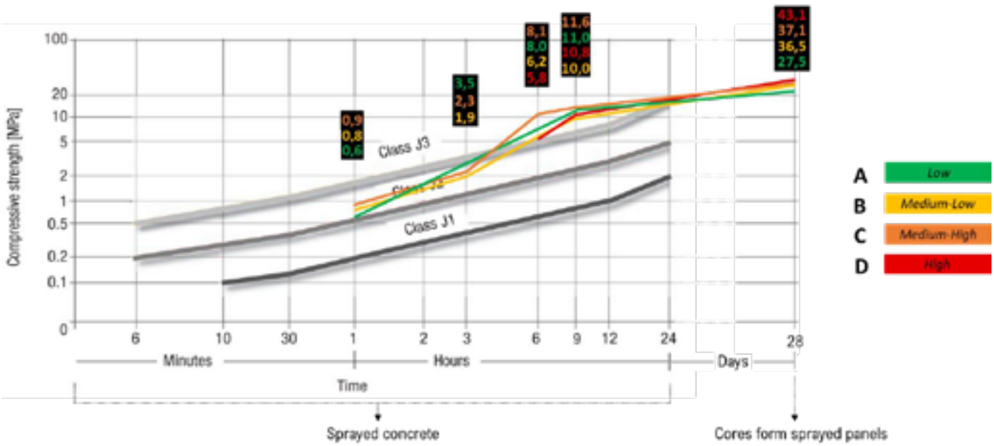



*Figure 4 – Early age strength development measured after spraying*

Mix A, B and C presented very early strength, meeting the requirements of the strength class J2. The three mixes achieved up to 0.6, 0.8 and 0.9 MPa respectively, within the first hour from spraying. Generally, a standard spray concrete present an early strength development between 0.5 and 1 MPa at one hour age after spraying. For Mix D, the penetration needle did not recorded any development before 6 hours after spraying. An assumption could be the lower percentage of hydration accelerator. Subsequent to the initial stiffening period (0 to 2 hours) the early strength of the four mixes had increased continuously between 1 and 9 hours from spraying. In this period, the mixes surpassed class J2 and reached the specifications of class J3. The cores from sprayed panels showed increasing compressive strength values according to the cement content on each mix, such as: 27.5 MPa (Mix A), 36.5 MPa (Mix B), 37.1 MPa (Mix C), and 43.1 MPa (Mix D). Only Mix B, C and D meet the Norwegian requirements for permanent tunnel lining which specifies a minimum compressive strength of 35 MPa at 28 days [8].

In previous trial carried out in a different tunnel (Drammen), a similar mix containing EVA polymer was tested. The proportion of materials of the mix was similar to Mix C (*Medium-High*), but the carbon

impact was 20% lower (361 Kg $CO_2$ eq./$m^3$). If the mixture had to be categorized according to the classification used in Table 1, it would be between Mix A (344 Kg $CO_2$ eq./$m^3$ - *Low*) and Mix B (381 Kg $CO_2$ eq./$m^3$ - *Medium-Low*). The reason for the lower carbon impact can be attributed to the type of cement used such as CEM II/B-M. It contains a lower percentage of clinker and, therefore, lower GHG emissions (0.6 Kg $CO_2$ eq./kg) compared to the cement used for the final mixes CEM II/A-V (0.8 Kg $CO_2$ eq./kg).
The measured spread flow of Drammen mix presented the lowest value (665 mm), similar to Mix A (680 mm). Thus, the sprayability and pumpability tend to show a more discontinuous flow and less homogeneous blend with the accelerator.
The early age strength of Drammen mix presented a very low development at 1 hour (0,15 MPa) compared to the four final mixes (above 0,5 MPa), as shown in Figure 5. It also showed the lowest development between 1 and 3 hours (0,35 MPa), 3 and 6 hours (2,5 MPa), and at 9 hours (7 MPa). Data at 28 days was not available.

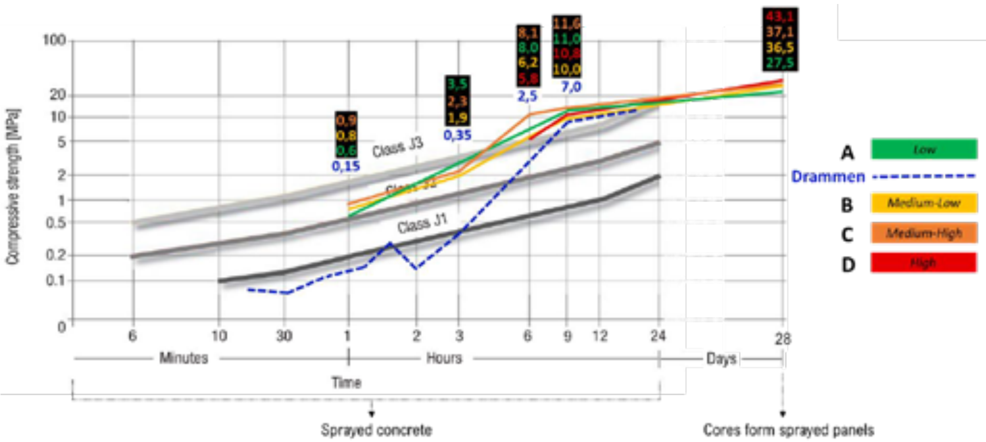
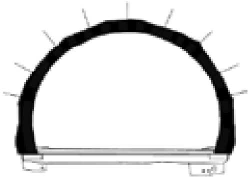
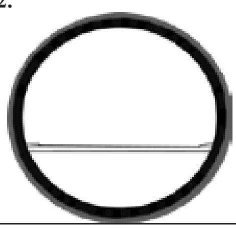
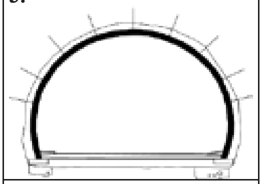
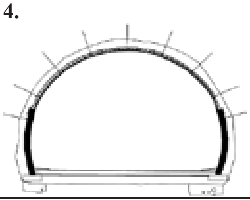
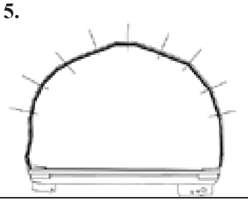
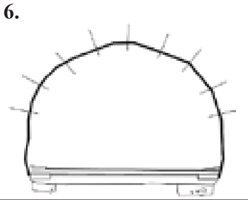



*Figure 5 – Comparison of early age strength development between the four final mixes tested and the Drammen mix*

**Comparison of the total GHG emissions from the production of alternative tunnel linings**

Table 4 compares the GHG emissions per meter tunnel of six different types of linings. The calculations are based in a scenario in which rock support (bolts and sprayed concrete) and secondary lining are needed. The functional unit is defined as one meter Norwegian road tunnel with a functional lifetime between 60 and 100 years. A typical Norwegian road tunnel refers to one tube and two lanes with a theoretical blast profile 66.62-$m^2$ cross section. The system boundaries account for the consumption of materials for each lining system.
By combining the results of Section 3.1, 3.2 and 3.3, the most efficient sprayed concrete chosen for this comparison is Mix B, classified with a Medium-Low carbon impact (381 Kg $CO_2$-eq/$m^3$ concrete). Mix B also presented a suitable workability and the most desirable pumpability and sprayability (according to field observations and subjective analysis). The early strength development between the first and 9 hours after spraying, met the requirements of class J2 – which is considered suitable, e.g. in case of slight water afflux and immediate subsequent work steps like drilling and blasting according to EN 14487-1:2022. The compressive strength at 28 days showed an acceptable development (36,5 MPa) given the reduced proportion of cement, when compared to a standard sprayed concrete (< 20%).

*Table 4 – Comparison of the total GHG emissions from the production of six different lining systems with the thicknesses of the different parts of the lining*

| | Type | Excavation method and area | Water and frost protection | Final lining | Expected lifetime (years) | GHG emissions (Kg $CO_2$-eq/meter tunnel) |
|---|---|---|---|---|---|---|
| *Continuous to rock* | **1.**  | Drill & Blasting 67 $m^2$ | Drainage layer and sheet membrane (sprayed concrete thickness 154 mm) | Cast in-place concrete (600 mm) | 100 - 120 | 27000 |
| | **2.**  | Tunnel boring machine (TBM) 67 $m^2$ | Pre-cast concrete segments (400 mm) | | > 120 | 23000 |
| *Discontinuous to rock* | **3.**  | Drill & Blasting 67 $m^2$ | XPS and sheet membrane (54 mm) | Pre-cast concrete elements (200 mm) | > 60 | 2800 |
| | **4.**  | | Arch: Steel reinforcing mesh + PE-foam (45 mm)<br>Walls: XPS (50 mm) | Arch: Sprayed concrete (85 mm)<br>Wall: Pre-cast concrete (200 mm) | > 60 | 1955 |
| *Continuous to rock* | **5.**  | Drill & Blasting 54 $m^2$ | a. Steel reinforcing mesh + PE-foam (45 mm) | Sprayed concrete (100 mm) | > 60 | 915 |
| | | | b. Polymer-based sprayed membrane (30 mm) | | > 60 - 80 | 650 |
| | **6.**  | | New sprayed concrete (80 mm)<br>Mix **B** - *Medium-Low impact* | | > 100 | 579 |

When comparing the overall GHG emissions from the production of one meter lining over their expected lifetime, the lower contribution is given by the tunnel executed with the new sprayed concrete (Type 6). Type 1 (cast in-place concrete) is the most emission intensive lining system accounting for

27000 Kg $CO_2$-eq/meter tunnel. Traditionally, pre-cast concrete segments (Type 2) are used when the Tunnel Boring Machine (TBM) is adopted as excavation method. This lining type is the second major contributor to GHG emissions (23000 Kg $CO_2$-eq/meter tunnel).
The pre-cast concrete elements of Type 3 have reduced concrete thickness than Type 2 and, therefore, emissions are decreased by 88%. Although Type 3 has a shorter service lifetime (60 years) than Type 2 (120 years), the significant reduction in lining emissions should not lead to assume that one excavation methods is preferable than the other. Rather the focus should be in a solution that allows a lower consumption of cement without compromising the structural and functional properties. Type 4 and 5a enable a further reduction in GHG emissions. However, the use of PE-foam requires a reinforcement before the application of the fire protection (sprayed concrete) which has a service lifetime of 60 years. Although the lifetime of the fire protection is expected to be longer, the maintenance of the reinforcement after 60 years leads to waste maximum potential of sprayed concrete (as the existing layer must be removed and a new one is then applied).

The new sprayed concrete (Type 6) can decrease the emissions between 98% to 70% when compared to cast in-place and pre-cast concrete solutions. If compared with a standard sprayed concrete lining the emissions can be reduced between 36% and 11% per meter tunnel, depending on chosen waterproofing solution (i.e., PE-foam or sprayable membrane). Type 6 also results in a reduction between 90% to 40% in the overall lining thickness, if compared with the other types. This also means a 20% reduction in the excavated volume of the tunnel when compared with Type 1, 2, 3 and 4. Moreover, the environmental impact is further reduced when compared to execution processes. From type 1 to 5, the construction of the lining consists in two phases: one for the execution of waterproof and frost protection and another for the final cladding (whether for protection against fire or for aesthetics purposes). Type 6 is executed in one step thus omitting the need for a separate waterproofing inner shield structure, or a cast-in-place concrete cladding. It means that operational times, critical processes, transport and energy consumption can be optimized. The robotized application of this new technology takes the same execution time as any other sprayed concrete lining (6 $m^3$/30 min.), and the expected lifetime is the same as cast-in-place or pre-cast concrete systems (> 100 years).

**Discussions**

This study demonstrated that the new sprayed concrete could contribute to a significantly lower carbon footprint, compared to the construction of existing tunnel linings. The analysis of technical and functional properties before, during and after spraying also indicated an acceptable performance with desirable outcomes. The preliminary results described in this paper may be useful for decision making during the early phases of an underground project. The intention is to provide practical support for optimizing tunnel emissions from a design perspective with today's solutions. The productivity of the new sprayed concrete is also higer than the existing alternatives. Regardless the excavation method, typical linings consists of multiple execution processes (i.e., waterproof and frost protection in addtion to fire protection or final cladding). The new sprayed concrete was desiged to be executed in a single process, continunous with rock. This optimization not only indicate a potential for GHG emission reduction from material consumption but also from energy use – i.e., less fossil diesel for transportation and machinery uses are needed during the lining construction. The reduction of lining thickness and excavated area also results in shorter construction times, which is often seen as a main cost driver in any construction project.

For all mixes, the major contributors of GHG emissions are cement, EVA polymer and steel fibres. Mix B was selected as the most efficient sprayed concrete according to the combined analysis of environmental and technical performance. The GHG emissions from its production are slightly higher (381 Kg $CO_2$ eq./$m^3$) than an industry reference mix (366 Kg $CO_2$ eq./$m^3$), despite the reduced cement content (371 Kg/$m^3$ Mix B vs. 475 Kg/$m^3$ industry reference mix).
Although the rebound of the tested mixes was not technically measured, the field observations led to assume a significantly lower rebound than expected. Previous studies demonstrated that the mixtures with SCMs had lower rebound values than industrial reference mixtures thus resulting in less material

waste [8][9] . Particularly, the addition of microsilica can make the concrete more fluid when agitated and more viscous when left undisturbed while fly ash may hinder the pumpability and shootability of shotcretes. Also, different dosages of additives such as, alkali-free accelerator, water reducing agent and polymer has been explored for wet-mixes which resulted in a reduction of rebound rate [11].
The increasing use of SCMs tend to delay the cement hydration and compressive strength development [12] thus requiring an acceleration of SCMs reaction to obtain an acceptable rate of performance. By doing so, the fresh-state behaviour of concrete can be affected under extended time between batching and spraying due to delays in schedule or due to long transport distances. As described in Section 3.2, the chosen approach to ensure the fresh properties of concrete was to add the hydration accelerator on site. It allowed a greater flexibility and control over processing and placing of concrete. This approach is similar to the "set-on-demand" process described in the field of smart dynamic casting or 3D concrete printing. As concrete is executed in highly automated processes it requires precise control of the material setting and hardening, to which the application of accelerators offers a suitable solution [13]. It means that the tunnelling sector can further benefit from the way these technologies are used and managed.

Previous investigations have explored novel combinations of waterproof spayed concrete for tunnel linings with positive preliminary results along the combination of steel fibres, polymers [14] and admixtures [15]. The waterproofing sprayed concrete developed in SUPERCON addressed a holistic approach to the effect which cause water seepage through concrete in low seepage areas. Also, several recipes with different contents of cement were tested to obtain an "optimal" mixture that meet the desired technical functionality and offer a reduction of GHG emissions from lining construction and maintenance, compared to traditional lining systems.
The proposed sprayed concrete has increased filler content compared to today's average. This was technically feasible by combining particle size control and different types of admixtures and specially designed additives dosages, resulting in concrete with low water demand. For the "optimal" sprayed concrete (Mix B), the contents of limestone (14% by the total volume of binder) and EVA polymer (3% by the total volume of binder) demonstrated positive effects on workability, shrinkage, mechanical properties and water permeability in accordance with the findings of several studies [16][17][18][19]. Limestone contributed 0,20% to the overall GHG emissions per $m^3$. Limestone is an abundant reserve on earth with very low embedded $CO_2$ emission and costs, compared to ordinary cement. In some applications, limestone contents up to 50% in concrete can offer satisfactory performance [20]. EVA polymer consists of two chemicals (ethylene and vinyl acetate) and contributed 7,9% to the overall GHG emissions per $m^3$. It is the third carbon intensive contributor after cement (78%) and steel fibres (9,2%). Yet, the global warming potential from its production can be reduced by 40% if the fossil-based ethylene is substituted with bio-based ethylene [21].

## Conclusions

The preliminary results of this research have demonstrated that the most efficient sprayed concrete (Mix B) had a positive influence on wet concrete properties, early strength development and water transport reduction one week after spraying. The field observations indicated an acceptable ability to retain moisture transfer and reduce water drainage from isolated dripping. Yet, the ability to retain frequent leakage should be further assessed. Future investigations should also explore:

(i) the combinations of binders and additives with CEM III (lower embodied $CO_2$ than CEM II);
(ii) the optimization of steel fibres consumption towards a reduction of GHG emissions;
(iii) the use of EVA Polymer based on bio-ethylene given the lower embodied $CO_2$ than ethylene-based EVA polymer;
(iv) other dimensions of life cycle thinking such as costs and social implications. It may offer a more comprehensive perspective for tunnels owner, contractors and producers as well as for the implementation of a new solution into the market.

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THE USE OF HARDENING ACCELERATOR ADMIXTURE IN FRESH SPRAYED CONCRETE TO PROMOTE COMPRESSIVE STRENGTH DEVELOPMENT AT THE EARLY AGE OF SPRAYED LOW CARBON CONCRETE

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## ABSTRACT

The use of low carbon concrete is becoming more in focus in today's society. All types of construction industry have the aim to use concrete with lower CO2-footprint. For spayed concrete however, the use of low carbon concrete with significantly less Portland clinker, can be a huge challenge. The early strength development in sprayed concrete is a result of chemical reactions between Portland clinker, water and spraying accelerator. By lower the Portland clinker by as much as 50 kg/m3, the CO2-footprint can easily be reduced by 35- 40 kg CO2/m3. The cement reduction will reduce the amount of paste in the fresh concrete and cement must be replaced by something else, like slag or fly ash. Sprayed concrete accelerator does not react with either fly ash or slag, so to increase the reactivity of the fresh concrete, a hardening accelerating admixture is introduced in order to compensate for some of the early strength development.

## BACKGROUND

United Nation's Sustainability goals, and especially no. 13 says it is a goal; to act immediately in order stop climate changes and the consequences of them. The goal is to limit the global warming to below 2 °C. [1]. The building and construction industry must take one of the leading roles in achieving the reduction in climate gas emissions, as the manufacturing process of Portland cement is contributing to ~8 % of the global CO2-emissions [2]. Reducing cement content only is not easy, as the water/binder-ratio (w/b) in modern concrete quality is a critical factor. If/when cement is reduced, the water must be reduced so that the w/b will continue to not exceed a defined limit. A concrete's flow, rheology and pump-ability are all strictly related to the content of cement paste and matrix in the fresh concrete. So in this study, the cement content of a sprayed concrete mix design is reduced, and at the same time fly ash is dosed into the mix design to compensate for the cement which has been taken out. Fly ash is available as a raw material for concrete production in Norway, and Heidelberg Materials is offering fly ash for commercial sales. Heidelberg Materials has approval for using factor = 1.0 for fly ash under certain conditions [3]. This means that in Norwegian sprayed concrete with w/b of 0.45 or 0.40, cement can be substituted with fly ash up to a maximum of 35 %, when Standardsement FA, CEM II/B-M (V-L) made in Norway is used [3].

An alkali-free accelerator (AFA) for sprayed concrete has a high reactivity with Portland cement and not with silica fume or fly ash. When Portland cement is replaced with fly ash, the sprayed concrete's ability to react with AFA is suspected to become less. In order to compensate for some of the reduction in Portland cement content in mix design, strength enhancing concrete admixtures can be used as raw materials in the sprayed concrete's mix design. Such strength enhancers can be either a solution of chemicals in water or a suspension of small nano-particles in water. Such strength enhancing admixtures are mainly developed to be used in ordinary cast concrete, but in this study the effect in sprayed concrete is investigated.

REDUCTION IN CO2-FOOTPRINT FROM CONCRETE MIX DESIGN

In Norway, there are only one producer of cement and this cement producer has two cement plants. The high volume product is Standardsement FA (Std FA) which is a CEM II/B-M (V-L) 42,5 R. This cement has ~18 % fly ash and 6 % limestone. Total content of supplementary cementitious materials (SCMs) are therefore ~24 %, which leaves the remaining part of 76 % to be clinker + gypsum. According to the Environmantal Product Declaration (EPD) for Std FA, the Global Warming Potential ($GWP_{total}$) = 568 kg CO2/ton cement [4] for the cement produced in Brevik (south), while the GWPtotal = 613 kg CO2/ton cement [5] for the cement produced in Kjøpsvik (north). Two strength enhancing concrete admixtures have been tested, and the supplier (Mapei) has Product Specific EPD for both products:

Strength enhancer (solution): 0.81 kg CO2/kg admixture [6]

Strength enhancer (suspension): 0.47 kg CO2/kg admixture [7]

Table-1 When removing 50 or 100 kg of cement in the mix design, the CO2-footprint will be reduced with:

| **Effect of Reduction in cement + addition of fly ash:** | **DGWP**<br>**-50 kg cement + 50 kg fly ash** | **DGWP**<br>**-100 kg cement + 100 kg fly ash** |
|---|---|---|
| **CEM II/B-M Brevik (south):** | -28.4 + 0.7 = -27.7 | -56.8 + 1.4 = -55.4 |
| **CEM II/B-M Kjøpsvik (north):** | -30.7 + 0.7 = -30.0 | -61.3 + 1.4 = -59.9 |

Notice: It has not been successful to obtain a documented value of the CO2-footprint from Fly Ash, but the value used is a just an estimated value which has been provided by R&D in Heidelberg Materials in Norway. The source of this value is therefore not referred to in the reference list. This is the "core of the challenge" with collecting data for $GWP_{total}$.

A sprayed concrete mix design with 450 kg/m3 of cement has a $GWP_{total}$ coming from the cement itself of 255.6 kg $CO_2/m^3$ when cement from Brevik is used and 275.9 kg $CO_2/m^3$ when cement from Kjøpsvik is used. The reduction obtained when substituting 50 or 100 kg cement is therefore significant on the total $CO_2$.

## TEST SETUP

In this study the positive improvements are expressed by the use of different hardening accelerators for concrete. The test is done using mortar prisms to simplify the test. The study is focusing on showing that use of hardening accelerators can improve the compressive strength also of a low carbon concrete if the right dose of accelerator is used. This is shown by 5 different mortar formulas with 3 different amounts of cement.

Table-2 shows the mix design of samples in this study

| ID | Cement | Fly Ash | Water | Sand | SP |
|---|---|---|---|---|---|
| **Reference (Ref)** | 450 | 0 | 202.5 | 1350 | 3.15 |
| **Low Carbon 1 (LC1)** | 400 | 50 | 202.5 | 1350 | 3.15 |
| **Low Carbon 2 (LC2)** | 350 | 100 | 202.5 | 1350 | 3.15 |

It is a reduction by -11% cement (LC1) and -22% cement (LC2). In addition to the 5 formulas of low carbon mortar, and reference sample of 100 percent cement. All mortar-samples are done in a lab with a climate of 50 % RH and 22°C. All the mortars are made of plastic mortar with a standard sand in order to EN-196 (1350g) with the most normal cement at the Norwegian market (CEMII/B-M 42.5R). The molds used in the study are steel molds 160 mm x 40 mm x 40 mm. All tests are done with W/C = 0.45 and with 0.7% superplasticizer. The accelerators that were used was (Es) (2-3%) for early strength based on a suspension of nanoparticles and (Ls) (0.5%) which is a solution and a late strength accelerator to develop strength later than 24 hours. In addition, some of the samples were prepared with a suspension of Alkali free accelerator (AFA) to simulate the effect of interaction with shotcrete accelerator. In these trials it was used a suspension of AFA based on aluminum sulfate. The dosage of the accelerator was 7% of the binder. The mortar was finished by using the mixing method EN 196. The AFA was dosed from a small container and added in the mixing bowl. The mortar was mixed for 5 seconds at slow speed to be sure that all the accelerator was mixed in the mortar and not splashing out of the control auto mixer bowl. Further the control mixer was speeded up for 15 seconds. The molds were then filled half full and tapped with a steel jolter before the next level. The prism was completely full and tapped before a jolting table with 60 jolts. The molds were scraped with a (steel float) so the surfaces were flat. During the first 24 hours the molds were cured at 95 % RH in a chamber. After 24 hours the prisms were stored in water. The prism was dried with a paper and weighed of each prism was controlled, to understand if the mortar had acceptable density. The prism was tested after 2, 4, 6, 8 and 24 hours and after 7 and 28 days. For the long-time compressive strength test prism with AFA, the mix was prepared with cold water. This makes the immediate set a bit slower and gives a more optimal cast process and density.

**Test equipment**

The mortar was mixed with Controls auto mixer, following the EN-196 standard. [8]

Figure 1 The Controls auto mixer, used for the mix of plastic mortar.

The measurement of early strength was tested by a IMADA penetrometer, with a 7 mm vide needle. The timing 0 starts after the molds are prepared.
The first test by Imada penetrometer was done after 15 min from mixing. Tests with a penetrometer is correlated to operator because of this the tests were all done by the same person. This is only tested for all compressive strengths lower than 0.5 MPa. For this reason, it is just used when prisms with AFA are tested.when 0.5 MPa was reached, a mortar compressive strength machine was used to measure the future strength development.
The mortar prisms were then stored in water in a lab with a 22 °C temperature.


Figure 2 showing Imada penetrometer, the needle for Imada for compression <0.5MPa and the compressive strength machine.
for > 0.5 MPa.

**RESULTS**

The results from the tests are expressed in tables below. The tables below show a reference sample. 1.Ref of a standard plastic mortar 100% (CEM II). The 1.REF consists of 450 g cement. The reference sample 1.1 has a reduction of 22 % cement, and to compensate the reduced cement the sample is added Fly-Ash. There is no hardening accelerator in ref 1.1 and cement is replaced with fly ash1:1 of cement in this formula. Sample 2, 3, 4, and 5 has different dosage of different hardening accelerator Es (Early- strength) and Ls (Late- strength). The values from 2-24 hours are measured only with formulas with 7 % (AFA). The prisms that are measured on day 7 and day 28 are developed separately, to improve the compaction of the prism for long term storage. All samples were checked for porosity and weighed before measuring compressive strength, the weight of prism were for all samples 598-620 g per prism and this indicates that the prism has got an acceptable density.

Table 3- Shows the effect of hardening accelerator in a mortar LC2, all with 7 % AFA in addition, from 2 to 8h:

| **-22% CEM II with AFA** | **2 h** | **4 h** | **6 h** | **8 h** | **24 h** | **7 days** | **28 days** |
|---|---|---|---|---|---|---|---|
| **1.Ref 450 g** | 0.1 | 0.5 | 1.8 | 3.8 | 25.1 | 42.5 | 54.2 |
| **1.1 Ref** | 0.2 | 0.5 | 1.8 | 2.9 | 14.4 | 31.0 | 40.7 |
| **2 2% ES** | 0.2 | 0.7 | 2.5 | 3.7 | 15.1 | 31.7 | 41.9 |
| **3 3% ES** | 0.2 | 1.2 | 2.5 | 4.7 | 16.1 | 29.3 | 40.7 |
| **4 2%ES 0,5% LS** | 0.2 | 0.8 | 1.9 | 4.3 | 17.3 | 33.3 | 43.1 |
| **5 3%ES 0,5% LS** | 0.2 | 1.1 | 3.9 | 4.5 | 17.4 | 32.3 | 43.5 |

Table 3. Shows the effect of hardening accelerator in a mortar with a low dosage of cement (-22% cement).
Table 4- Shows the effect of hardening accelerator in a mortar with LC1, all with 7 % AFA in addition, from 15 min to 8h:

| -11% CEM II with AFA | 0.25 H | 0.5 H | 1 H | 2 H | 4 h | 6 h | 8 h |
|---|---|---|---|---|---|---|---|
| 1.Ref 450 g | 0.02 | 0.03 | 0.06 | 0.09 | 0.54 | 2.95 | 3.79 |
| 1.1 Ref | 0.04 | 0.08 | 0.13 | 0.40 | 0.75 | 2.88 | 3.37 |
| 2 2% ES | 0.05 | 0.08 | 0.13 | 0.20 | 0.78 | 3.42 | 4.22 |
| 3 3% ES | 0.06 | 0.08 | 0.13 | 0.21 | 0.7 | 3.72 | 4.52 |
| 4 2% ES 0,5% LS | 0.02 | 0.03 | 0.04 | 0.15 | 0.43 | 1.62 | 3.15 |
| 5 3% ES 0,5% LS | 0.03 | 0.04 | 0.10 | 0.18 | 0.77 | 2.17 | 5.47 |

Table 4 Shows the effect of hardening accelerator in a mortar with -11 % cement.

Table 5- shows results with LC1 from 24h up to 28 days, with AFA.

| 7% AFA -11% CEM II | 24 h | 7 days | 28 days |
|---|---|---|---|
| 1.Ref 450 g | 25.1 | 42.5 | 54.2 |
| 1.1 LC1 Ref | 17.5 | 36.5 | 46.4 |
| 2 2% ES | 19.2 | 36.7 | 46.1 |
| 3 3% ES | 20.0 | 35.9 | 46.3 |
| 4 2%ES 0,5% LS | 21.3 | 39.7 | 48.5 |
| 5 3%ES 0,5% LS | 21.5 | 37.8 | 47.4 |

In table 6 the tests are repeated on plastic mortar without AFA.

Table 6- shows results with LC2 from 24h up to 28 days.

| -22% CEM II | | | |
|---|---|---|---|
| -22% CEM II | 24 H | 7 D | 28 D |
| 1.Ref 450 g | 23.3 | 40.9 | 54.8 |
| 1.1 LC2 Ref | 13.0 | 32.0 | 42.3 |
| 2 2% ES | 17.0 | 31.7 | 44.7 |
| 3 3% ES | 16.6 | 31.6 | 44.1 |
| 4 2%ES 0.5% LS | 18.5 | 34.3 | 47.5 |
| 5 3%ES 0.5% LS | 20.1 | 35.3 | 49.1 |

Table 7- shows LC1 with different dosage of accelerators without AFA.

| -11% Cem II | | | |
|---|---|---|---|
| | **24 H** | **7 D** | **28 D** |
| **1. Ref 450 g CEM II** | 23.3 | 40.9 | 54.8 |
| **1.1 LC1 Ref** | 18.3 | 37.5 | 48.3 |
| **2 2% ES** | 21.7 | 38.3 | 51.3 |
| **3 3% ES** | 22.6 | 39.7 | 50.5 |
| **4 2%ES 0,5% LS** | 23.1 | 42.7 | 55.2 |
| **5 3%ES 0,5% LS** | 24.2 | 43.2 | 55.0 |

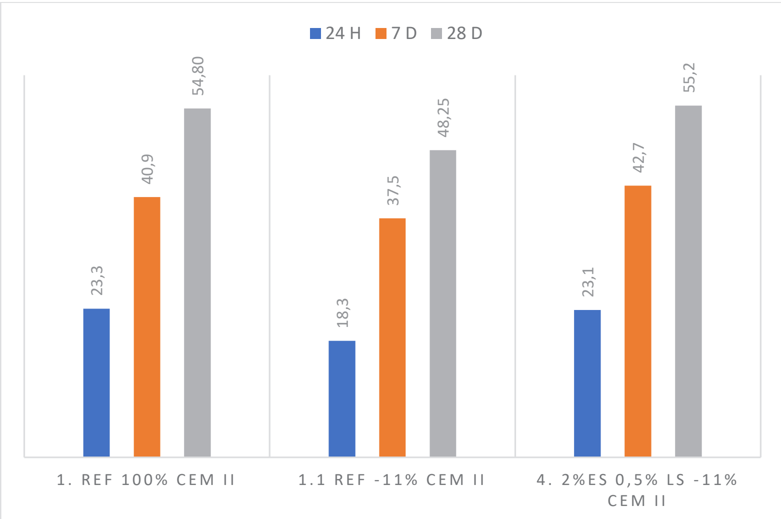



Figure 4 positive effects of Ls accelerator

Table 8- shows the effects of increasing the Cement or the dosage of Ls accelerator.

| **Formula** | **24 H** | **7 days** | **28 days** |
|---|---|---|---|
| **ref 450 g cement** | 23.1 | 39.9 | 50.2 |
| **425g CEM II Es 2% Ls 0,5%** | 28.7 | 47.6 | 58.3 |
| **400g CEM II Es 2% Ls 1%** | 26.2 | 44.5 | 57.1 |

**DISCUSSION**

The method is meant as an early-stage method to discover if the effect of hardening accelerator can improve the performance of a concrete by a lowering the content of clinker. These tests have not been done in shotcrete, since it requires another setup on a large scale with equipment for spraying concrete. Mortar prisms are easier to caste than full size tests with shotcrete. The value from this test is not meant to be exactly, but showing the trend and effect with different dosage of accelerator in different mix of mortar. Prisms without accelerator has good workability and it is easy to cast, since it is easy to compact the mortar into the mold. In the samples with AFA in the mortar mix, the workability completely changed with AFA added in the mixing procedure, the setting occurs significantly faster, and it is the hard to obtain good and stable density of mortar prismns. The negative effect are greater after 0.5MPa. At this stage the standard control's compressive strength on machine is required for the compressive strength of prisms. Early strength 2-24h is difficult to measure on casted mortar prisms in lab. A test in industrial scale on sprayed concrete would given a better results.

From early age compression at prisms with AFA the hardening effect of low carbon mortar gives greater performance than high dosage of cement, in the first period up to 4 hours. This can be due to the amount of accelerators calculated of binder. The total amount of binder is always 450 g, at reference 1. The amount of accelerator AFA is based on 450 g of binder. In all other samples the dosage of cement is lower than the reference sample. For example, the 22% reduction of cement the dosage of accelerator in terms of cement will be higher. The AFA is just working with clinker in binder and therefore the actual dosage of AFA will be significantly higher when cement is reduced. For this reason, the most reliable values are after 4 hours.

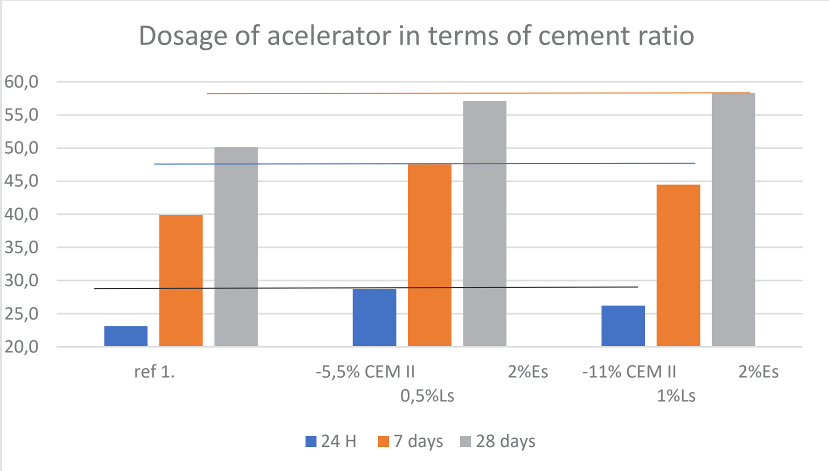
The late strength results without an accelerator is generally higher and more stable than results with a AFA. It can be related to it being difficult to create proper prisms without the shotcrete as sprayed. The spraying gives good compacting in comparison by hand jolting. The effect of extra mixing time of 20 seconds after the accelerator is added also delays the hardening effect. In a shotcrete test this mixing time is avoided when the product is sprayed.

The w/c ratio is also slightly different from plastic-mortar with, and without AFA. The mortar has always the same w/c ratio before the added AFA to the mortar. Keep in mind that AFA has a dry solid content of 52%. This introduces 15.2g of extra water in the mortar. The difference in terms of water cement ratio is the same for all prisms with AFA, but it makes it difficult to compare the mortar with AFA to mortars without AFA.

The values in table 6. show clearly that Ls accelerator has a positive effect compared to the ref sample, but it is too heavy to obtain the same, performance for a LC1 mortar as a reference sample with 100% cement at the binder. At table 7. LC2 the binder is only reduced by 11 percent and the difference between ref and sample is greater. Values at table 5 showing Ls accelerator, improve the compressive strength compared with the reference sample at 24 and 7 days. And 28 days.) If the dosage of LS accelerator is increased to 1% the mortar of LC2 gives greater values than Ref sample.

## CONCLUSION

Figure 5- shows the critical limit of cement reduction as working for the Ls accelerators.




- Hardening accelerator has a promising effect on low carbon-mortar with a reduction of 11 % cement of the binder.
- By using both Es and Ls accelerators the shotcrete with the cement of low carbon content gives values near of greater than a reference sample.
- In this study LC1 (with 11 % reduction in cement content) shows the most promising results.
- In this study a mortar with 450 grams of cement was used as a reference. "Transformed" into 1 m3, this means a concrete mix design with 450 kg cement/m3. If this mix design has a reduction of 11 % cement, this will correspond to a reduction of 50 kg cement/$m^3$. According to calculations in table 1, the saving in $CO^2$ will be between 28 and 30 kg $CO^2/m^3$ sprayed concrete. In Norway the yearly production of sprayed concrete is ~400 000 $m^3$. If all this volume is Low Carbon with reduction of 50 kg cement/$m^3$, this will be in the range 11 200 – 12 000 tons $CO^2$/year.

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# SPRAYED ULTRA-HIGH-PERFORMANCE FIBRE-REINFORCED CONCRETE FOR TUNNEL REHABILITATION

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**Abstract**
Rehabilitation of tunnels is crucial for maintaining transportation infrastructure. Multiple challenges are related to achieving the best possible quality at the lowest possible cost while minimising societal costs caused by downtime. Rehabilitation of concrete linings in railway tunnels with limited space for new overlay before conflicting with traffic area, might be one example. Tunnels face issues with water ingress, requiring additional waterproofing measures to protect against hydrostatic pressure, icing and environmental damage. Shotcrete, a common tunnel lining material, is essential but not impermeable, necessitating supplementary waterproofing layers. This situation calls for innovation in both materials and processes. Ultra-High Performance Fibre Reinforced Concrete (UHPC/UHPFRC) has existed for several decades already but has not yet achieved widespread application despite some superior properties compared to standard concrete. The use of UHPFRC in tunnel rehabilitation might offer enhanced durability and strength, crucial for ensuring the long-term integrity of tunnels. The composition of UHPFRC significantly impacts its mechanical properties and resistance to external agents like chlorides and water – and not least its ability to be sprayed. This paper presents an exploration of the state-of-the-art on sprayed UHPFRC. Overall, the integration of UHPFRC in tunnel rehabilitation presents a promising solution for enhancing structural performance and longevity in tunnel infrastructure, but there are challenges to be solved before application.

**Keywords:** Ultra-High Performance Fibre-Reinforced Concrete, UHPC/UHPFRC, Sprayed concrete/ Shotcrete, Materials development, Innovation

## INTRODUCTION

Every fourth year, the Norwegian Engineers' Association prepares a "State of the Nation" report to evaluate our physical public infrastructure. In the 2021 edition, the condition of the railway network was considered to be in poor conditions, requiring immediate action to prevent loss of functionality [1] Lots of tunnels were found to need rehabilitation. New tunnels have an expected lifespan of 100 years. Traditionally, they are built as drained structures meaning that water from surrounding areas can seep in [2]. The reason for this is to mitigate potential harm to the surrounding natural environment, such as flooding. Water sources situated above the tunnel may exert hydrostatic pressure on the upper surface of the tunnel [3], emphasizing the importance of a waterproof layer preventing water from penetrating to the traffic area. The rock surface in new tunnels is traditionally covered by a layer of shotcrete or the use of bolts to secure the rock wall. The secured rock wall must withstand suction- and pressure forces caused by running trains, as well as hydrostatic force from surrounding water. However, neither rock bolts nor shotcrete is impermeable, which results in the need an additional layer to fulfil the functional requirements of a waterproof inner structure. Creating a waterproof "umbrella structure" is usually obtained by adding Polyethylene panels (PE-panels) or another waterproof membrane covered with a layer of shotcrete [4]. Traditional shotcrete requires an average thickness of a minimum 60 mm to fulfil the technical requirements, according to national regulations [5].

Shotcrete may be applied using two different methods: dry mixing, or wet mixing. Dry mixing was the dominating technique 40-50 years ago, but it poses some weaknesses, including a relatively high rebound rate reaching up to 30-40 per cent, high production of dust [6], and may cause uneven hydration of cement due to limited mixing time [7]. Wet mixing is today the predominantly used method and has a rebound rate of around 5 per cent [3, p. 51]. Wet mixing often yields more predictable results due to the use of ready-mix, where the water-to-binder ratio (w/b-ratio) is more controllable [7]. The shotcrete needs to be able to adhere to a surface instantly and relies on early strength development to avoid fallout. Typically, the early strength development is characterized by a steady hourly increase of 1 MPa, leading to the attainment of 70-80 per cent strength within 7 days [8]. To obtain these properties, a set accelerator is added to the mixture at the nozzle. The components are mixed in the nozzle by the use of compressed air and sprayed onto the intended surface. This causes the mixing time to be limited to a small amount of time starting from the accelerator's addition at the nozzle, to the shotcrete reaching the intended surface.

Durability is a crucial characteristic of concrete, referring to its capacity to withstand diverse environmental loads without significant deterioration [9]. Today, many national railway tunnels are experiencing damaged inner linings caused by insufficient waterproofing, freeze-thaw cycles, mechanical damage, etc., allowing water to leak through the cracks. One consequence of damaged linings might be the build-up of ice on pavement or railway tracks or falling icicles from the ceiling. Parts of old linings are often structurally healthy, and reuse of this rather than removing all would be economically advantageous. Reusing parts of the old structures would also reduce downtime and, thus, be advantageous from a socioeconomic perspective. For this purpose, the use of Ultra-High Performance Fibre-Reinforced Concrete (UHPFRC) might be beneficial due to the superior strength, bonding, and durability of this material.

UHPFRC is a cement-based composite characterised by a compressive strength of minimum 120MPa [10], exploitable tensile strength and increased durability compared to traditional concrete caused by a discontinuous system of only very small pores. UHPFRC typically contains a large amount of cement, silica fume and fine particles, causing a dense and almost impermeable material. The w/b-ratio is low, typically range from 0.15-0.25. To obtain a self-compactable matrix, high dosages of superplasticiser are added. Large amounts of micro steel fibres result in a relatively high tensile strength. Properties obtainable for UHPFRC are relatively well known within research communities and the material is used for rehabilitation in some countries like the USA [11], France, and Switzerland [12]. In other countries the use is limited. UHPFRC might offer cost-effective solutions for some types of projects, by allowing a reduction in material usage and construction time. It enhances structural lifespan, matches new constructions and might when used as an overlay, improve stiffness, mechanical strength, and waterproofing. Typically, it requires preparing the existing surface with water jetting for optimal bonding. A study conducted by Abellán-García et al. [13] evaluated the bonding between existing concrete and UHPFRC, showing that the failure occurs in the existing concrete, indicating a sufficient adhesive strength in the intersection. In addition, UHPFRC is considered to have a low permeability due to its low porosity and the small maximum grain size [14]. To further reduce the permeability and obtain an impermeable layer, silica fume may be added to the matrix.

## RESEARCH QUESTION AND INDUSTRIAL IMPLICATIONS

The predominant method for placement of UHPFRC is by using gravity and formwork. When used as a repair material for tunnel lining, spraying might be a beneficial application method to limit construction time and extent. The object of this paper is to reveal state-of-the-art of the use of UHPFRC as shotcrete, and to answer the following research question:

*How can UHPFRC be used as shotcrete for the rehabilitation of railway tunnels?*

Using UHPFRC as a shotcrete may enable a more efficient application of the material, and repair railway tunnels with a thinner layer. This may be used to prevent a more comprehensive, resource-demanding and time-consuming rehabilitation process.

## CASE

The railway tunnels in Norway are exhibiting severe damage including water leaking through the ceiling. Water may hit the tracks, allowing for the tracks to be covered in ice during wintertime. Removing this ice to preserve a safe travel passage is a resource-demanding process, that negatively impacts society. In addition, the water may freeze before dripping down, causing the ceiling to be further exposed to degradation and the creation of icicles, as pictured in Figure 1. Consequently, the state might pose a threat to passing trains.

*Figure 1: Pictures of water penetrating the lining and creates icicles in an existing tunnel.*

Conventional tunnel rehabilitation is usually done to detain damaged sections and improve the lifespan, instead of doing comprehensive rehabilitation obtaining a lifespan equal to a new structure. Two methods are often used in the traditional rehabilitation of tunnels in Norway. Primarily, water leakages are sealed with waterproof plates, with an expected lifetime of 20 years. These plates are bolted to the rocks and drain the water down to each side of the tunnel. The installation is simple and quick, thus is the most used method in railway tunnels that exhibit water leakages. Downtime is often limited to 2-3 hours and might even be decreased in more trafficked areas. This seems efficient. However, the repair is temporary and not sustainable or cost-efficient in the long run. The structure may exhibit other damage because the structure's weak spot is covered, and not repaired. When the concrete or rock ceiling is severely damaged, the predominating solution is to chisel away existing concrete, and water the surface before applying a new layer of shotcrete.

Alternatively, water jetting and shotcrete may be used. This is a permanent repair. However, considering railway tunnels, the gap between the lining and the train is limited. If a new layer of shotcrete were to be applied, the opening of the tunnel would decrease, and the trains may not be able to pass through. Due to this, a large section of the tunnel lining must be removed in a time-consuming and costly process, to use conventional rehabilitation methods with traditional concrete. It would be beneficial to utilise the superior properties of UHPFRC that could be applied in thinner layers and yet meet the requirements without sufficiently decreasing the tunnel's cross-section.

## METHODS

A literature review of research databases is used to map state-of-the-art of sprayed UHPFRC. Since the number of papers seems very small, personal contacts were established towards what seems to be key researchers in the area. This second part was limited to European stakeholders, having relatively corresponding regulations for tunnel repair. A snowballing technique was used to increase the number of informants, also including industrial actors. The result was an insight also into ongoing projects with the aim of increasing existing knowledge.

## RESULTS AND DISCUSSION

A few studies have been conducted on sprayed UHPFRC (SUHPFRC). All the article identified by the authors in this report, have been divided into two groups; material composition [15] [16] [17] [18], and structural level [19] [20] [21] [22] [23] [24] [25] [26]. The majority of the published research is, to the authors' knowledge, considering a patent-restricted material developed by Lafarge Holcim, called Ductal Grey Shotcrete. Thus, the composition of this material is not known. These studies, however, contain valuable research considering the application of SUHPFRC in a structure. To distinguish between the various approaches described in the articles, this chapter starts with an introduction to previous research, followed by a description of SUHPFRC at the material level and finally implementation of the material on a structural level.

Conventional shotcretes face challenges in strength, durability, and impermeability. Sprayed UHPFRC might address these issues by blending shotcrete and UHPFRC technologies, allowing for a thinner layer with superior properties. According to Cui et al. [15] SUHPFRC exhibits enhanced mechanical strength, improved impermeability, and higher tensile strength. Research done by Strotmann et al. [21] describes two different methods of achieving a sprayable UHPFRC, using the wet mixing method. One method involves introducing a set accelerator at the nozzle during the application, facilitating rapid solidification similar to spraying conventional shotcrete. Alternatively, one can modify the UHPFRC to attain rheological properties that obviate the need for accelerators. Critical to pumpability is ensuring low plastic viscosity to prevent pipe clogging, and adhesion ensured by sufficient yield stress.

Research conducted by Lafarge Holcim [16] stated that the crucial aspect of converting UHPFRC into a substance that can be pumped and sprayed lies in finding the right balance between viscosity and yield stress. Conventional UHPFRC typically has a low yield stress due to its self-compactness, making it suitable for pumping but less so for spraying. To prevent the SUHPFRC from rebounding, the material's yield stress must be increased by modifying the rheology, unless a set accelerator is used. Lafarge Holcim's tests, conducted using an Anton Paar MCR301 rheometer, identified a relatively narrow range of viscosity and yield stress values conducive to both pumping and spraying, as detailed in Table 1. Similarly, Cui et al. [15] used Visomat eBT2 to determine sufficient yield stress and plastic viscosity of 0.48Nm and 7.6Ns, respectively. The attractive yield stress is obtained by adding a viscosity-enhancing agent (VEA).

*Table 1: Value of yield stress and viscosity for pumping and spraying UHPFRC, determined by LafargeHolcim [16]*

| **Material property** | **Value** |
|---|---|
| Yield stress (at shear rate $0.07s^{-1}$) | >300 Pa |
| Viscosity (at shear rate $15s^{-1}$) | 20 – 40 Pa |

**Material level**

Cui et al. [15] [17] focus on SUHPFRC at a material level by using VEA to obtain sufficient yield stress and viscosity. Among three evaluated VEAs with different dosages [17], 0.7‰ Hydroxypropyl methylcellulose (HPMC) illustrated the best results and was used to further evaluate the material properties [15]. The mixture was claimed to show promising results; however, the early strength development was low. This corresponds to the test done by Strotman et al. [27] on the material Ductal Grey Shotcrete, which exhibits no strength development during the first 12 hours. Nonetheless, the material's high yield stress ensures sufficient adhesion to prevent the material from rebounding. The low strength early development may limit the use of the material in some structures, as further elaborated in "Structural level" later in this paper.

To address the disadvantage with the low early strength development, Cui et al. [18] substituted the VEA with a set accelerator, $Al_2(SO_4)_3$. A sample test was conducted of a specimen oriented 75 degrees towards the ground, containing a set accelerator dosage of 2 percent of the weight of the binders. Results indicate that the accelerator enhances hydration in the early stage, utilizing a significant amount of water, which negatively impacts the later hydration of SUHPFRC and final strength. Additionally, the reaction leads to air bubbles being trapped in the fresh matrix, resulting in macro pores in the hardened SUHPFRC. According to Strotmann et al. [27] adding a set accelerator may yield better results; however, it is limited by today's equipment. During pumping of SUHPFRC, the material flow may be irregular, due to the consistency. Pre-nozzle set accelerator addition risks uneven distribution, hindering proper reaction. This phenomenon happens in conventional shotcrete as well, but the larger amount of cement in the UHPFRC may cause the reaction to differ. Uneven accelerator distribution may impede setting in some areas and cause excess in others. Additionally, surface finishing may be more challenging with accelerators. The reason for this is that concrete theoretically solidifies upon impact with the wall, thus may not be possible to process further in fresh state. However, the method involving modifying the fresh properties of the SUHPFRC does not set immediately, allowing for further processing while the material remains workable.

The strength of SUHPFRC depends on the amount of fibres and the constituents of the material. When spraying UHPFRC, the fibres tend to arrange parallel to the sprayed surface [15]. This causes an increased tensile strength and makes it harder for chlorides, water, and other substances to penetrate the hardened SUHPFRC [23], due to the fibres limiting cracks to grow beyond initial state. Rather, when engaging the fibres in the initial state of cracking, the tensile strength in this crack increase. The result is a strain hardening behaviour leading to the growth of a distributed net of micro-cracks giving the material an increased tensile strength, rather than a few larger cracks which weakens the material and allows for penetration of degrading agents. The recommended fibre content differs in the existing literature, and fibre content should be determined based on the required mechanical properties. Cui et al. [15] argue that the fibre content should be limited to 1.5 vol-%, as exceeding this amount tends to result in clogging and fibre agglomeration. A study by Trucy et al. [16] considering Ductal Grey Shotcrete recommends a fibre content of at least 3 vol-% to obtain the required strength and to achieve strain hardening. The fibre content directly affects the strength of the material, partly explaining why Cui et al. [15] [17] [18] obtained a lower strength than Strotmann and Jungwirth [21], as shown in Table 2 and Table 3, respectively.

Table 2: Mechanical properties of SUHPFRC with HPMC [15] [17] and alkali-free accelerator [18].

| Mechanical Properties | SUHPFRC with HPMC | SUHPFRC with alkali-free set accelerator |
|---|---|---|
| Compressive strength (Cube) | 109.2 MPa | 113.6 MPa |
| Split strength | 13.7 MPa | 18.2 MPa |
| Uniaxial compressive strength (28 days) | 89.2 MPa | 90.7 MPa |
| Elastic modulus | 39.1 GPa | 40.1 GPa |
| Spraying thickness | 50 mm | 45 mm |

Table 3: Mechanical properties of the sprayed UHPFRC evaluated by Strotmann et al. [21].

| Strength parameter | Value |
|---|---|
| Compressive strength (Cylinder) | 120 MPa |
| Compressive strength (Cube) | 130 MPa |
| Compressive strength (Direction of injection) | 190 MPa |
| E-modulus | 41.5 GPa |
| Tensile strength | 6 MPa |
| Flexural tensile strength | 15.6 – 23.5 MPa |
| Spraying thickness | 30-80mm |

SUHPFRC exhibits a higher strength and higher resistance to degradation than conventional shotcrete. Table 4 shows a comparison of the average range of the known mix design of sprayed UHPFRC [15] [17] [18] [21], compared to conventional shotcrete. Silica fume enhances pumpability, durability, and permeability, reduces rebound, and improves concrete adhesion. This, together with the small grain size, results in an impermeable material. However, the tests are not conducted on a thinner specimen. According to Strotmann and Jungwirth's [21] [24], SUHPFRC shows similar behaviour in small scale and large scale projects, and the material will likely remain impermeable in thinner layers. It is also claimed that further research is needed on the scaling effect of SUHPFRC concerning water penetration.

Table 4: Vital constituents of shotcrete and correspondingly SUHPFRC [15] [17] [18] [21].

| | Shotcrete | UHPFRC shotcrete |
|---|---|---|
| Cement content [kg] | 450 – 500 | 775 – 785 |
| Silica fume [kg] | 20 | 140 – 160 |
| Limestone powder [kg] | N/A | 140 – 170 |
| w/b-ratio | 0.41 – 0.42 | 0.15 – 0.25 |
| $D_{max}$ [mm] | 8 | 1 – 2 |
| SP [% of weight of cement] | 0.85 | 1.5 – 2.5 |
| Accelerator [% of weight of cement] | 6 – 10 | 1.0 – 3.0<br>0 if VEA is added |

**Structural level**

Studies by Al-Ameen et al. [22], Cui et al. [15] [17] [18] and Strotmann and Jungwirth et al. [20] [24] [25], concludes that SUHPFRC is feasible to be used as a repairing material. Considering the usage of SUHPFRC, a research group headed by Jungwirth [20], has conducted several tests using Ductal Grey Shotcrete, to investigate how a layer of SUHPFRC can improve existing infrastructure in need of repairing. A composite structure is formed when SUHPFRC is applied to an existing structure made with another material, which emphasizes the importance of evaluating the mechanical properties of both layers to identify the weakest one. Strotmann et al. [25] carried out tests on a casted beam by adding a repairing layer of SUHPFRC, resulting in an increased bearing capacity and a decreased crack width. A follow-up study [26] was conducted to analyse the crack distribution in the SUHPFRC layers to mitigate chloride penetration in an overloaded or damaged structure. Beams with and without additional reinforcement in the SUHPFRC layer, and the joints between them, were examined. The result showed that a layer of SUHPFRC without additional rebar is not sufficient to prevent chloride penetration in beams, due to cracks exceeding 0.1 mm. Furthermore, Strotmann and Jungwirth concluded that there were no systematic failures observed at the joints and hypothesized that the conventional concrete and the new SUHPFRC layer operated as a monolithic structure.

Cracks occurs when the tensile stress exceeds the capacity of the material. Since a tunnel is designed as an arc, the joints and the SUHPFRC layer in general is not exposed to a significant amount of tensile stress, compared to bridges and other beam structures. Considering this, a layer of SUHPFRC even without additional rebar might be sufficient for rehabilitation of railway tunnels where the main purpose is to prevent water from penetrating.

Today's need for tunnel rehabilitation, combined with budget constraints and the need to minimize downtime, prompts consideration of a more comprehensive approach than what is traditionally used. While traditional solutions involve waterproof plates designed to last 10-20 years, newer structures offer a designed lifespan of 100 years. Covering the entire prepared surface with sprayed UHPFRC, might enhance water resistance, prevent seepage, and even strengthen resistance towards rock fallout. This measure might effectively create a monolithic umbrella structure, and consequently also reduce problems related to the formation of icicles or ice on the tracks. Such enhancements may significantly improve train safety. Currently, tracks are manually cleared of ice during winter using labour-intensive methods. By preventing water accumulation, this task may become redundant, freeing up manpower for allocation elsewhere. The conventional shotcrete layer applied may cause the cross-section to be reduced below the minimum requirement needed for the trains to pass. Thus, the method would increase downtime, might provoke the need to chisel away a larger amount of concrete, or in the worst-case scenario require blasting a bigger hole to achieve a sufficient cross-section. This is an extremely costly and time-consuming process that beneficially could be avoided. By employing SUHPFRC, a thorough rehabilitation could potentially be carried out more efficiently and at a lower cost, yielding long-term benefits. Both Al-Ameen et al. [22] and i-SCUP [26] emphasize that SUHPFRC can be applied in thin layers, with minimum rebound, to strengthen and prolong a damaged structure.

Railway tunnels in Norway must fulfil functional requirements to obtain a dry surface of tunnel ceilings, as well as being structurally secured. The necessary layer is traditionally found to be 60-80 mm. As illustrated in Figure 2, SUHPFRC can be used to obtain a smaller thickness while maintaining integrity. Utilizing the mechanical properties of SUHPFRC, a wholesome rehabilitation may be conducted in a resource-efficient, sustainable and time-saving process.

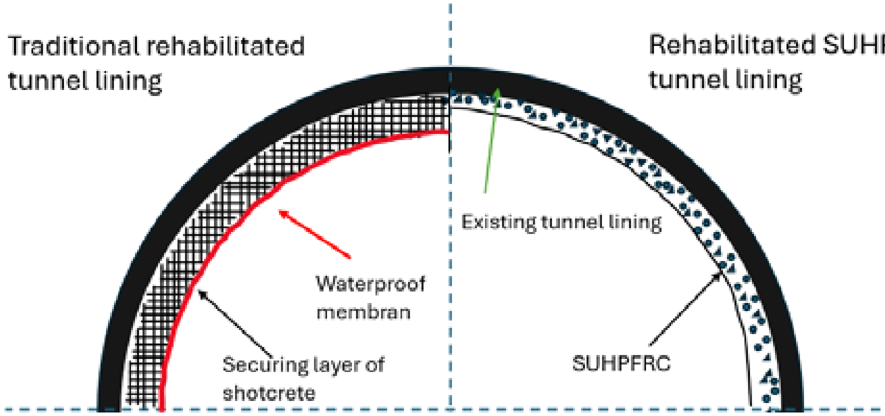

*Figure 2: Functional requirements of a dry surface possibly achieved by traditional methods and SUHPFRC.*

Railway tunnels face a tight rehabilitation schedule due to limited traffic-free windows. To mitigate economic losses and societal disruptions from train traffic interruptions, downtime should be minimized. As mentioned earlier, the strength development of SUHPFRC involving modifying the rheology, is low. Given the typical downtime of a few hours, expediting the material's strength development would be beneficial, and may have a positive socio-economic impact. One way to achieve this may be by incorporating a set accelerator. Although the technology for set accelerators in SUHPFRC is still underdeveloped, it might offer a more efficient solution. In contrast, in other rehabilitation projects, downtime may not be as critical, allowing structures to remain undisturbed until the strength development has grown sufficient. This disparity underscores one of the key differences between railway tunnels and other infrastructure, emphasizing some potential impacts of efficient set accelerators.

## CONCLUSIONS

Utilisation of sprayed UHPFRC seems to be a feasible solution to enhance the rehabilitation of railway tunnels while maintaining a sufficient cross-section area for trains to pass and minimising societal costs caused by downtime. The material properties of UHPFRC enable a thinner layer of repairing, substituting the traditional waterproofing layer combined with traditional shotcrete. UHPFRC can be made sprayable by modifying the rheology by adding a VEA, or by adding a set accelerator. The known technology is limited and should be further developed.

One main challenge when spraying concrete of all types is to achieve an even distribution on the substrate while maintaining the properties needed for rapid hardening and avoiding rebound. Shortcomings of knowledge in this field might be summarised as:

- When modifying the fresh state of the UHPFRC to obtain sufficient viscosity and yield stress needed to avoid rebound, the early strength development is low. In railway tunnels, with limited downtime, the strength development should start instantly to be able to open the tunnel for passing trains. Research done on the SUHPFRC without the addition of a set accelerator, indicated that the strength development does not start until 10-12 hours after application, thus the material cannot withstand the suction- and pressure force in this period. In railway tunnels, this is not sufficient, emphasizing the need of using set accelerator.
- SUHPFRC with set accelerator is not a well-developed technology. Applying the set accelerator is a complex process that requires better equipment to obtain an even distribution of set accelerator in the mixture. The research is limited and should be further explored.
- When spraying fibre-reinforced UHPFRC, some fibres tend to be located at the surface, causing a sharp and uneven surface. The surface then needs to be treated to prevent hazardous situations,

where people and animals can be hurt. These situations are likely to occur in a case of emergency, where passengers need to evacuate the tunnel.
- The lack of formal knowledge, experience, successful pilot projects and documentation, prevents implementation.
- Today's equipment for spraying concrete is developed for traditional concrete. Using the same for spraying UHPFRC might be challenging. Not least, spraying concrete requires skilled operators who have learned through experience. Equipment and operator skills needed to efficiently and evenly apply a set accelerator in the spraying process must be developed before this method can be utilised. Alternatively, development of special knowledge and equipment to exploit the utilising of rheological properties is required.
- Knowledge is needed on how the SUHPFRC's thickness and additives affect properties like permeability, to ensure that the repairing layer is kept as thin as possible while maintaining its structural integrity.

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# COMPACTION-, ANISOTROPY- AND CRACK EFFECTS ON WATER TRANSPORT IN WET-SPRAYED CONCRETE

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## Abstract

Durability of wet-sprayed concrete tunnel linings depends on water transport. In addition to the material and structure (w/b, thickness etc), transport is affected by execution of spraying, curing etc that can cause open compaction voids, anisotropic material properties and cracks. Full-scale spraying experiments were made to study the effect of compaction, anisotropy and cracks on transport properties in concrete specimens from cores drilled from sprayed panels. Macro void volume measured on colour-impregnated polished sections by IMage Analysis ($\varepsilon_{IMA}$) was always larger than (closed) macro void volume measured by pressure saturation of capillary saturated samples ($\varepsilon_{PF\ cl}$). Also, the additional water uptake from one sided capillary suction to submerged suction was measured and taken as open macro voids ($\varepsilon_{cap\ suc\ open\ macro}$). From the 3 measurements made on 20 panels with varying mix compositions, accelerator dosages and spray execution the fraction of open macro voids was calculated as (1): 1-($\varepsilon_{PF\ cl}/\varepsilon_{IMA}$), varying from 0.04 to 0.43, and as (2) $\varepsilon_{cap\ suc\ open\ macro}/(\varepsilon_{cap\ suc\ open\ macro}+\varepsilon_{PF\ cl})$, varying from 0.09 to 0.2. The fractions (1) and (2) correlated so the fast measurement (2) quantifies compaction well. CT-scanning on selected cores showed somewhat higher macro void content than IMA. IMA also showed a tendency to elongated macro void shape (L/W >1) with L oriented normal to spraying and there was a marked increase in water penetration normal to the spraying direction compared to parallel to spraying, hence anisotropic macro void- and water transport properties. CT-scanning also showed steel-fibre orientation normal to spray direction. Studies of flow in cracks ($w \approx 0.1 - 0.3$ mm) induced by DIC-controlled tensile splitting (“Brazilian test”) showed marked increase of capillary suction and laminar crack-flow at increased w. A co-polymer (“latex”) powder reduced crack flow markedly compared to in reference mix. Furthermore, self-healing in the form of reduced flow through the cracks was observed in wet sprayed concrete following water storage and retesting crack flow several times over an 85 day-period. 3D CT-scanning gave similar crack-volume as estimates from 2D DIC and optical microscopy.

Key words: sprayed concrete, tunnel linings, water transport, compaction, anisotropy, cracks, healing

## INTRODUCTION

There is increasing interest in using sprayed concrete for permanent tunnel lining, in addition to the function as temporary rock support during production of tunnels. The desired, or required, concrete properties for function as permanent lining and for temporary support are often quite different. High early tensile strength is needed for temporary (or immediate) support, and long-term water tightness is often needed for permanent linings. There are, however, coinciding desires for the properties needed for the two functions. Permanent linings can also function as final or permanent rock support, low compaction void content is always desirable, avoiding the formation of wide cracks is important and low environmental impact is a must. An overview of the development of sprayed concrete technology and required properties desired for both immediate support and permanent lining can be obtained from different sources of information and perspectives. The overview can be based on experience from industrial practice, from owners of tunnels with their requirements, academic publications etc. Present Norwegian practice, recommendations, and requirements of concrete spraying in tunnel production is well summed up in [1]. To succeed in making satisfactory permanent sprayed concrete tunnel linings the kind of knowledge just mentioned should be reviewed carefully. In Norway 4 previous PhD-theses have been dedicated to sprayed concrete. In the first Norwegian PhD on sprayed concrete by Opsahl [2] the technology as we know it today was mainly already developed and measuring permeability was central in the research, though only alkali-based accelerators were used. In [3] international recommendations were presented including pointing to sealing as a main functional requirement of permanent linings. The work by Hagelia [4] focused entirely on durability, hypothesizing that degradation of sprayed concrete layers < 100 mm is due to chemical and/or biological attack. In the work by Holter [5] the focus was on water transport. Sprayed concrete according to [1] without defects was mostly found to have permeability $K < 5.10^{-14}$ m/s. The work [5] proposed various double-shell solutions and use of membranes in between the shells to avoid transport and frost-related problems. In the recent work by Manquehual [6] long-term durability of field exposed sprayed concrete was investigated. Long-term chemical changes were studied in cores from an early subsea tunnel produced with alkali-free accelerators. The results [6,7] indicated severe leaching after 25 years of service, pointing to the importance of pore structure and transport properties for the durability, service life and environmental impact of sprayed concrete, as expected [8]. Sprayed concrete tunnel linings in contact with water-bearing ground are subjected to water transport. Water transport through tunnel linings is undesirable due to visible water ingress within the tunnel, durability of the fixings and fittings within the tunnel and for the durability of the concrete lining itself. Water transport underlies most degradation phenomena in concrete [8]. However, sprayed concrete tunnel lining properties are resulting from at least as many factors as for conventionally cast concrete structures. The factors include composition and production of ready mixed concrete, transport, execution of substrate and spraying including accelerator use, curing and exposure of young and hardened sprayed concrete lining. There is a high number of possible execution parameters such as equipment, accelerator type- and dosage, substrate condition (rock type and surface topography, -roughness, -cleaning, -water leakage etc), concrete pumping speed, air pressure, nozzle movement at spraying, spray distance and angle, temporary halts in the concrete flow, the compaction in terms of amount of irregular compaction voids and their properties, curing conditions after spray (drying, temperature) etc. The publication [1] is addressing most of this to ensure high quality. In practice, however, cracks are seen in sprayed concrete tunnel linings [5], compaction may be variable, and layering can occur during spraying [9]. Some potential pathways for transport of groundwater and deleterious substances through the concrete are open macro voids, the capillary pore system, or cracks in the lining. There is a need for research to understand these challenges and to make improvements wherever needed.

This paper is based on results of the PhD thesis [9-14] written as part of the project [15] where the main objectives were to investigate and develop SUstainable sprayed PERmanent CONcrete tunnel linings. The background for the project was the interest from Norwegian sprayed concrete industry to promote sprayed concrete tunnel linings as a sustainable alternative for permanent concrete tunnel linings by offering less concrete consumption per unit length of tunnel compared to solutions with cast vaults. The sustainability of existing concrete structures depends largely on the consumption of concrete and on the service life of the concrete. This work was limited to specific spraying experiments used to produce wet-sprayed concrete specimens made with today's materials and practice among spraying contractors. Specimens were drilled from sprayed panels and investigated in the concrete laboratory. The scope of this paper is to present selected results of [9-14] on effect of compaction, anisotropy, and cracking on water transport in wet sprayed concrete tunnel linings. Open compaction void content was quantified as 1) difference between macro void content detected by optical microscopy and closed voids filled by pressure saturation, and as 2) fraction of macro voids that fill when submerging specimens that first had been capillary saturated by one-sided suction. Anisotropy of water transport properties, of macro-void orientation, and of steel-fibre orientation were measured by water penetration measurements normal and parallel to spraying direction, image analysis and CT-scanning. Finally, the effect of cracking on water transport by capillary suction and hydrostatic water-pressure were measured, and possible remedies against crack-induced transport (polymer modification, self-healing) were studied.

## MATERIALS AND METHODS

Table 1 shows a selection from [9] of the full-scale sprayed concrete mixes included in this paper to quantify effect of compaction, anisotropy and cracking on transport.

*Table 1 - Overview of concrete mixes and spraying experiments for this paper*

| Location | AMV, Flekkefjord | At SINTEF/NTNU concrete lab | Svorkmo access tunnel Orkanger |
|---|---|---|---|
| Date | February 2020 | June 2020 | March 2021 |
| Concrete mix | w/b=0.42-0.47 | w/b=0.42-0.45 | 0.46-0.49 |
| Bindertype (kg/m$^3$) (CEMIIA+SF)<br>Cement reduction<br>Polymer modification | 462+19<br>-<br>- | 433+43<br>-<br>- | (356-459)+(15-19)<br>Additional FA, LS, EVA polymer etc |
| Matrix volume (litres/m$^3$) | 384 | 437 | 383-445 |
| Accelerator (% of b) | 0-10 % MB Masterroc SA 188 | 0-7% MB Masterroc SA 168 | 7% MB Masterroc SA 188,<br>Hardening acc in high vol FA-mix |
| Concrete batching and transport | Flekkefjord<br>5 min.drive to spraying location | Trondheim<br>10 min. drive to spraying location | Trondheim<br>60 minutes drive to spraying location |
| Spraying robot | AMV 7450 shotcrete robot | Normet Spraymec NorR. 140 DVC shotcrete robot | Normet Spraymec NorR. 140 DVC shotcrete robot |
| Concrete flow (m$^3$/hr) | 10 | 15 | 15-20 |
| Nozzle distance to substrate and angle | 2.0m - 90° | 1.5, 2.5, 3.5 m – 45, 67 and 90° | 1.5-2m - 90° |
| Curing and coring | Wrapped in plastic 1 month, taken to lab for coring.<br>Cured in lab air before capillary suction, PF and image analysis tests. | Wrapped in plastic 1 month, taken to lab for coring. Cured in water until capillary suction, PF and image analysis tests. | Panels stored in tunnel for 5 weeks, taken to lab for coring. Cores stored in water before laboratory tests. |

Concrete spraying of panels was done in full scale with the same kind of equipment used in ordinary production of tunnel linings. Spraying was done at 3 different locations and temperatures. In the first two locations (Flekkefjord, around 5 °C and Trondheim, around 20 °C) the main concrete variables were accelerator dosage (both locations) and spraying mechanics (Trondheim) whereas in the last location (Svorkmo, around 8 °C) the main variable was concrete mix proportions. The other variables possibly affecting quality of sprayed concrete tunnel linings (production of ready mixed concrete, transport, execution of substrate, exposure of young and hardened sprayed concrete lining) were minimized by producing in rather similar ways and as realistically as possible, see Table 1. The full-scale spraying experiments were made on plywood substrates placed on euro-pallets. The sprayed panels were either 120 by 80 cm with 45-degree outward tilted wooden frames or Ø 60 cm circular moulds with vertical steel frames, in both cases providing 10 cm high vertical formwork so that the thicknesses of the sprayed panels were ≈10 cm.

Six sets of laboratory experiments were used to quantify effect of compaction-, anisotropy- and cracking on transport:

<u>Image Analysis (IMA) on polished sections</u> was used to measure total macro- (or compaction-) void content. Grinding and polishing gave a surface in accordance with ASTM C457 with sharp edges of macro voids down to 10 microns [16], then dying the polished concrete surface with a black marker and filling macro voids with white $BaSO_4$-powder with particle size 2 – 4 microns as described in [17]. Finally flat-bed scanning at 2400

ppi gave volume fraction of total macro voids (closed and open) as the white-coloured area fraction, $\varepsilon_{IMA}$. Figure 1 shows specimens for IMA, capillary suction and PF from cores from the sprayed concrete panels.

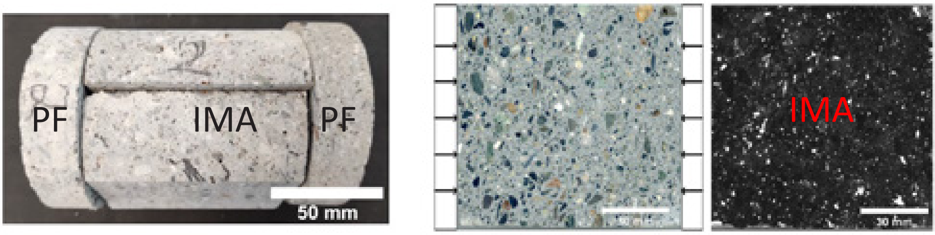


*Figure 1 - Specimens for capillary suction/PF and for IMA (polished, coloured black-white)*

Capillary suction was done on concrete slices cut from cores drilled normal to the sprayed surface, Figure 1. After oven-drying at 105 °C to constant mass, one-sided suction was measured over time to constant mass, see Figure 2. Then the slices were fully submerged until constant mass. Finally concrete specimen volume was measured by weighing in water. The additional water uptake from saturation by one-sided suction to water saturation by submersion at atmospheric pressure was calculated by unit volume of concrete and taken as a measure of open macro void content, $\varepsilon_{cap\ suc,\ open}$.

*Figure 2 - One-sided capillary suction experiment (left) on slices of wet sprayed concrete from drilled cores, with (centre) and without (right) crack.*

PF-testing was then completed, after submerged capillary saturation and volume measurement as described above, by weighing after pressure saturation in a water tank at 5 MPa for 24 h, giving closed macro void content, $\varepsilon_{PF\ cl}$.

IMA of macro void orientation. In addition to measuring macropore content, $\varepsilon_{IMA}$, as described above, IMA was used to analyse the Length/Width ratio and orientation of the irregularly shaped macro voids formed by the spraying compaction with simultaneous flash – set due to the accelerator. This was done by modification of the code [17], see Figure 3 and details in [13].

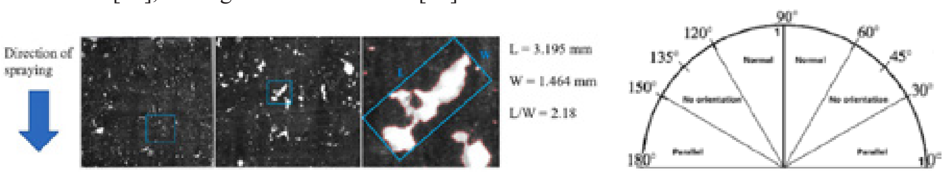


*Figure 3 - Measurement of Length/Width ratio and quantifying orientation of macro voids*

CT-scanning was done on selected cored cylinders of diameter Ø 98 mm and variable heights with a Zeiss METROTOM 1500 CT scanner with the acquisition software METROTOM OS 3.6.2.19227 to observe macro void volume, orientation of steel fibres, crack-volume and crack geometry. The voxel sizes (length in x-, y- and z- dimensions) were down to 67 microns. More details can be found in [13,14] and [18].

Water penetration depth after one-sided water pressure on a concrete surface was measured on sawn concrete specimens. After applying water pressure, the specimens were split mechanically normal to the surface with applied water pressure. Then the penetration depth of the darker waterfront was measured directly by marking with a black marker and photographing the water penetration profile. Selected specimens from the Flekkefjord spraying with 3% and 10 % accelerator were investigated in this way. Water penetration depth was measured in parallel and normal to the spraying direction and equivalent permeability calculated according to [19] from applied pressure, pressurized time, and measured depth.

Crack effect on flow of water in sprayed concrete was studied after crack-width- (w) -controlled tensile splitting loading ("Brazilian test") of selected specimens. Figure 4 (left) shows the DIC-controlled tensile splitting load experiment, the DIC-analysis of crack width, the steady state flow experiment, see Figure 4 (centre) and flow vs time plots, see Figure 4 (right). Crack-induced flow was also detected by capillary suction (see Figure 2, centre).

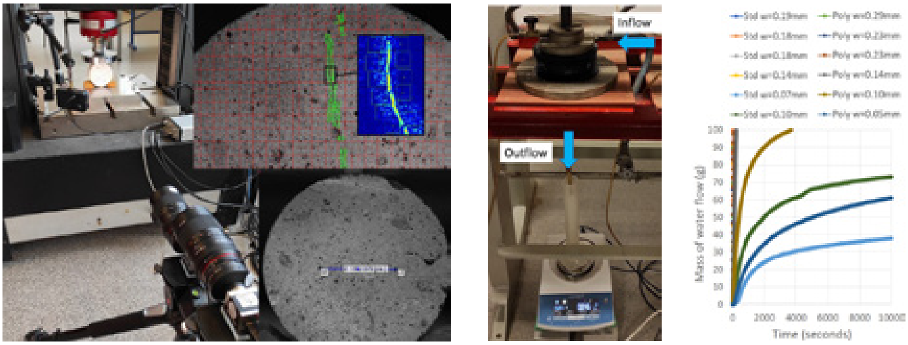


*Figure 4 - crack flow experiment with DIC-controlled crack-width (w) in Brazilian test and crack-width analysis (left), steady-state water crack-flow experiment (centre) and flow plot (right)*

The effect of cracks on steady state flow can be expressed by a crack flow-rate coefficient, $\zeta$ = (measured flow/theoretical laminar flow). For $\zeta = 1.0$ the crack is a perfect slit with laminar viscous flow between the parallel smooth crack surfaces. This is not the case for cracks in concrete due to rough surfaces, varying crack widths and tortuous flow-paths. From the literature $\zeta < 1.0$, see results and discussion. For capillary suction, cracks also increase the flow, though in a different way mainly by increasing the area for suction. More details can be found in [14].

## RESULTS AND DISCUSSION

The fraction of open compaction voids can be calculated in the following two different ways based on IMA, capillary suction and PF:

| | | |
|---|---|---|
| Fraction open (of IMA) | $= 1 - (\varepsilon_{PF\ cl} / \varepsilon_{IMA})$ | (1) |
| Fraction open (of capillary suction+PF) | $= \varepsilon_{cap\ suc,\ open} / (\varepsilon_{PF\ cl} + \varepsilon_{cap\ suc,\ open})$ | (2) |

This was done based on data produced from the experiments. A total of 20 panels were sprayed with varying concrete mixes, accelerator dosages and executions. Then specimens were taken from each panel for IMA and capillary suction/PF. Figure 5 shows the ratios ($\varepsilon_{PF\ cl} / \varepsilon_{IMA}$) and ($\varepsilon_{PF\ cl} + \varepsilon_{cap\ suc,\ open}$) / $\varepsilon_{IMA}$. The first ratio is always < 1. The latter ratio is < 1 with one exception – Flekkefjord 3 % accelerator: 1.05 which we ascribe to scatter. Therefore, with our experimental conditions (105 °C drying, 5 MPa water pressure, polishing, impregnation and scanning as described above) IMA measures the highest macro void content.

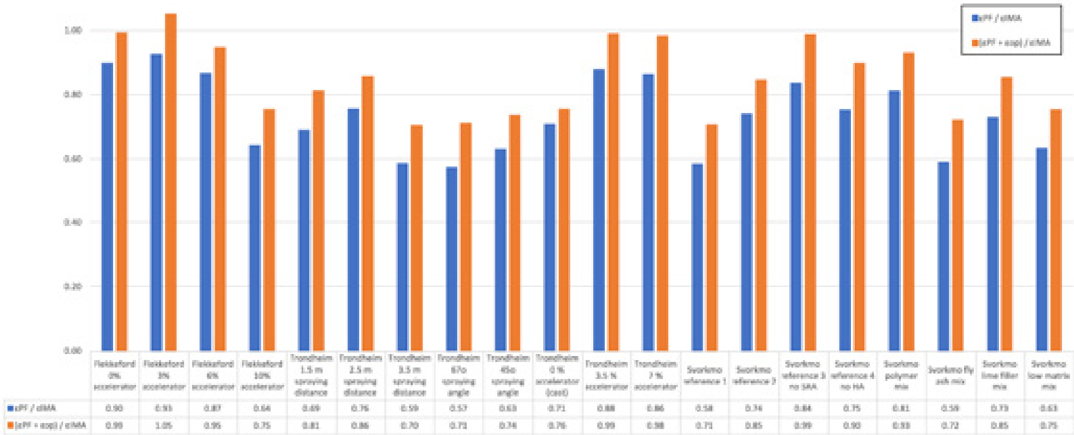


*Figure 5 - Closed and (Closed + open) macro void content measured by PF/capillary suction and expressed as fraction of total macro void content measured by IMA.*

Figure 6 shows Fraction open compaction voids measured and calculated in the two different ways given by equations (1) and (2) above for all the sprayed concrete panels of Table 1. Figure 6 is plotted with the most time-consuming method using Image Analysis and PF along the abscissa and the faster method based on only capillary suction and PF along the ordinate.

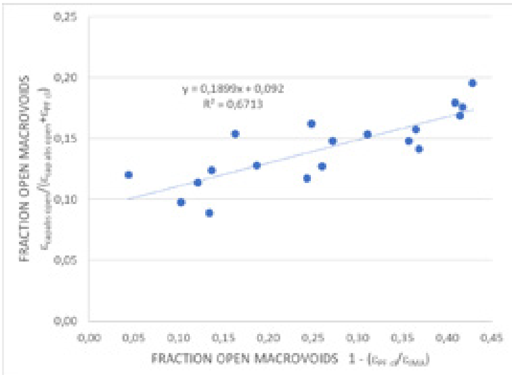


*Figure 6 - Fraction of open macro voids measured by two methods, see eq. (1) and (2)*

Figure 6 shows that the fraction of open macro voids by equation (2) plotted on the ordinate varies from 0.09 to 0.2. The fraction of open macro pores by equation (1) plotted on the abscissa varies more, from 0.04 to 0.43. The plot indicates that the two methods of measuring fraction open macro voids correlate reasonably ($R^2$ = 0.67). Hence the simpler and less laborious (capillary suction + PF)-test and then using equation (2) can probably be used alone as an indication of compaction expressed as fraction of open macro voids in sprayed concrete.

Macro void contents were also measured with CT-scanning on cores from 5 different panels from the Trondheim spraying experiments (90° - 1.5 m, 90° - 2.5 m, 90° - 3.5 m, 45° - 1.5 m, 67° - 1.5 m). These showed (4.7 – 10.1 vol-%). IMA taken from the same 5 panels showed (5.5 – 6.3 vol-%) and PF showed (3.3-4.2 vol-%). CT-scanning at the selected settings (voltage, distance, image analysis parameters in the VGA-software etc) hence gives on average larger macro void content than both IMA and PF, with a scatter though. For IMA and PF the same main result still holds as for all 20 experiments shown in Figures 5 and 6: IMA detects larger macro void content than PF. Again; CT clearly showed how sprayed concrete contains large, irregularly shaped macro pores that are a product of the spray application (pumping, propulsion, possible short halts in flow, impact, flash-set) [13, 18]. These compaction voids are different to spherical pores formed by an air entrainment agent for frost protection [13, 18].

Figure 7 shows examples of orientation of macro voids analysed by IMA using the orientation method of Figure 3. The length of the vectors quantifies the fraction of pores split in 3 different directions: 90-60° (normal) to substrate, 60-30° (no orientation) to substrate and 30-0° (parallel) to substrate. Each vector length is the mean of two parallel specimens. Figure 7 shows that for these two series there is a trend of orientation of macro voids towards parallel to the substrate, with 30–50 % of macro voids oriented in this direction, while 18–33 % are orientated normal to the substrate, although there is a degree of scatter in the data. There is also a tendency that increased accelerator dosage increases the orientation degree and hence increases anisotropy.

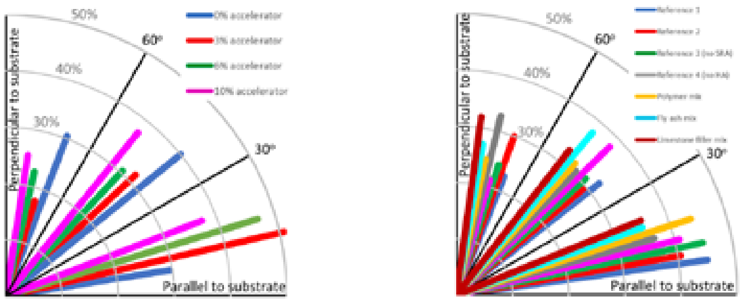

*Figure 7 - Orientation of length axis of macro voids Flekkefjord (left) and Svorkmo (right)*

Results of CT-scanning showed, as expected [20], that steel fibre orientation was anisotropic with a clear tendency that fibres orient normally to the spraying direction, see details in [13].

The results of water penetration measurements performed normal and parallel to the concrete spraying direction were performed for specimens with 3 and 10 % accelerator from the Flekkefjord experiments. Equivalent permeability [19] was calculated from the applied hydrostatic pressure and – time and the corresponding measured penetration depth. The results showed for 3 % accelerator permeabilities $K \approx 3 \cdot 10^{-15}$ m/s and $K \approx 5 \cdot 10^{-12}$ m/s parallel and normal to the spraying direction, respectively. For 10 % accelerator the penetration was much higher giving permeabilities $K \approx 7 \cdot 10^{-12}$ m/s and $K \approx 3 \cdot 10^{-9}$ m/s parallel and normal to the spraying direction, respectively. Hence the results showed clear effects of both orientation and accelerator dosage on water penetration. Permeability was clearly anisotropic with much larger permeability parallel to the spraying direction than normal to the spraying direction. 10 % accelerator gave higher permeability than 3 % accelerator.

In sum the studies of macro void orientation, fibre orientation and permeability in two directions show that for the actual wet sprayed concretes these 3 properties are clearly anisotropic with respect to the spraying direction. The reasons are probably pulsation during spray giving variable accelerator concentration normal to spraying direction, increased tendency to percolation of oriented elongated macro voids and percolated ITZ around oriented and connected steel fibres.

Figure 8 shows the effect of cracks on transport of water by capillary suction (left) and by water pressure (right). Both figures show a clear effect of cracks on water transport and that EVA-polymer reduces transport in both uncracked concrete (capillary suction) and in cracked concrete (capillary suction and steady-state crack flow). In capillary suction rapid rise of water in the crack caused absorption occurring over the surface area of the crack in addition to the area of the base of the disc. Discs with wider crack widths exhibited a higher rate of water permeation per area of crack [14]. The flow-rate coefficient was found to fit better to maximum observed crack width than to average crack width [14]. Cracks were characterized in different ways: by DIC, with optical microscope and by CT-scanning. The latter gave 3D-crack-volumes close to crack-volume estimates from the 2D-measurements with DIC and optical microscopy, but also showed interesting features such as closed parts of the crack and fracture zones around the steel-fibres. Another important finding from the studies of steady state flow in cracks was that sprayed concrete tends to self-heal over time. Best to our knowledge this is the first observation of self-healing in sprayed concrete. This was observed both over the relatively short duration of the flow measurements, and by storing selected specimens up to 3 months in water and repeating measurements of water flow in this period. The latter experiment showed a drop of flow corresponding to a drop of flow rate coefficient, ζ, from 0.078 to 0.005 [13].

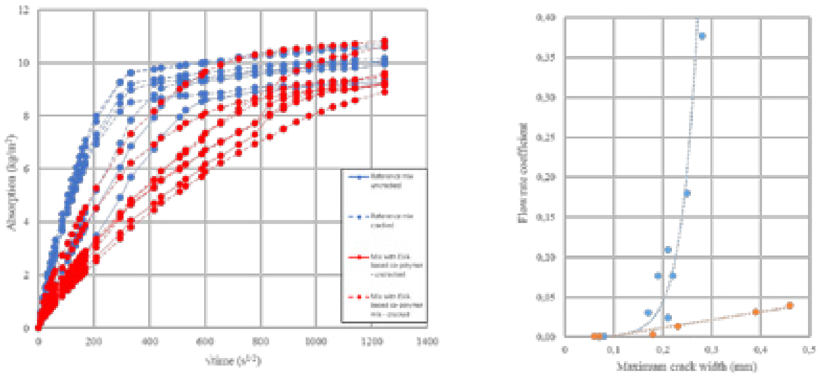

*Figure 8 - Effect of cracking on capillary suction (left) and steady-state flow in cracks (right) for reference mix and mix with polymer [14].*

In the publications [9-14] more thorough reviews are found on early work, e.g. [21], later work on the spray mechanisms, consolidation and macro void characterisation, e.g. [22-25], mitigation of cracking in sprayed concrete e.g. [26-28], alkali-free accelerators e.g. [29,30], environmental aspects [31] etc. Also new developments that combine reduced environmental footprint with isotropic material properties in additive manufacturing can improve spray quality [32]. Finally, based on the observation of mitigation by polymer and healing of cracks when $w < \approx 0.3$ mm one should investigate to what extent crack widths in linings can be limited to this value by design, material selection and execution. The work [5] revealed substantial cracking in tunnel linings with crack width w in the order of 1 mm.

## CONCLUSIONS

Sprayed concrete contains irregularly shaped macro voids caused by the spray application (pumping, propulsion, impact, variable concrete flow with temporary halts, flash-set). These compaction voids are different to spherical pores formed by an air entrainment agent for frost protection. Image analysis of scanned colour-impregnated polished sections, capillary suction and pressurised saturation (PF test) on slices from cores and CT scanning of whole cores were made. These measurements were used to analyse the macro voids in different panels sprayed with varying accelerator doses, varying spraying distances and angles and varying sprayed concrete mixes. The results show that open macro voids defined as filling by water at atmospheric pressure = 9-20 % of total macro voids measured by capillary suction and PF-test. The open macro voids are 4 – 43 % of total macro void content measured by scanning of colour-impregnated polished sections.

The shape of the macro voids was measured to be non-spherical with orientation tending to be parallel to the substrate. Likewise, fibre orientation was measured by CT scanning to tend towards orientation parallel to the substrate. Water penetration tests were done on specimens with the water pressure applied parallel and perpendicular to the direction of concrete spraying. Water penetration was measured to be lower parallel to the direction of spraying than normal to spraying. Therefore permeability was higher perpendicular to the direction of application, due to macro voids, laminations and fibre orientation in this direction, indicating the importance of execution in wet spraying of concrete. Water penetration was lower with 3 % accelerator than with 10 % accelerator.

Experiments on the effect of cracking (w ≈0.1 – 0.3 mm) on water transport of sprayed concrete for tunnel linings showed clear increase of both capillary suction and steady state permeation through cracks. Inclusion of EVA based co-polymer reduced the rate of capillary suction and the permeation flow rate coefficient (measured flow / theoretical viscous laminar flow) for a given crack width compared to the reference mix. Self-healing of cracks occurred during water permeation measurements and most after water storage of cracked discs and repeated measurements.

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**REINFORCED SPRAYED CONCRETE ARCHES FOR PERMANENT ROCK SUPPORT – QUALITY CONTROL.**

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**SUMMARY**

In tunnels and rock caverns with wide span and low rock cover, sprayed concrete embedded lattice girders have become a preferred method of rock reinforcement. These are an alternative to in situ-built reinforced arcs made of rebars (RSS) as described in the Q-method.

Lattice girders can be manufactured according to a theoretical profile and can therefore be mounted as an ideal load-bearing arch anchored to the rock mass with mounting bolts.

The lattice girders are made of rebars and is not corrosion protected. Sprayed concrete is applied to the lattice girders to establish a rigid beam and at the same time corrosion protect the steel during the lifetime of the rock support.

For the sprayed concrete to act as corrosion protection, cavities and shadows of air that will later become waterfilled must be avoided. This is to avoid corrosion on the rebars.

In the new Oslo Metro Fornebubanen, lattice girders are used as permanent support in station caverns with low rock cover. It has not been possible to find any NS or ES standard or any other recognised international control routine for how to test and document the homogeneity of such sprayed concrete arches.

The article describes how a routine for inspection was implemented and a procedure for how cavities should be filled to prevent corrosion in the long term. The article also describes what measures can be taken to ensure that lattice arches are fully embedded.

*Figure 1 - Fornebubanen, Cavern for Lysaker metro station.*

## INTRODUCTION

Fornebubanen is an agency in the City of Oslo that is carrying out the construction of a new metro line from Oslo city centre to Fornebu where Oslo's main airport was located until 1998. The area along the new Fornebu Line is growing rapidly and is being developed with many office buildings, technology companies and new residential areas. The metro line will be 7.7 km long and is laying below ground for its entire length. Overburdens vary between 6,5 m and 35 m with an average of about 20 meters depth.

The rock is mainly chalky shale with elements of tuberous lime. The tunnel is in urban areas with varying soil thickness and areas with running clay. In the short and long term, leaks into the tunnel can lead to subsidence in the soil masses and hence damage to buildings, real estate, and infrastructure. A general leakage target of > 4 litres/minute/100 m of tunnel has been set for tunnels and caverns. Pre-injection of the rock mass before excavation with mikrosement and colloidal silica is performed over the entire length of the tunnel.

Installed rock support is classified and designed according to the Q-method, and bolts and sprayed concrete constitute the permanent support in all caverns. The large rock caverns for the stations include sprayed concrete embedded lattice girders as part of the permanent rock support. The type of lattice girders is determined by the rock quality and cavern span. Therefore, both traditional in situ-built arches (RSS) as described in the Q-method and prefabricated lattice girders with triangular and rectangular cross sections.

Spaying of reinforced arches is demanding and dependent on good basic concrete, suitable accelerator and a skilled nozzleman. Spraying of arcs is carried out with alkali free accelerator and without steel fibre reinforcement. Unreinforced sprayed concrete flows better in between the reinforcing bars than a fibrous reinforced sprayed concrete.
Sprayed concrete arches are part of the permanent rock support together with rock bolts and sprayed concrete in various thickness on ensure 100 years of design life.

## THE CONCERN

Complete embedding in the arches is critical to achieve full corrosion protection of the reinforcement bars. Possible reinforcement corrosion will weaken the arches' load bearing capacity and thus rock stability in the rock caverns for the stations.

## STANDARDS AND GUIDELINES

European and Norwegian standards say little about the construction and control of sprayed concrete arches. The authors believe this is because permanent rock support with sprayed concrete in general and sprayed concrete arches in particular is considered as temporary rock support in Europe and elsewhere. The common design outside Scandinavia still is to install a full concrete lining as permanent support and water in-leakage control.

In NB Publication no. 7, edition August 2011, it includes a reference to the EN-NS standard, hence sprayed concrete arches shall be homogeneous and without voids.

*3.4.3 In terms of execution, reinforced structures made with sprayed concrete are basically to be regarded as other reinforced concrete structures, i.e. NS-EN 1992, NS-EN 206-1, NS- 13670 apply.*

In NB Publication No. 7, edition December 2022, the text with reference to NS-EN standards for reinforced structures has been removed. However, the guidelines indicate advice for how to succeed spraying on reinforces concrete arches clause 4.5.4.1.

1. *Before installing reinforcement, pits in the substrate should be filled up.*
2. *To avoid poor application, the reinforcement should be designed and mounted in the most "spray-friendly" construction possible, i.e. "open" construction where not all rebar joints are laid in the same area.*
3. *If several layers are to be reinforced, it may be sensible to spray and reinforce layer by layer.*
4. *The reinforcement should be firmly fastened and braced to prevent it from vibrating during spraying, thereby creating poor adhesion and pockets around the reinforcement.*
5. *To prevent concrete from building up on reinforcement as far as possible, efforts should be made to use a plastic concrete by using an alkali free accelerator, possibly also reducing accelerator consumption, increasing compressed air flow and reducing spray distance.*
6. *It should be sprayed alternately at an angle, under reinforcement, and with a small distance, to avoid shadows (inadequate filling) behind the reinforcement.*

*Figure 2 – AMW sprayed concrete equipment.*

## LYSAKER STATION EXECUTION

The rock cavern for Lysaker metro station is 200 m long and has a width of 24 m. The cavern is 18 m high and is excavated by traditional heading and bench principle in two steps. The station hall crosses under the Lysaker river with a rock cover of only 7m – 15 m. Two access tunnels, tunnel A 32 m2 and tunnel B 120 m2 will be blasted from the station cavern and into the shaft, which will be the future access to the platform. The station hall and the junctions for access tunnels are supported with prefabricated lattice arches embedded in sprayed concrete. The executing contractor is VEIDEKKE

Norway, which has high expertise and extensive experience with sprayed concrete and has experienced nozzle men.

The equipment used is state of the art AMV robotic spraying rigs.

## QUALITY CONTROL AND DEVIATIONS.

Before the work started, several QC meetings were held where the contractor presented his procedure for an execution that would provide complete enveloping of the reinforcement.

There is no Norwegian Standard, EU Standard, EFNARC Guideline or equivalent that confirm how quality control of sprayed concrete arches should be executed as far as the authors has been able to find. Nor has it been possible to find guidelines for how such cavities should be repaired so that reinforcement corrosion is avoided, and the load-bearing capacity of the arches is maintained for 100 years.

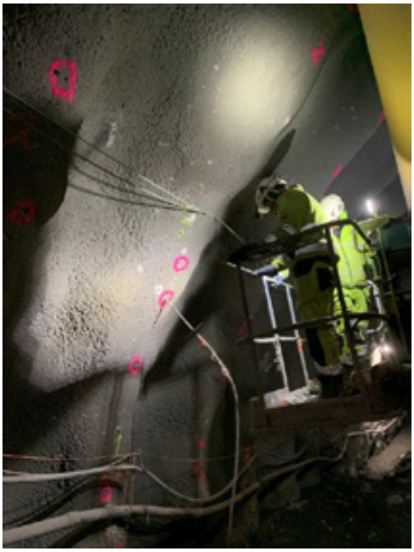

*Figure 3 – Void filling with cement grout.*

*Figure 4 Complex installation of lattice girders.*

**In agreement with the contractor, the following control procedures were implemented:**

1. The client performs quality control of lattice arches with test drilling to identify porosity and voids. Control hole drilling is done systematically on all arches.
2. Minimum 2 holes are drilled per 6 m arc section around the entire profile giving a distance between the control holes of 2 m. The control holes are 12 mm in diameter and 600 mm deep. The holes are checked with a keyhole camera with a diameter of 10 mm.
3. The client issues the non-conformity report to identify the checked arches in which control holes with cavities have been found.
4. The protocol is sent to the contractor as a basis for repairing and filling the voids.

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*Figure 5 – QC report with nonconformities listed.*

**The contractor's procedure for rectifying the nonconformities with voids in the lattice arches:**

Areas with large cavities detected are injected with cement-based injection.

1. For arcs where cavities have been found, several holes are drilled on each side of the control hole to find the extent of the cavity before starting to inject.
2. The drill hole diameter is extended up to 32 mm and packers are placed in all drilled holes, the upper hole act as an air evacuation hole.
3. The packers are 32 mm in diameter, and they are placed far enough in to get a firm grip in the concrete. Injection rods at 50 cm length are used.
4. Standard cement injection equipment with mixer, agitator and pump is used.
5. Injection starts in the lowest hole and seek spread to the nearest ventilation hole.
6. The injection material is micro cement with $D^{95} < 25$ microns with water/cement ratio of 0.8 – 0.9. Plasticizing additive at a dosage of 1% by weight of cement is added to the mix.
7. Applied injection pressure, max 7 bar to prevent the sprayed concrete from cracking.
8. The injection crew shall make use of a tunnel lift to get access to all areas in the arches with cavities.
9. A protocol with the injection parameters including date, hole number, pressure and material consumption shall be handed over to the client for approval.

Areas with smaller cavities are injected with epoxy-based injection materials.

1. For arcs where cavities have been found, several holes are drilled on each side of the control hole to find the extent of the cavity before starting to inject.
2. 14 mm holes are drilled for injection.
3. 13 mm injection packers with a grease-nipple connection are used. The packer is placed approximately 5 cm from the surface.
4. Injection epoxy consisting of resin and hardener with filler added shall be used to avoid damaging heat generation when curing in larger cavities.
5. Diaphragm injection pumps with a maximum capacity of 5 litres/min.
6. Always start in the lowest hole and seek spread to the nearest ventilation hole.
7. Applied injection pressure, max 7 bar to prevent the sprayed concrete from cracking.
8. The injection crew shall make use of a tunnel lift to get access to all areas in the arches with cavities.
9. A protocol with the injection parameters including date, hole number, pressure and material consumption shall be handed over to the client for approval.

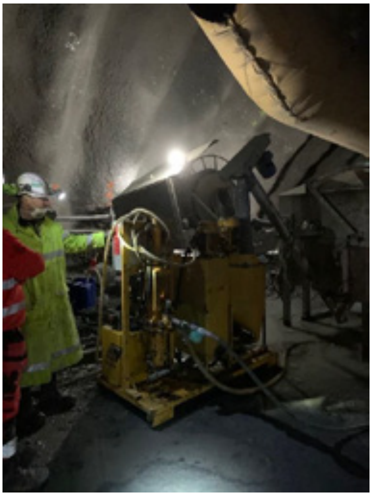

*Figure 6 – Pump set-up for cement grouting*

*Figure 7 – Pump set-up for epoxy grouting.*

**LESSONS LEARNED**

Several measures were taken to optimise spray execution.

1. Method statement seminar held with the nozzlemen before operations start.
2. AMV spraying rig with full flexibility for manoeuvring the nozzle.
3. Modified base concrete with higher dosage of plasticizing additive to maximize flowability.
4. Use a type of alkali free accelerator that provides good flow and better enveloping of the reinforcing bars in the lattice girders during spraying.

But despite the strong focus on spray execution, more cavities were uncovered by the control regime than expected. The inspection report was handed over to Veidekke and was the basis for drilling supplementary injection holes and vents.

Injection with cement requires relatively large equipment and has complicated cement logistics. Cement injection for such application is time-consuming. The cavities are narrow, and it is difficult to get a passage between injection holes and vents. Strength of injection mass with water - cement ratio of 0.9 has a compressive strength of < 20 MPa and will be the weakest material in the arc construction.

Injection with epoxy is done with small portable diaphragm pumps with a 5-liter material container. The injection goes easier and faster as epoxy material of this type does not contain particles. It also has a relatively short hardening time so that the risk of leaks is less. Such epoxy has a compressive strength of > 80 MPa and will also not impair the capacity of the structure.

**STATEMENT**

There is a need for clear guidelines for the execution of reinforced sprayed concrete arches. Current Norwegian and international standards do not provide clear rules for how sprayed concrete arches should be checked for cavities and shadows that may cause reinforcement corrosion and loss of sprayed concrete arches' securing capacity.

**REFERENCES:**

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## Carbonation of Wet-Mix Shotcrete for Ground Support

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### Abstract

When wet-mix shotcrete is used as the final liner in civil tunnels, carbonation will occur over the service life of the tunnel. The effect of $CO_2$ on carbonation penetration in wet-mix shotcrete has been studied in accordance with the European code EN13295 [Ref 5, 6]. Test results indicated that the carbonation depth varies with the dosage of accelerator used. Permeability of the shotcrete and wet-dry conditions of the environment are found to affect the carbonation depth in the shotcrete.

### Introduction

Carbonation of concrete refers to the process whereby $Ca(OH)_2$, which is a by-product of cement hydration, leaches out of the concrete/shotcrete during its service life. On exposure to $CO_2$ in the atmosphere, the $Ca(OH)_2$ reacts to form $CaCO_3$. This process results in reduced alkalinity of the cement matrix. In reinforced concrete structures, steel is in a state called passivation. At this state, steel forms a thin passivity layer of oxide which strongly adheres to the underlying steel and protects it from reaction with oxygen and water, i.e., from formation of rust or corrosion [Ref. 1]. Once the pH value has dropped from 12.6-13.5 for Portland cement paste to about 8.3-9 due to carbonation and depletion of $Ca(OH)_2$, the protective oxide film on the steel is removed and if oxygen and moisture penetrate into the cement paste, corrosion of steel could take place [Ref. 1]. Therefore, carbonation of Portland cement paste is important when a reinforced concrete/shotcrete structure is exposed to $CO_2$.

Most of the time, $CO_2$ comes from the air to which the structure exposed. Natural air contains about 0.03% $CO_2$ content. In large cities, the $CO_2$ content is on average about 0.3% and can go up to about 1% due to emissions by motor vehicles [Ref. 1]. When the environmental conditions increase or decrease exposure to $CO_2$, the carbonation process will increase or decrease accordingly. In the civil tunnel industry, a sprayed concrete liner is commonly constructed for ground support purposes and is normally considered as the primary liner for temporary support. Sprayed concrete, also called shotcrete in North America, is also used for the final liner. When a shotcrete final liner is constructed, it will be exposed to the atmosphere with $CO_2$ and carbonation is inevitable. In motor way tunnels, the final liner shotcrete may be exposed to higher concentration of $CO_2$ and therefore, the carbonation process may be aggravated.

The carbonation rate in concrete/shotcrete structures is a long term process and depends on the transportation properties of the concrete/shotcrete. The transport properties in shotcrete and the subsequent service life were studied previously [Ref. 2]. ***Transport properties*** include:

- *Absorption* (liquid uptake in a porous medium);
- *Diffusion* (liquid, gas or ion movement under a concentration gradient);
- Permeability (resistance to flow of a liquid under a pressure gradient);
- *Sorbtivity* (absorption of a liquid by capillarity); and
- *Wicking* (capillary transport through a porous medium to a drying surface)

Transport properties for shotcrete are dependent on a number of factors, including wet/dry conditions, temperature, mixture design and application. During the application process, control and addition of accelerator is found to greatly affect the transport properties [Ref. 2]. There is little information on the effect of the transport properties on the carbonation process in shotcrete. This paper presents studies on the carbonation resistance of shotcrete involving different dosages of accelerators, use of hand or robotic spraying processes, and curing conditions involving different ambient moisture conditions. The primary objective of the study is to provide testing data on the carbonation resistance for underground wet-mix shotcrete with variable factors.

This paper provides test data for carbonation resistance from several trial shoots conducted by the authors. The shotcrete mixture design information is included in another paper presented in conference [Ref. 3].

Carbonation rate of cement is dependent on many factors. According to Bamforth, the chemical buffering factor of the mix, defined as the equivalent $C_3A$ content in cement may be used as the primary determinant of carbonation rate in a given environment [Ref. 4]. The equivalent $C_3A$ content in cement is defined by the chemical buffering capacity of $b_1$. The buffering factor, $b_1$, is defined as 1.0 with an equivalent $C_3A$ content in cement of 12.8% as shown in Table 1. The lower the buffering factor, i.e., the lower the $C_3A$ content, the higher the carbonation rate.

*Table 1. $C_3A$ Content in Cement and the Buffering Factor [Ref. 4]*

| $C_3A$ Content | Buffering Factor |
|---|---|
| $C_3A$ = 12.8% | $b_1$ = 1.00 |
| $C_3A$ = 8.6% | $b_1$ = 0.85 |
| $C_3A$ = 1.6% | $b_1$ = 0.75 |

The cement used in the present wet-mix shotcrete study had a $C_3A$ content of 7%. This is lower than the 12.8% and 8.6% $C_3A$ content referred to above. Therefore, the cement has a buffering factor of less than 0.85 and will consequently result in a potential higher carbonation rate. It should be noted that the present study used the same basic wet-mix shotcrete mixture design with different types of fibers, and different dosages of the AFA accelerator [Ref. 3]. Therefore, test results are comparable though the overall values of natural carbonation are higher due to the low $C_3A$ content in the cement.

**Carbonation Test Setup**

The carbonation test was setup to the EN 13295 [Ref. 5] and the fib 34 (2010) Model Code Accelerated Test (56 days) [Ref. 6]. An environmental chamber was instrumented with:

Consistent temperature between 23-27 C
Consistent $CO_2$ concentration of 1.90-2.10%
Consistent relative humidity of 65+/-5%
The environmental chamber was sealed and a separator $CO_2$ sensor was attached to the outside of the chamber to detect any leaks for laboratory safety. A data acquisition system was used to monitor the $CO_2$ concentration, temperature and relative humidity (RH) and the test could be terminated should the $CO_2$ value exceeded the specified limits. Fig. 1 shows a plot of the temperature, RH and $CO_2$ concentration during the test.

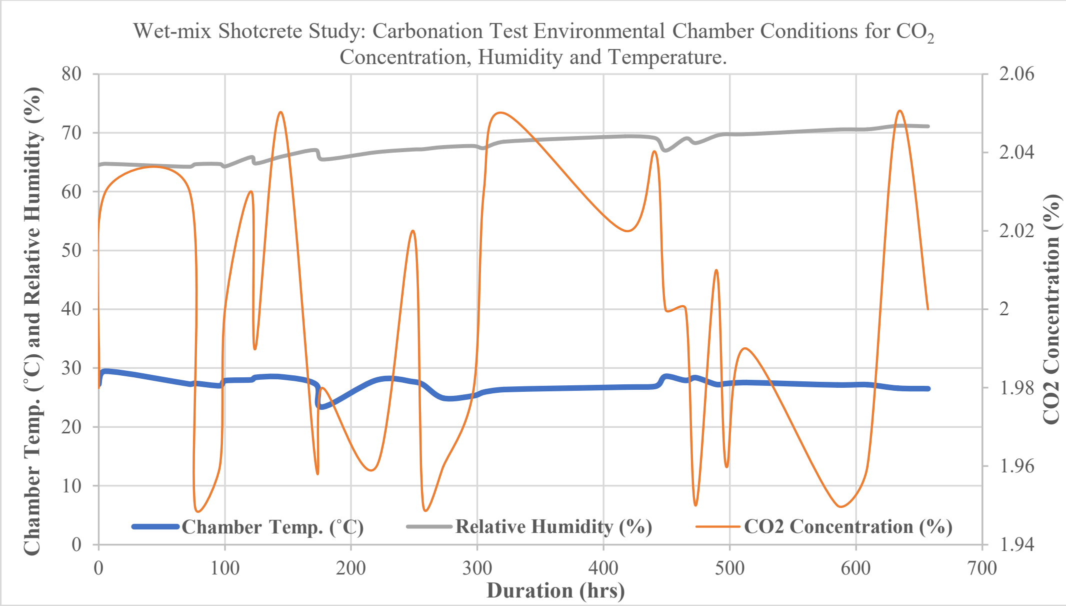


*Fig. 1 Environmental Conditions for Test: (Temperature, Relative Humidity) and $CO_2$ Concentration Values Recorded.*

**Effect of Relative Humidity during Curing**

The Fib test method requires concrete samples to be wet cured for 7 days, and placed in a dry condition for a minimum of 14 days. The shotcrete test panels were shot on site, and cut into beams with dimensions of 100x100x400 mm. Samples were wet cured in the laboratory conditions (23+/-2 C and RH of 100%) for 7 days, and then moved to the drying room with RH of 50% and temperature of 21+/-2C until 28 days. This is called the dry curing condition. Then samples were then placed into the environmental chamber with a controlled temperature of 25+/-2 C, RH of 65+/-5% and $CO_2$ concentration of 1.90% to 2.10%. During the dry curing condition, a comparative study was conducted by altering the curing condition from 50% RH to oven dry for 3 days prior to the test. The purpose was to study the effect of various dry curing conditions, i.e., 21 days at 50% and 21+/-2C vs. 18 days at 50% and 21+/-2C with 3 days at 0% and 100 C on carbonation resistance. Table 2 summarizes the sample preparation and curing procedures.

*Table 2. Sample preparation and testing procedures*

| Procedure | Age (days) | Activities |
|---|---|---|
| Sample Cast Date: | 0 | Samples covered in shrink wrap, wet burlap, and poly and left overnight in plastic containers on site |
| Samples in Water: | 1 | Samples stripped from formwork and placed in tap water at 21C +/- 2C |
| Dry Curing Conditioning: | 7 | Samples removed from water and placed in Shrinkage room at 21 +/- 2C @ 50% R.H. |
| Oven Drying: | 25 | Half of the samples placed in an oven for reduction to 0% R.H. moisture. |
| Start Date: | 28 | Samples placed into the Carbonation Chamber and start of Carbonation Test |

Test samples were kept in the carbonation chamber for 28 days, immediately after being cut, and sprayed with phenolphthalein. Carbonation penetration depth was measured and natural carbonation, $R_{NAC,0}^{-1}$ (mm$^2$/year)/(kg/m$^3$), was calculated. Natural carbonation, $R_{NAC,0}^{-1}$, is the inverse effective carbonation resistance of dry concrete (65% RH) determined at a certain point of time T0 on specimens with the normal carbonation test NAC [(mm$^2$/year)/(kg/m$^3$)] [Ref. 6]. Natural carbonation values of 6000-8000 (mm$^2$/year)/(kg/m$^3$) are generally specified for cast in place concrete.

*Fig. 2 Carbonation penetration depth being measured for wet-mix shotcrete*

*Table 3. Carbonation Resistance for Macrosynthetic Fiber Reinforced Wet-Mix Shotcrete under different dry curing conditions.*

| **Carbonation under different condition** | **50% RH for 21 days** | | **50% RH for 18 days and Oven Dry for 3 days** | | **Increases of Natural Carbonation with 3 days oven dry cure** |
|---|---|---|---|---|---|
| | Inverse Effective Carbonation Resistance | Natural Carbonation | Inverse Effective Carbonation Resistance | Natural Carbonation | |
| **Accelerator Dosage** | $R_{ACC,0}^{-1}$ (mm$^2$/year)/(kg/m$^3$) | $R_{NAC,0}^{-1}$ (mm$^2$/year)/(kg/m$^3$) | $R_{ACC,0}^{-1}$ (mm$^2$/year)/(kg/m$^3$) | $R_{NAC,0}^{-1}$ (mm$^2$/year)/(kg/m$^3$) | % |
| **0%** | 6031 | 7854 | 7615 | 9834 | 25% |
| **6.0%** | 8292 | 10680 | 13395 | 17060 | 60% |
| **9.0%** | 9348 | 12000 | 13697 | 17437 | 45% |

Table 3 shows the test results for carbonation resistance for the macrosynthetic fiber reinforced wet-mix shotcrete with different accelerator dosages. Test results show that the natural carbonation increases with increasing accelerator dosages from 0%, to 6% and 9%. Increasing accelerator dosages increases the porosity and voids in the shotcrete, and thus allow more $CO_2$ exposure to the cement hydration product including $Ca(OH)_2$, and therefore increase the carbonation reaction.

Table 2 also shows that with a higher temperature (oven dry) and 0% RH, the carbonation process accelerates and results in increased natural carbonation. Results show that the natural carbonation rate increases from 25% to 60% when samples are cured in the oven. These comparative results show that the RH (drying condition), together with changes in temperature will impact the carbonation resistance significantly.

**Carbonation for Steel Fiber Reinforced Wet-Mix Shotcrete with Different Accelerator Dosages**

Carbonation resistance for steel fiber reinforced wet-mix shotcrete is included in Table 4. Results for Natural Carbonation increase from cast wet-mix shotcrete, to shot wet-mix shotcrete without accelerator, to shotcrete wet-mix shotcrete with 4%, 6% and 8% AFA accelerator. This is consistent with the trend observed in Ref. 1 that transport properties are affected by the AFA accelerator dosage.

It should be noted that the natural carbonation for both cast and shot wet-mix shotcrete mixtures is below the generally accepted limit of 6000-8000 ($mm^2$/year)/($kg/m^3$) for concrete. Adding AFA accelerator increases the natural carbonation rate.

The shot wet-mix shotcrete exhibits higher natural carbonation than that observed with the cast shotcrete (CIP). This is likely related to the shooting process. The shooting was conducted with a robotic sprayer operated by an experienced nozzleman. The shooting process may include overspray and rebound being embedded into the test panels, and subsequently affect the transport properties.

*Table 4. Carbonation Resistance for Steel Fiber Reinforced Wet-Mix Shotcrete with different accelerator dosages.*

| **Mixtures Tested** | **Inverse Effective Carbonation Resistance** | **Natural Carbonation** |
|---|---|---|
| | $R_{ACC,0}^{-1}$ ($mm^2$/year)/($kg/m^3$) | $R_{NAC,0}^{-1}$ ($mm^2$/year)/($kg/m^3$) |
| **0% CIP** | 1573 | 2282 |
| **0% SHOT** | 4382 | 5793 |
| **4%** | 8166 | 10523 |
| **6%** | 9550 | 12253 |
| **8%** | 13749 | 17502 |

The boiled absorption and volume of permeable voids evaluate the capillary voids inside the cement paste, and the voids in the aggregates. When the aggregates have a low absorption rate, BA & VPV will be primarily dependent on the hydrated cement paste product. The transport properties of chloride ion penetration, ionic migration potentials, and absorption rates of the wet-mix shotcrete is dependent on BA and VPV [Ref. 1]. BA and VPV has been used as a performance requirement as well as a basic quality control parameter for durability considerations for shotcrete on many tunnel projects [Ref. 1, 7, 8, 9]. Generally, BA of 8% and VPV of 17% indicate good quality shotcrete, based on which the shotcrete structure is considered to be durable and have a long service life.

Fig. 3 shows the test results for boiled absorption and volume of permeable voids (BA & VPV) tested to ASTM C642. It shows that BA & VPV increases from cast wet-mix, shot wet-mix, to shot wet-mix with 4%, 6% and 8% AFA accelerator. This is consistent with the results from the carbonation test. It appears that the carbonation test results correlate with the BA & VPV values.

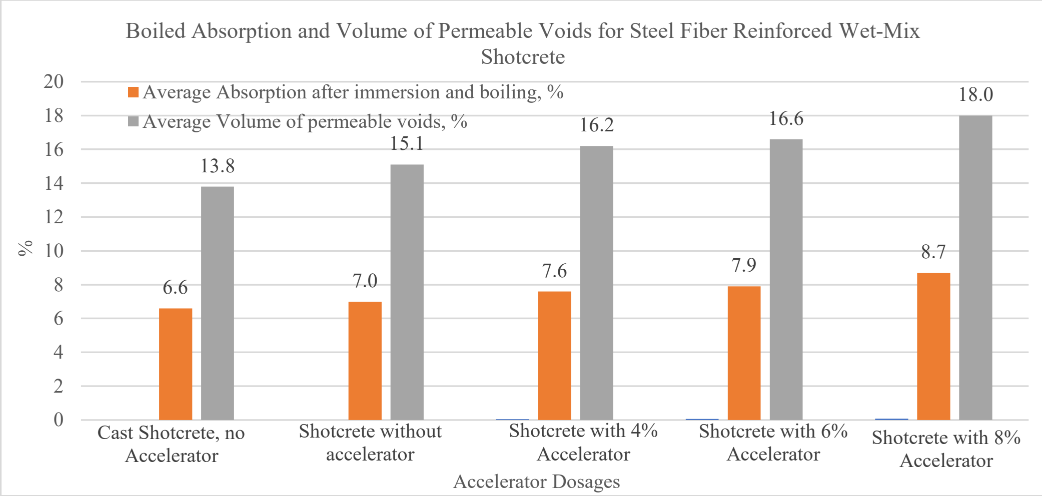


*Fig. 3 Boiled Absorption and Volume of Permeable Voids for Steel Fiber Reinforced Wet-Mix Shotcrete*

**Conclusions and Recommendations**

1. Shotcrete test panels were produced by casting and shooting with different dosages of AFA accelerator. These test panels were cut into beams, cured and tested as per EN 13295 procedures.
2. The carbonation depth was measured and natural carbonation was calculated based on the fib model. Results indicate that shotcrete with accelerator shows higher natural carbonation, and the natural carbonation increases with increasing AFA accelerator dosages. The BA and VPV properties are negatively affected by increasing dosages of AFA accelerator. Natural carbonation appears to be correlate well with the BA and VPV results observed in this study.
3. The tested natural carbonation for wet-mix shotcrete with both macrosynthetic fibers and steel fibers show similar behavior. This indicates that the natural carbonation for wet-mix shotcrete is independent on types of fibers.
4. The natural carbonation for cast wet-mix shotcrete and shot wet-mix shotcrete without AFA accelerator are consistent with the typical values observed for cast in place concrete, although the cast wet-mix shotcrete has a lower natural carbonation than that of shot wet-mix shotcrete. This shows that the shooting process, in particular, with robotic sprayer, could increase the natural carbonation process.
5. The present studies provide data on wet-mix shotcretes with different dosages of accelerators. The effects and correlations between transport properties and carbonation resistance are not fully studied in this report. Future studies will be conducted with tests on transport properties and carbonation resistance to establish a correlation between the transport properties and the carbonation resistance of shotcrete. Variables that affect the transport properties and carbonation resistance of shotcrete can then be used to predict carbonation resistance based on shotcrete transport properties such as BA & VPV.

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**Development of Wet-Mix Shotcrete with Portland Limestone Cement in North America**

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**Abstract**

Portland Limestone Cement (PLC) is now being specified and produced to replace Portland Cement (PC) across the USA and Canada. The most widely used cement was Type GU in Canada and Type I in the USA but now has become Type GUL in Canada and Type IL in the USA. PLC is increasingly being used in wet-mix shotcrete projects. Shotcrete made PLC has exhibited different properties of early age strength development, cement hydration development, compressive strength development, flexural toughness and residual tensile strength development as well as having an impact on pumping and shooting compared to shotcrete made with PLC. This paper presents properties of wet-mix shotcrete made with PLC from some tunnel projects in North America.

## 1 Introduction

In Canada, Type GUL cement refers to General Use (GU) cement with addition of up to 15% limestone. Limestone is added to the clinker and ground together during the cement production process. In the USA, Type IL cement refers to Type I cement with up to 15% limestone addition. For discussion purposes, Type GUL cement is used in this paper for both Type GUL and Type IL cement. Cement with up to 15% limestone content was introduced into concrete construction in the USA and Canada about a decade ago when CSA A23.1/23.2 listed Type GUL cement back in 2014. Adding up to 15% limestone into the cement reduced the usage and production of cement, and as a result, reduced energy consumption and greenhouse gas emissions. Type GUL cement is now widely used in concrete production and is the primary cement used in the building and construction industry in North America.

Since 2023, the major cement suppliers in North America have minimized the production of Type GU cement and only Type GUL cement is commercially available in most of the provinces in Canada and the states of the USA. The changes in cement production and supply were rapid and left no choice for the construction industry. For the civil tunnel industry, this change caused many challenges as contractors did not have much experience in working with Type GUL cement in concrete and shotcrete. Although suppliers have conducted research and provided some project experience for concrete using Type GUL cement, project experience for the use of Type GUL cement in wet-mix shotcrete for ground support, in particular, with accelerators and fibers addition is relatively new to the tunnel construction industry. This paper discusses the properties of shotcrete made with Type GUL cement when used in wet-mix shotcrete. Most importantly, it provides testing data related to early age and later age compressive strength development, residual tensile strength, and experience with shotcrete application during construction.

**1.1 Code development:** Since early 2010s, the Canadian Standard Association (CSA), American Concrete Institute (ACI), American Standard of Testing and Materials (ASTM), Portland Cement Association (PCA), and other industry technical authorities have started to require limestone to be added into the cement to reduce the usage of Portland Cement. Limestone is added into the clinker and ground to a fineness that is suitable for optimal cement hydration. Since 2016, CSA A3000 Cementitious Materials, CSA A23.1/23.2, Concrete Making materials and methods of concrete construction/Test methods and standard practice for concrete have added a requirement that Type GUL cement should include up to15% limestone. ASTM C150 also requires that Type IL cement include 15% limestone. Type IL cement is already listed as an approved material in ACI 301, ACI 318 and P501 for use. Type GUL/IL cement is typically ground slightly finer than conventional Portland Cement (PC) (450-500 versus 350-420 Blaine Fineness $m^2$/kg).

**1.2 Blaine fineness:** When limestone was originally introduced into Type GU cement at a replacement rate of up to 15%, it was found that concrete made with Type GUL/IL cement takes a longer time to set, early age compressive strength development is slow, and the heat of hydration is reduced. These changes received mixed feedback from the industry. The delayed set time and early age compressive strength development extended the construction schedule, while the reduced heat of hydration is beneficial for the thermal control for structural elements, such as walls and in particular, mass concrete/shotcrete structures. The clinker was then ground finer and the Type GUL cement now has an increased fineness compared to conventional PC. This helps to improve the early age compressive strength development for wet-mix shotcrete made with Type GUL cement.

| Item | Spec limit | Test Result |
|---|---|---|
| Blaine Fineness (m2/kg) | --- | 410 |
| Retained 45 um (%) | 28 max | 1.4 |

| Item | Spec limit | Test Result |
|---|---|---|
| Air content of mortar (%) (C 185) | --- | 6.1 |
| Blaine Fineness (m2/kg) | --- | 455 |
| Passing 45 um (%) | 72 min | 99.0 |

*Table 1. Blaine Fineness for Type GU cement (left) and Type GUL cement (right)*

**1.3 Impact on compressive strength:** Type GUL cement was reported to have a slightly lower early age compressive strength but improved later age compressive strength, such as at 56 days and 91 days. This may not be beneficial for wet-mix shotcrete used for ground support as it requires higher early age compressive strength. Later age compressive strength is generally satisfactory as the 28 days compressive strength can usually meet the specified compressive strength requirement. It is worth testing compressive strength at later ages to see if shotcrete made with Type GUL cement meets the later age compressive strength requirement, particularly, when used with an alkali-free accelerator (AFA).

**2 Wet-Mix Shotcrete Mixture Design with Type GUL Cement:**

Table 2 shows an example of a wet-mix shotcrete mixture design with Type GUL Cement. The total cementitious materials content of cement, silica fume, fly ash and slag is about 450 kg/cu.m. Sometimes, the total cementitious materials content is increased to facilitate pumping and increase both the early and later age compressive strength development. Silica fume is commonly used as supplementary cementitious material (SCM) for wet-mix shotcrete in tunnels. Silica fume is normally added at 5-10% by mass of total SCM with 8% being an optimum percentage. Fly ash, when used, is added at up to 15% by mass of the total SCM. Slag, when used, is commonly added at up to about 40% by mass of the total SCM [Ref. 2], although it may be added up to 70% in mass shotcrete structures [Ref. 3].

*Table 2. Typical Wet-mix shotcrete design for ground support:*

| **Material** | **Mass per cu.m SSD Agg, [kg]** |
|---|---|
| Cement Type GUL | 365 |
| Silica Fume | 40 |
| Slag/Fly Ash | 55 |
| Coarse Aggregate (14-5 mm, SSD) | 430 |
| Fine Aggregate (SSD) | 1320 |
| Estimated Water, L | 185 |
| Superplasticizer, L | 1 |
| Air Content (As-batched 5-8%) | 4% (As-shot) |

For wet-mix shotcrete application, reaction between the cementitious materials and the AFA accelerator is critical for early age compressive strength development. SCMs of fly ash, slag and silica fume do not directly react with the AFA accelerator. Therefore, the total SCMs used in the wet-mix shotcrete should be designed with consideration of the reactivity with the AFA accelerator. A trial shoot and test of early age compressive strength is always necessary prior to construction with wet-mix shotcrete.

### 3. Plastic Properties

Wet-mix shotcrete made with Type GUL cement has been found to exhibit similar workability during pumping to wet-mix shotcrete made with Type GU cement. However, it was observed that mixtures made with Type GUL cement tend to bleed at higher slumps, such as 190-200 mm and above. This becomes more prominent when no silica fume is added. With silica fume added, the mixture tends to be more cohesive and shows no signs of bleeding even at higher slumps of up to 220 mm.

Pumpability is generally not impacted significantly by using the Type GUL cement. As the particle size of the Type GUL is finer, with Blain fineness of 450-500 $m^2/kg$, adding larger particles including fly ash or slag is always beneficial to fill the gap of size distribution between the fine aggregate and the Type GUL cement powder.

### 4 Early age compressive strength development

**Early age compressive strength** is one of the most important performance requirements for ground support in tunnels and mines. Generally, an early age compressive strength of 1.0-2.0 MPa is required for the applied shotcrete to facilitate construction activities. Alkali free accelerator (AFA) is commonly added at the nozzle at dosages required to achieve suitable early age compressive strength development. Since AFA was introduced into North American, it has been found that it generally takes an AFA dosage of about 6% to 8% by mass of cement to reach the required strength in about 1-3 hours [Ref. 1]. Variations in the early age compressive strength development are dependent on the shotcrete mixture design including the type of cement and chemical admixtures used, accelerator brand and performance, shotcrete temperature, ambient temperatures, proper handling and dispensing of accelerator and shooting skills of the shotcrete nozzlemen.

When Type GUL cement is used to replace Type GU cement, the early age compressive strength development is also affected by the type of cement. During recent projects, the author has tested early age compressive strength with a needle penetrometer and end beam tester. Figs. 1 and 2 shows typical early age compressive strength development with AFA dosage when plotted against the J1-J2-J3 curve template developed by the Austrian Concrete Society. J2 is generally regarded as the minimum performance requirement for shotcrete early age compressive strength development for most ground support projects. For some ground conditions including soft ground, a modified J2 is specified to allow a slightly higher early age compressive strength to provide sufficient ground support.

Fig. 1 shows the early age compressive strength development of wet-mix shotcrete with macrosynthetic fiber. The early age compressive strength with 5.6% accelerator appeared not to meet the J2 requirement. At a dosage of 7.0% AFA, the early age compressive strength marginally exceeded the J2 requirement. The early age compressive strength increases with increasing accelerator dosages of 7.9% and 8.3%. This shows that a minimum of 7.0% AFA is needed to achieve the J2 requirement for the early age compressive strength development.

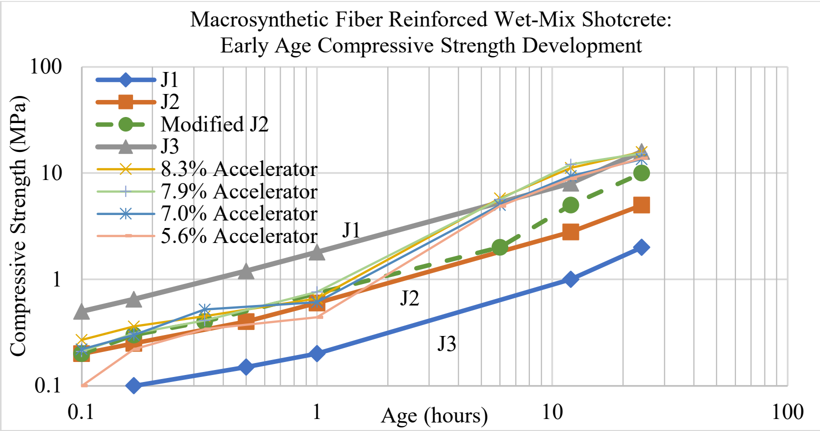


*Fig. 1 Early Age Compressive Strength Development for Macrosynthetic Fiber Reinforced Wet-Mix Shotcrete*

Fig. 2 shows the early age compressive strength development of wet-mix shotcrete with steel fiber. The early age compressive strength with 8.0% accelerator appeared to exceed the J2 requirement with a good margin of safety.

Early age compressive strength development in both Fig. 1 and Fig. 2 increases at a similar slope as the J2 curve for up to 3-4 hours, and then increases at a much faster pace than the J2 curve after 6 hours, and actually meets/exceeds the J3 curve. This shows that the compressive strength develops faster after 4-6 hours. The early age strength development for steel fiber reinforced wet-mix shotcrete and the macrosynthetic fiber reinforced wet-mix shotcrete are similar.

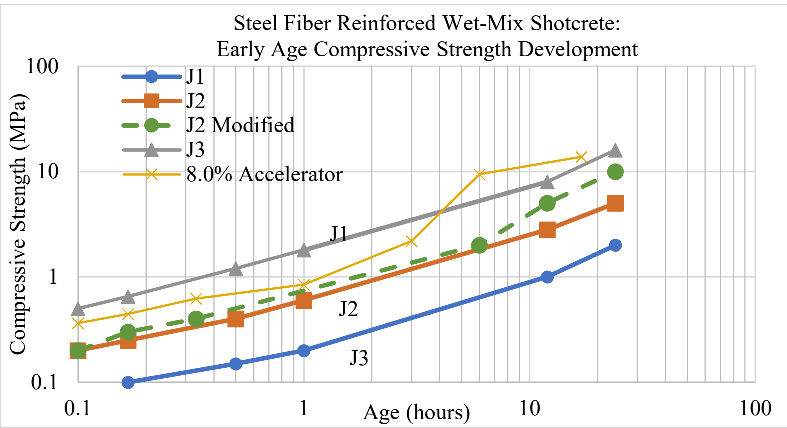


*Fig. 2 Early Age Compressive Strength Development for Steel Fiber Reinforced Wet-Mix Shotcrete*

In summary, the trial batch shoot and test results for both macrosynthetic fiber and steel fiber mixes showed that the wet-mix shotcrete with Type GUL cement can meet the J2 curve requirement with dosages of 7.0% by mass of cement. This appears to be slightly higher than the 6.0% AFA commonly used in the wet-mix shotcrete with Type GU cement.

## 5. Compressive strength development

Compressive strength for wet-mix shotcrete is normally specified at 3, 7 & 28 days. Shotcrete test panels are produced and cured for 2 days. Cores are extracted from the test panels and delivered to the testing laboratory where they are stored at standard laboratory curing conditions with 100% moisture and 23+/-2 C temperature. Compressive strength testing for shotcrete cores is specified in the ASTM C1604 standard.

Compressive strength development for wet-mix shotcrete is dependent on factors such as: water:cementitious ratio, cementitious materials type and content, as-shot air content, age of testing, consolidation of shotcrete, curing and handling of test panels, and most importantly, the dosage of AFA accelerator (Ref. 1).

Figure 3 shows the compressive strength development for macrosynthetic fiber reinforced wet-mix shotcrete. Compressive strength increases with ages from 1 day, to 7 and 28 days. At each age, compressive strength decreases with increasing dosages of AFA accelerator. Without accelerator, i.e., the cast shotcrete mixture, compressive strength can reach 47 MPa at 28 days, but will reduce to 36 MPa with AFA accelerator of 8.3%. It should be noted that the cast shotcrete has an as-batched air content of 6.6% and an as-shot air content of 4.3%. The reduction in the air content generally increases the compressive strength. Therefore, the increase of the compressive strength due to air content reduction is offset by the addition of the accelerator.

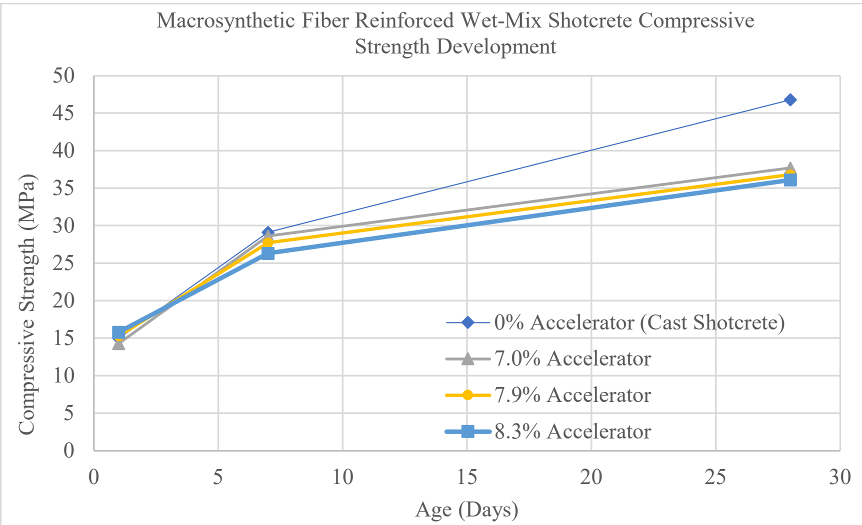


*Fig. 3 Compressive Strength Development for Macrosynthetic Fiber Reinforced Wet-Mix Shotcrete*

Figure 4 shows the compressive strength development for steel fiber reinforced wet-mix shotcrete at 7, 28, 56 and 91 days. The compressive strength reached 41 MPa for cast shotcrete without AFA accelerator. With 4% AFA accelerator, the 28 days compressive strength is close to the 43 MPa. Compressive strength results for both cast shotcrete and shotcrete with 4% AFA accelerator are close at age of 56 and 91 days. The as-batched air content was 8.6% and the as-shot air content was 4.9%. Typically, reduction in air content during shooting increases the compressive strength. Shotcrete with 4% AFA accelerator has a higher compressive strength than cast shotcrete. This is likely due to the reduced air content in shotcrete vs. high air content in the cast shotcrete.

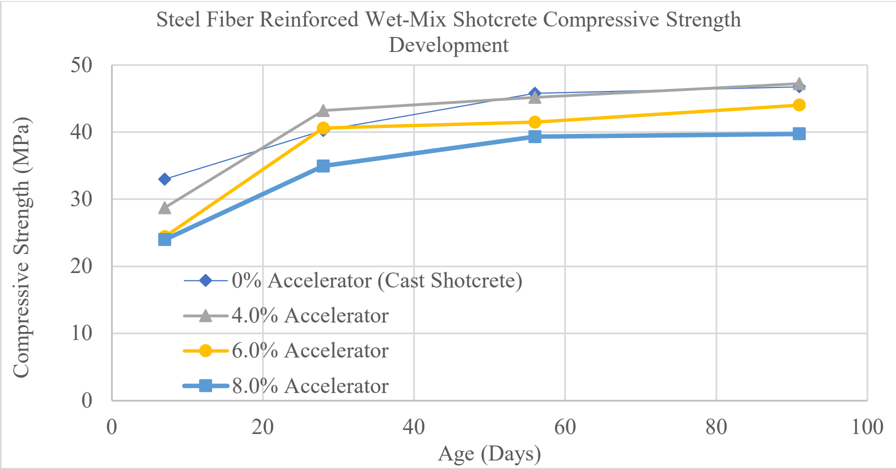


*Fig. 4 Compressive Strength Development for Steel Fiber Reinforced Wet-Mix Shotcrete*

Figure 4 shows that the compressive strength increases significantly from 7 to 28 days, but much less from 28 to 56 days, and just slightly increases from 56 to 91 days. This shows that although compressive strength results at 91 days are expected to be higher than those at the 28 days, the increase is minimal. Therefore, regardless of the contribution of the late age compressive strength by the Type GUL cement, 28 days compressive strength should be specified for the wet-mix shotcrete with AFA accelerator

Figure 4 also shows that compressive strength reduces with increasing AFA accelerator dosages for up to 8.0%.

Reduction of compressive strength with increasing AFA accelerator dosage is commonly seen in wet-mix shotcrete with AFA accelerator [Ref. 1]. Therefore, the effect of AFA accelerator on compressive strength for wet-mix shotcrete with Type GUL cement is consistent with the effect on wet-mix shotcrete with Type GU cement.

## 6. Fiber reinforced wet-mix shotcrete

Nowadays, macro fibers, including most commonly used macrosynthetic fiber and steel fiber, are added into the wet-mix shotcrete for ground support. Fiber reinforced wet-mix shotcrete, when used with rock bolts, has become the predominant method of ground support in tunnels and mines across the USA and Canada. When added into the shotcrete, fibers contribute to the crack resistance after shotcrete cracks, i.e., minimizes cracks from widening by fiber deformation and elongation, fiber pull-out and fiber fracture. Due to the addition of fibers, shotcrete is able to bear the load after shotcrete cracks and absorb energy as cracks propagate. Performance of fiber reinforced wet-mix is characterized by the energy absorbed, also called flexural toughness, or residual tensile strength during the post crack period.

### 6.1 Residual tensile strength development

Residual tensile strength testing to the BS EN14651 standard is gradually being used in the civil tunnel industry in the USA and Canada. The residual tensile strength test is conducted for beams with dimensions of 150x150x550 mm. A notch, at depth of 25 mm, is cut in the center of the tension face of the beam as indicated in Figure 5. The beam is therefore precracked and then loaded in three-point bending. The deformation of the beam is recorded as the opening of the crack at the bottom notch, i.e., crack mouth opening displacement (CMOD). The test sample is loaded with a closed-loop testing machine and the load vs. CMOD curve is recorded during the test (Fig. 6).

*Fig. 5 Fiber Reinforced Wet-Mix Shotcrete Notched Beam tested to BS EN14651*

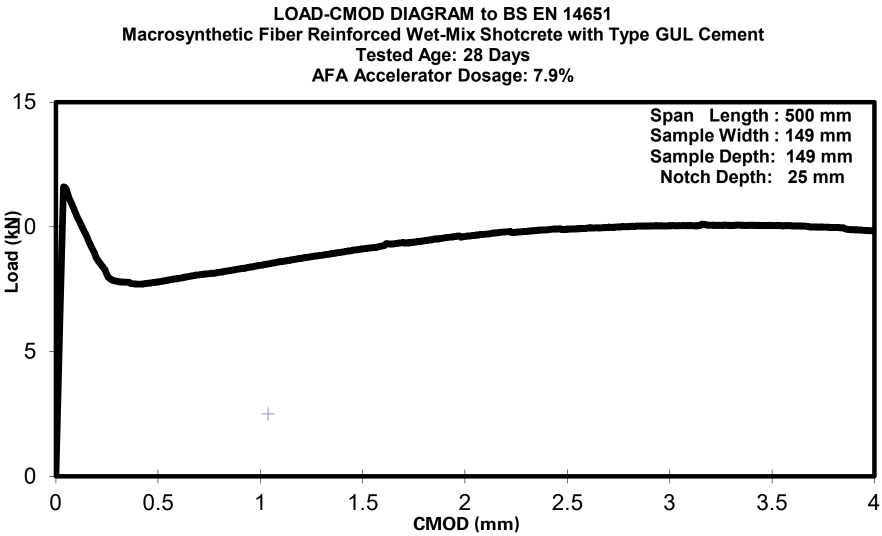


*Fig. 6 Load vs. CMOD for Fiber Reinforced Wet-Mix Shotcrete Notched Beam tested to BS EN14651*

Wet-Mix shotcrete with macrosynthetic and steel fibers were trial shot and tested. Residual tensile strength tests were conducted to the BS EN 14651, i.e. the notched beam test method. Fig. 7 shows the residual tensile strength results for macrosynthetic fiber reinforced wet-mix shotcrete and Fig. 8 shows the residual tensile strength results for steel fiber reinforced wet-mix shotcrete.

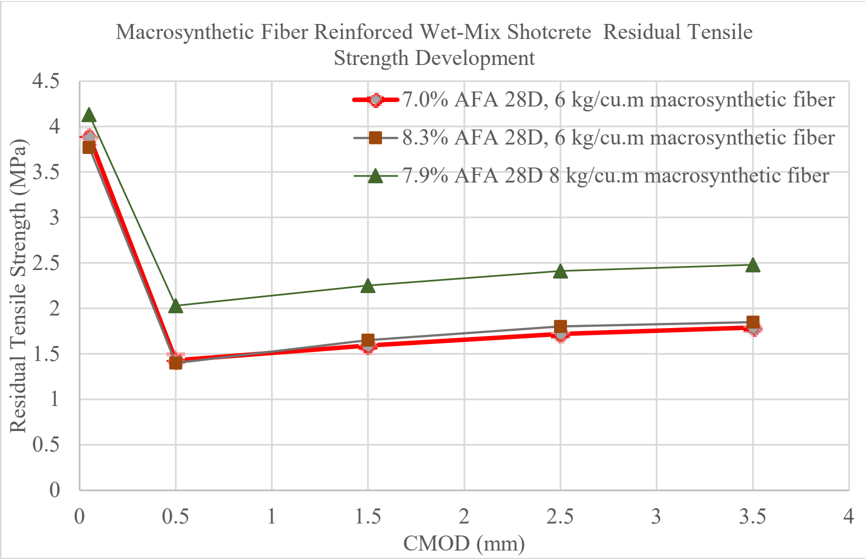


*Fig. 7 Residual Tensile Strength for Macrosynthetic Fiber Reinforced Wet-Mix Shotcrete tested to BS EN14651*

Fig. 7 shows the results of residual tensile strength for **macrosynthetic fiber** at a dosage of 8 kg/cu.m with an AFA accelerator and dosage of 7.9% and 6 kg/cu.m macrosynthetic fiber dosage with an AFA accelerator dosage of 8.3% and 7.0% respectively. Results of Limit of Proportionality (LOP), i.e., the peak tensile strength at CMOD < 0.05 mm, and residual tensile strength at CMOD of 0.5 mm, 1.5 mm, 2.5 mm and 3.5 mm are plotted in Fig. 7. Each data point is the average of 9 beams tested at age 28 days. Therefore, these results are statistically representative of a large sample population.

Fig. 7 shows that wet-mix shotcrete with higher macrosynthetic fiber dosage of 8.0 kg/cu.m resulted in higher residual tensile strength, including 2.5 MPa at CMOD of 3.5 mm, while it exhibited lower residual tensile strength of about 1.8 MPa with macrosynthetic fiber dosage of 6.0 kg/cu.m. The ratio of residual tensile strength vs. dosage of fiber is about 0.3 for the wet-mix shotcrete with both fiber dosages. The most commonly specified residual tensile strength for ground support is about 2.0-2.4 MPa at CMOD of 3.5 mm. Therefore, a macrosynthetic fiber dosage of 8 kg/cu.m is generally recommended for wet-mix shotcrete with AFA accelerator for ground support purposes. This has been consistently observed by the authors with several hundreds of beam residual tensile strength tests for wet-mix shotcrete with several high performance macrosynthetic fibers in major tunnel projects across the USA and Canada.

Fig. 7 also shows that wet-mix shotcrete with fiber content of 6.0 kg/cu.m but with two dosages of AFA accelerator exhibited close performance of residual tensile strength. This shows that although dosages of AFA accelerator affects the early age compressive strength and compressive strength at 7 & 28 days, it has minimum impact on residual tensile strength.

Wet-mix shotcrete with macrosynthetic fibers tends to have a lower load bearing capacity at lower CMOD, such as 0.5-1.0mm, but then increases at larger CMOD of up to 4 mm. This shows that macrosynthetic fibers tends to bear higher loads at larger CMOD, i.e., larger deformations. This makes the macrosynthetic fiber beneficial for higher deformations related to larger ground movement.

**Steel fibers** were trial shot and tested at dosages of 40, 50 and 60 kg/cu.m and AFA accelerator dosage of 8.0%. Results of Limit of Proportionality (LOP), i.e., the peak tensile strength at CMOD < 0.05 mm, and residual tensile strength at CMOD of 0.5 mm, 1.5 mm, 2.5 mm and 3.5 mm are plotted in Fig. 9. Each data point is the average of 9 beams tested at age 28 days. Therefore, these results are statistically representative of a large sample population.

A typical load vs. CMOD curve is plotted in Figure 8. It shows that the load bearing capacity decreases slightly after the LOP, but increases at low CMOD, such as less than 0.5 mm. The load bearing capacity then increases to the peak value at a larger CMOD, such as at about 1.0 mm as shown in Fig. 8. This peak load value is higher than the load of LOP. This is also called a strain hardening effect, i.e., the post peak load bearing capacity (residual tensile strength) exceeds the load when the beam first cracks (LOP). This strain hardening behavior in the fiber reinforced wet-mix shotcrete is very beneficial for ground support as it provides confidence for using fiber reinforced wet-mix shotcrete to provide sufficient ground support even when shotcrete cracks occur. The strain hardening behavior occurs with high performance fibers. It is dependent on fiber dosages, strength, types and bond with the shotcrete. It is also related to the shotcrete process, including equipment, mixture design, and most importantly, nozzleman's shooting skills (Ref.1).

Fig. 9 shows the results of LOP and residual tensile strength for wet-mix shotcrete with steel fibers at dosages of 40, 50 and 60 kg/cu.m. It clearly shows that the residual tensile strength increases with increasing fiber dosages. Fig. 9 also shows the the load bearing capacity at first crack, i.e., LOP, increases with increasing fiber dosages from 40, to 50 and 60 kg/cu.m.

Fig. 9 shows that wet-mix shotcrete with steel fiber at dosages of 50 and 60 kg/cu.m exhibit strain hardening behavior, i.e., post crack residual tensile strength is higher than the tensile strength at first crack (LOP). Wet-mix shotcrete with steel fiber at dosage of 40 kg, however, does not have a significant strain hardening behavior.

Wet-mix shotcrete with steel fibers tends to have higher load bearing capacity, i.e. residual tensile strength, at lower deformations, i.e., small CMOD. The residual tensile strength increases to a peak value with increasing CMOD of 1 mm, and then decreases at larger CMOD of up to 4 mm. This shows that steel fiber is beneficial for lower deformations, i.e., small ground movement.

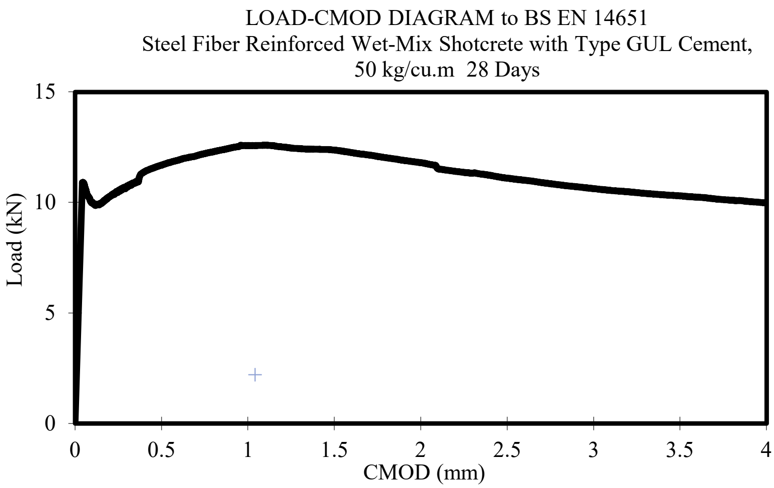


*Fig. 8 Load vs. CMOD for Steel Fiber Reinforced Wet-Mix Shotcrete Notched Beam tested to BS EN14651*

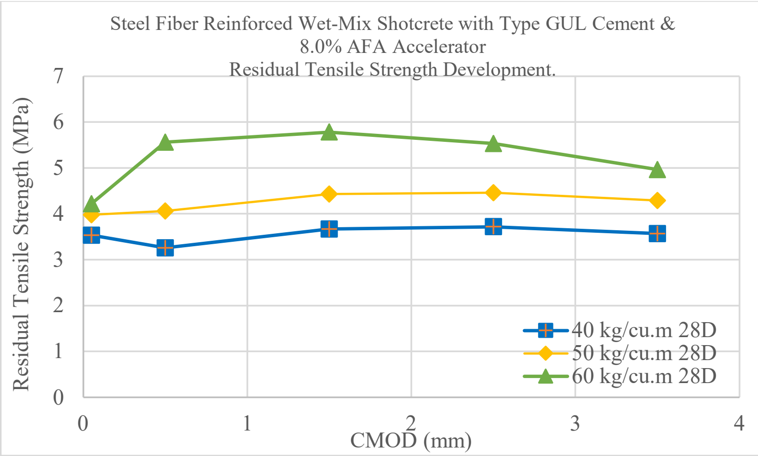


*Fig. 9 Residual tensile strength test for steel fiber reinforced wet-mix shotcrete*

The residual tensile strength behavior of wet-mix shotcrete using Type GUL cement is consistent with observations of wet-mix shotcrete with Type GU cement. Characteristics of wet-mix shotcrete with both steel fibers and macrosynthetic fibers are consistent with what has been observed by the authors [Ref. 1].

## Conclusions

Type GUL cement is ground finer than conventional Type GU cement. This results in increased early age compressive strength development but increased heat of hydration.

When used in wet-mix shotcrete, Type GUL cement requires a slightly higher dosage of the AFA accelerator in order to achieve the desired early age compressive strength for ground support, i.e., J2 curve.

Compressive strength for Wet-mix shotcrete with Type GUL cement is affected by the AFA accelerator dosage. Compressive strength reduces with increasing the AFA accelerator dosages.

Wet-mix shotcrete with Type GUL cement exhibited similar behavior to that made with Type GU cement for residual tensile strength development for both steel fiber and macrosynthetic fiber.

In summary, Type GUL cement is becoming the most widely used cement in wet-mix shotcrete for ground support in North America. Besides requirements for increases in dosage of the AFA accelerator, properties of compressive strength and residual tensile strength are similar to that of wet-mix shotcrete with Type GU cement. As more and more project experience has become available, Type GUL cement will become the normal cement to be used in wet-mix shotcrete projects in North America.

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