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**Efectis Nederland report**

**2009-Efectis-R0358**

# Investigation of concrete cracking due to fire in scaled immersed tunnel elements: analysis of test results

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# Contents

# 1 Investigation

An investigation of fire in immersed scaled tunnel elements was reported in Efectis report 2008-Efectis-R0146, based on nine fire tests which were carried out in 2008.

In this report the results of the fire tests are systematically compared in order to investigate the influence of each investigated parameter on possible cold-side cracking of fire exposed concrete tunnels.
Moreover a theoretical discussion of the underlying mechanisms is given, which is partly based on the Master's thesis of Ben Nieman, who graduated as a civil engineer at Delft University of Technology in 2008 after analysing the test results on the immersed scaled tunnel elements.
Finally, an outlook is given of potential remedies and suggestions are given on how to proceed towards a solution of the cold-side cracking problem.

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# 3 Overview of tested elements

Nine fire tests were carried out, on segments which varied in presence and type of polypropylene fibres, fire protection of the concrete, fire temperatures and vertical loading on the tunnel roof. In table 3.1 an overview of all tested segments is given.

Table 3.1 Overview of the tested elements

| Element | Test date | Fibre ∅/L | Ceiling and wall protection | High/low fire curve | Load | Cubic strength [N/mm$^2$] |
|---|---|---|---|---|---|---|
| 0 | 07-03-2008 | - | - | high | - | n.a. |
| 6 | 18-06-2008 | 18 µm/6 mm | X | high | - | 38.6 |
| 10 | 20-06-2008 | 32 µm/12 mm | X | high | - | 46.0 |
| 8 | 23-06-2008 | 18 µm/12 mm | - | high | - | 34.3 |
| 6 loaded | 27-06-2008 | 18 µm/6 mm | X | high | perpendicular to tube | 38.6 |
| 2 | 03-07-2008 | - | - | high | - | 47.4 |
| 9 | 04-07-2008 | 18 µm/12 mm | - | low | - | 37.7 |
| 5 | 08-08-2008 | 18 µm/6 mm | - | low | perpendicular to tube | 47.3 |
| 7 | 12-08-2008 | 18 µm/6 mm | - | low | in line with tube | 41.3 |

The fire curves are marked “high” and “low”. Moreover, a concrete element protected by a fire protection board will be effectively exposed to an even lower fire curve. The temperatures to which the concrete surface was exposed during the fire tests were roughly as shown in figure 3.1.

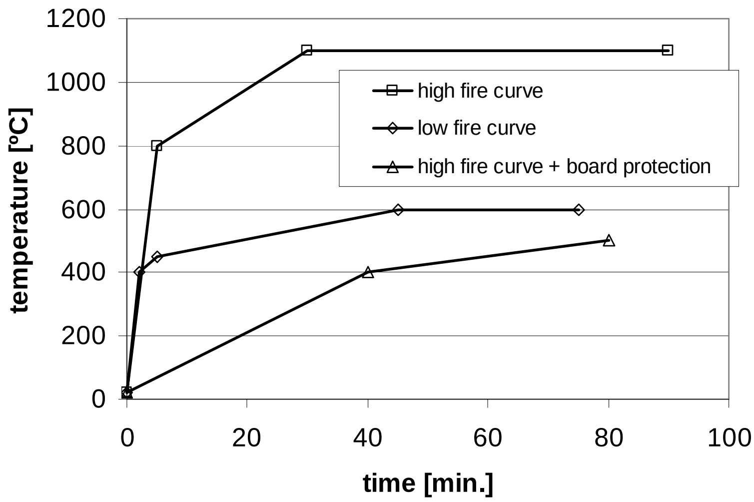


*Figure 3.1: effective fire temperatures*

# 4 Parameter study

The test plan has been chosen in such a way that the influence of a number of parameters can be evaluated by comparing two individual test results. The condition is that the two individual tests are identical except for the investigated parameter. The possible combinations which can be compared directly are shown in table 4.1.

Table 4.1 Possible comparison of test result for study of a certain parameter (columns and rows represent concrete segment numbers, the parameter is mentioned in the appropriate cell, between brackets the paragraph number in which this comparison is discussed)

| | **10** | **9** | **8** | **7** | **6-2** | **0** |
|---|---|---|---|---|---|---|
| **2** | | | with or without fibres (4.4) | | | age of the concrete (4.1) |
| **5** | | with or without loading (4.3) | | method of vertical loading (4.3) | heating rate (4.2) | |
| **6-1** | fibre type (4.4) | heating rate (4.2) | heating rate (4.2) | | with or without loading (4.3) | |
| **8** | | heating rate (4.2) | | | | |

Please note that test 6-1 was carried out on segment 6, without vertical loading. Nine days later, the segment was turned upside down and another test, 6-2, was carried out on the other tunnel tube, with a vertical loading.

The varied parameters are:

- Age of the concrete (only in one case, comparing a one month old segment with a four months old segment)
- Heating rate
- Vertical loading
- Addition of PP-fibres

Production and testing of concrete segments always results in some (unintended) variations between segments, even if it is aimed to produce segments with identical properties. Therefore it is necessary to assume that some variations are negligible, in order to be able to make a comparison. The following variations were not further investigated:

- No distinction of the age of the concrete segments was made if the concrete segments were at least approx. 3 months old. Only in one case, a one month old segment was compared with a four months old segment.
- No distinction of concrete compressive strength was made. With all tested segments, cubes were cast and tested after 28 days, always resulting in strength values between 34 and 47 N/mm$^2$.
- No distinction of PP-fibre lengths was made. The fibres used had lengths of 6 and 12 mm. From previous experience of Efectis, it is known that the length of the PP-fibres does not influence their effectiveness against spalling of concrete.

The test results were compared on the following aspects:

- crack pattern;
- crack width;
- vertical deflection (defined as the vertical displacement of the roof at mid span relative to the vertical displacement of the roof above the walls);
- and sometimes there were other clear effects, such as water leakage or spalling of concrete.

The evaluation of the crack widths could not be done when testing the loaded segments, because it was not considered safe to approach the segment during the fire test when 7.5 tons of dead weight were stacked on top of it.

## 4.1 Age of the concrete

To investigate the effect of the age of the concrete, the two test results of segments 0 and 2 can be compared.

<u>Varied parameter:</u>

| | |
|---|---|
| Concrete age: | segment 0: 28 days |
| | segment 2: 133 days |

<u>Constant parameters:</u>

| | |
|---|---|
| Heating rate: | high fire curve |
| PP-fibres: | none |
| Loading: | none |

The test result of segment 0, with an age of only 28 days, shows spalling of concrete, especially on the fire exposed surface the tunnel walls, and locally on the ceiling. Segment 2, with an age of 133 days, didn’t spall significantly.
Also, during the test on segment 0, a lot of water came out of the cracks on the tunnel roof. During the test on segment 2 also water came out of the cracks but not as much as with segment 0.

The cracking pattern on the top side of the roof was basically identical in the two tests, with large longitudinal cracks above the walls, and the roof covered by a pattern of evenly distributed transverse and longitudinal cracks. The transverse cracks extended well over the roof of the cold tunnel tube in both cases.

## 4.2 Heating rate

To investigate the effect of the heating rate, the three test results of segments 6-1, 8 and 9 can be compared, as well as the two test results of segments 5 and 6-2.

Varied parameter:

| | |
|---|---|
| Heating rate: | segment 6-1: very low (high fire curve but fire protection)<br>segment 9: low fire curve<br>segment 8: high fire curve |

Constant parameters:

| | |
|---|---|
| PP-fibres: | 3 kg/m$^3$, 18 µm diameter, length 6 or 12 mm |
| Loading: | none |
| Concrete age: | 100-107 days |

The deflection of the roof after 60 minutes was roughly 2 mm for segment 8, 0.5 mm for segment 9 and 0 mm for segment 6-1. This clearly shows that faster heating (resulting in a larger thermal gradient over the cross-section of the roof) leads to larger deflection.

The same is true for the width of the crack above the central wall, which after approx. 30 minutes was 0.4 mm for segment 8, 0.2 mm for segment 9 and less than 0.1 mm for segment 6-1.
Also the number of cracks reduces with lower heating rates. For the transverse cracks they were almost eliminated on segment 6-1, and the longitudinal cracks were less in number.

Obviously, the measured temperature on top of the roof (unexposed side) increased when the heating rate was higher.

From this comparison it can be concluded that slower heating, which can be obtained by applying fire protective insulation, can be an effective measure against cracking and roof deflection.

Varied parameter:

| | |
|---|---|
| Heating rate: | segment 6-2: very low (high fire curve but fire protection)<br>segment 5: low fire curve |

Constant parameters:

| | |
|---|---|
| PP-fibres: | 3 kg/m$^3$, 18 µm diameter, length 6 mm |
| Loading: | 7560 kg dead weight |
| Concrete age: | 109-157 days |

The deflection of the roof after 60 minutes was roughly 1.1 mm for segment 5 and 0.6 mm for segment 6-2. Again, this clearly shows that faster heating (resulting in a larger thermal gradient over the cross-section of the roof) leads to larger deflection.

The crack pattern and widths could not be evaluated due to the presence of the loading blocks.

## 4.3 Vertical loading

To investigate the effect of vertical loading, the two test results of segments 5 and 9 can be compared, as well as the two test results of segments 6-1 and 6-2. Moreover, the orientation of the loading blocks on top of segments was varied between segments 5 and 7.

Varied parameter:

| | |
|---|---|
| Loading: | segment 9: none |
| | segment 5: 7560 kg dead weight |

Constant parameters:

| | |
|---|---|
| Heating rate: | low fire curve |
| PP-fibres: | 3 kg/m$^3$, 18 µm diameter, length 6 or 12 mm |
| Concrete age: | 107-157 days |

The deflection of the roof after 60 minutes was roughly 1.1 mm for segment 5 and 0.5 mm for segment 9. This shows that the presence of vertical loading on the tunnel roof leads to larger deflection during fire, which makes sense from a mechanical point of view.

The crack pattern and widths on segment 5 could not be evaluated due to the presence of the loading blocks, so it is not possible to compare them here.

The unexposed surface temperatures on top of the roof were identical, reaching about 60ºC after 60 minutes.

Varied parameter:

| | |
|---|---|
| Loading: | segment 6-1: none |
| | segment 6-2: 7560 kg dead weight |

Constant parameters:

| | |
|---|---|
| Heating rate: | very low (high fire curve but fire protection) |
| PP-fibres: | 3 kg/m$^3$, 18 µm diameter, length 6 mm |
| Concrete age: | 100-109 days |

The deflection of the roof of segment 6-1, without vertical loading, is more or less zero after 60 minutes. This can be explained by the use of fire protection, resulting in a very low heating rate. When the heating rate is very low, the thermal gradient in the cross section is limited. The roof does expand laterally and therefore slightly pushes the walls apart, resulting in a hogging moment at the supports of the roof.
When vertical load is applied, the deflection is downwards again (about 0.5 mm after 60 minutes).

The temperatures at the unexposed top side of the roof were more or less identical. The crack pattern could not be evaluated on segment 6-2 due to the presence of the loading blocks.

Varied parameter:

| | |
|---|---|
| Loading method: | segment 5: 7560 kg dead weight, transverse location on roof |
| | segment 7: 7560 kg dead weight, longitudinal location on roof |

Constant parameters:

| | |
|---|---|
| Heating rate: | low fire curve |
| PP-fibres: | 3 kg/m$^3$, 18 µm diameter, length 6 mm |
| Concrete age: | 153-157 days |

When the roof starts to deflect downwards, due to the transverse placement of the loading blocks (as on segment 5), it can be expected that the distribution of the load shifts towards the points were the roof is supported on the wall. In that case, the effective mechanical loading on the roof will be reduced, unlike a real tunnel were at all times the soil and (ground) water pressures will remain active. To investigate the effect of the placement of the loading blocks another test was carried out with the loading blocks supported longitudinally on top of the roof (on segment 7).

Contrary to the expectation, the measured deflection of the roof decreased slightly when the loading blocks were positioned longitudinally instead of transversely. However, the difference is rather small. It seems reasonable to conclude that the direction of the loading blocks doesn't significantly influence the deflection within the accuracy of the measurement.

The crack pattern again could not be evaluated due to the presence of the loading blocks, but it was observed that in both cases the large crack above the central wall appeared after approx. 15 minutes. The temperatures inside the tunnel and at the unexposed top side of the roof were more or less identical although they are slightly higher on segment 7 than on segment 5, which might contribute to the slightly increased vertical deflection.

## 4.4 PP-fibres

To investigate the effect of adding PP-fibres, the two test results of segments 2 and 8 can be compared. Moreover, two different fibre types can be compared by looking at segments 6-1 and 10.

Varied parameter:

| | |
|---|---|
| PP-fibres: | segment 2: none |
| | segment 8: 3 kg/m$^3$, 18 µm diameter, length 12 mm |

Constant parameters:

| | |
|---|---|
| Heating rate: | high fire curve |
| Loading: | none |
| Concrete age: | 101-133 days |

Visual observation of the development of a longitudinal crack above the central wall was at the same time, approx. 5 minutes, for both segments. However after this point the crack development seems to be somewhat different.
For the segment with PP-fibres, the crack width after some 30 minutes is limited to approx. 0.4 mm, whereas the crack width in the segment without fibres reaches 0.9 mm. However, the segment with PP-fibres shows a larger number of cracks.

The deflection of the roof is more or less the same for both segments, with and without PP-fibres.

On the segment without PP-fibres more water is coming through the cracks towards the unexposed top surface of the roof than on the segment with PP-fibres. The temperatures on top of the roof are similar, about 50-60ºC after 30 minutes.

Varied parameter:

| | |
|---|---|
| PP-fibres: | segment 6-1: 3 kg/$m^3$, 18 µm diameter, length 6 mm |
| | segment 10: 3 kg/$m^3$, 32 µm diameter, length 12 mm |

Constant parameters:

| | |
|---|---|
| Heating rate: | very low (high fire curve but fire protection) |
| Loading: | none |
| Concrete age: | 83-100 days |

It is known from fire tests that the diameter of the fibres can influence their effectiveness against spalling. Therefore, two segments with PP-fibres of different diameters were tested in order to compare the possible influence of the type of PP-fibre on cold-side cracking.

In both cases, the deflection of the roof was almost zero, probably because of the very low heating rate and consequent small thermal gradient.

The crack pattern was quite similar, and the width of the crack above the central wall after 30 minutes was limited to less than 0.1 mm on segment 6-1 and less than 0.2 mm on segment 10. Both crack widths are quite small, which can be explained from the fact that the thermal gradient is small and therefore there will be little rotation of the roof in the top corners.

On both segments there is only very little water coming out of the cracks in the roof, and the temperatures on top of the roof are similar.

# 5 Conclusions and evaluation of the fire tests

In all tests, more or less the same crack pattern developed. Possible variations include the number of cracks and their intermediate distances, and the widths of the cracks. The global pattern was always the same, see figure 5.1.

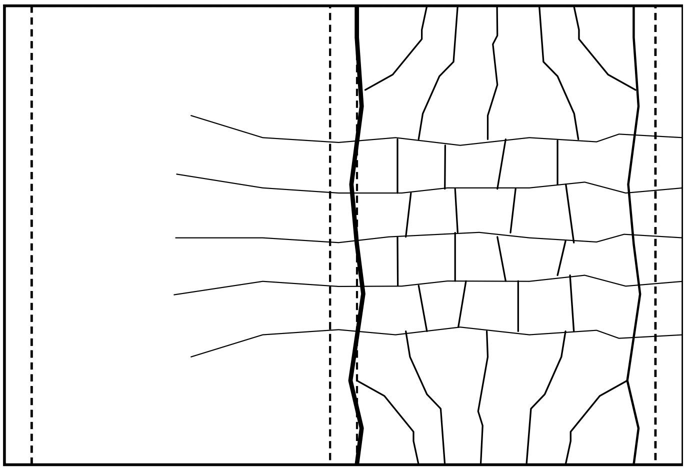

*Figure 5.1: top view of the global crack pattern on top of the roof*

Looking at the cross section of the tunnel, the largest cracks occurred near the walls, as shown in figure 5.2

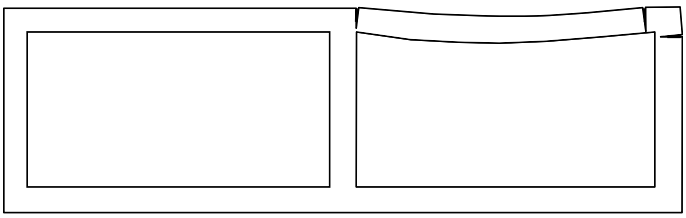

*Figure 5.2: schematic view of the main cracks near the walls and the deformed shape of the cross section*

When looking at each individual studied parameter, the consequences in terms of fire behaviour are presented in table 5.1.

Table 5.1: Parameters and their influence on fire behaviour of the concrete tunnel segments

| | Vertical deflection | Crack width | Crack pattern | Moisture on cold side | Temperature on cold side |
|---|---|---|---|---|---|
| Fire protection / Lower heating rate → | Less deflection | Smaller crack width | Smaller number of cracks, esp. in transverse direction. | No influence | Lower temperature |
| Adding PP-fibres → | No influence | Smaller crack width. No influence of the type of fibre | Larger number of cracks. No influence of the type of fibre | Less moisture escaping on cold side | No influence |
| Higher vertical load → | More deflection | * Indirect information: as the deflection increases, the cracks above the walls must have been wider. | * | No influence | No influence |

***** *due to the loading blocks, crack widths could not be measured and the crack pattern could not be observed during tests with vertical loading.*

Due to the thermal gradient in the roof, the roof bows towards the fire. When applying a vertical load this curvature is increased. The bowing towards the fire causes hogging moments at the rigid connection of the roof to the walls.

The thermal gradient causes vertical cracks in the centre of the cross section in many locations in the roof; near the rigid connections to the wall and the consequent hogging moment, these cracks extend to the outer surface of the tunnel. On the inside of the tunnel the cracks are not visible.

Given the mechanism it is obvious, as shown in figure 5.3, that there is a geometrical relation between the rotation of the roof plane near the rigid connections in the corners and the crack width, and also there is a geometrical relation between the rotation of the roof plane near the supports and the deflection of the roof at mid span.
Therefore the crack width near the corners (or the sum of the crack widths of multiple cracks occur close to each other) must be more or less proportional to the vertical deflection of the roof at mid span. An estimation of this proportional relation can be obtained by assuming that

- the crack opening is a V-shape, with the maximum opening on the top and zero opening on the bottom of the roof, which means that the corner rotation $\phi = w_{cr}/d_{roof}$
- the deformed shape of the roof is roughly parabolical, which means that the mid span deflection $\varepsilon = \phi * L_{roof}/4$

With these assumptions, $\varepsilon/w_{cr} = L_{roof} / (4*d_{roof})$

When appling this formula on segment 2, with $L_{roof}$ = 1000 mm and $d_{roof}$ = 125 mm, the ratio $\varepsilon/w_{cr}$ = 2. After 26 minutes a crack width of 0.9 mm was measured and the vertical deflection of the roof at mid span relative to the connections to the walls was approx. 1.85 mm, a ratio of 2.06. This shows that the proportionality of the vertical deflection and the crack width can be well approximated using these formulae.

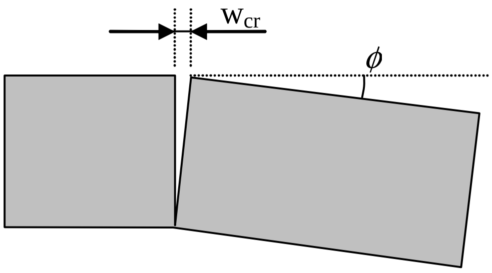


*Figure 5.3: illustration of the relation between rotation and crack width*

At the connection with the outer wall, during the fire tests typically two cracks occurred: a horizontal crack in the wall and a vertical crack in the roof. The total rotation in this corner is therefore spread over two cracks. In a full scale tunnel situation, the geometry is somewhat different because the corners are normally designed as a short diagonal part (chamfer). Therefore the number of cracks near the corner in a real tunnel might not be exactly two, it could also be only one crack or maybe slightly more than two cracks, depending on the exact geometry. If only one crack occurs, it is to be expected that the crack width is larger because all rotation is concentrated in that single crack.
At the connection to the central wall there is only one crack, so all rotation is concentrated in this crack. Therefore the crack above the central wall is typically twice the width of the cracks on the corner of the roof and the outer wall.

A lower heating rate reduces the thermal bowing. As a result, the deflection at mid span is strongly reduced and so are the crack widths at the connections of the roof to the walls. A lower heating rate can be obtained by applying a thermal barrier (passive fire protection) on the exposed side of the concrete.

Adding PP-fibres somehow seems to slightly change the crack behaviour of the roof above the central wall. The deflection at mid span is completely unaffected and one would therefore expect the same crack width above the central wall. Nevertheless, the crack width is reduced, but judging from the crack pattern the number of cracks is more than one. Even though there is still one relatively large crack, some other parallel cracks are also present close to the connection. Therefore the rotation is spread over these cracks and the width of each individual crack is reduced, see figure 5.4.

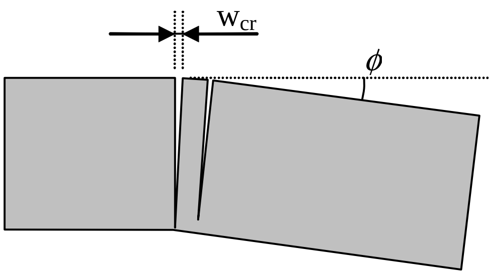


*Figure 5.4: smaller individual crack width in case of multiple cracks, compared to the situation in figure 5.3*

An explanation for the changing cracking behaviour could be that the concrete is less brittle when PP-fibres are included. On the cold side, the PP-fibres do not melt, so they can bridge a crack just when it occurs, causing some redistribution of forces. When the concrete is less brittle, cracks will be less localised. In that case, instead of one large crack two or more smaller cracks may develop.
However when the crack width becomes larger the PP-fibres can no longer bridge the gap and the effect will disappear. Therefore it has to be seen how the scaled test results relate to a full scale situation; when the crack width would be significantly larger in a full scale situation, the PP-fibres probably would no longer have any influence on the crack pattern.

# 6 Further analysis of mechanisms causing cold-side cracking

The tests have provided a good overview of the approximate relations between heating, fire protection, adding PP-fibres, vertical loading on one hand and the cold side cracking on the other hand.

Parallel to the fire test series, numerical analyses where carried out by Ben Nieman as part of his Master’s Thesis study at Delft University of Technology, see “Cracking on the unheated side during a fire in an immersed tunnel” , Ben Nieman, Master’s Thesis TU Delft, August 2008.

The numerical analyses where based on a Diana 9.3 model of the cross section of an immersed tunnel, using a random triangular mesh of continuum elements, see figures 6.1 and 6.2. The model was validated by calculating the tested situations of the scaled tunnel segments. After the validation study showed good correlation, the model was also applied to a typical geometry of an immersed tunnel.

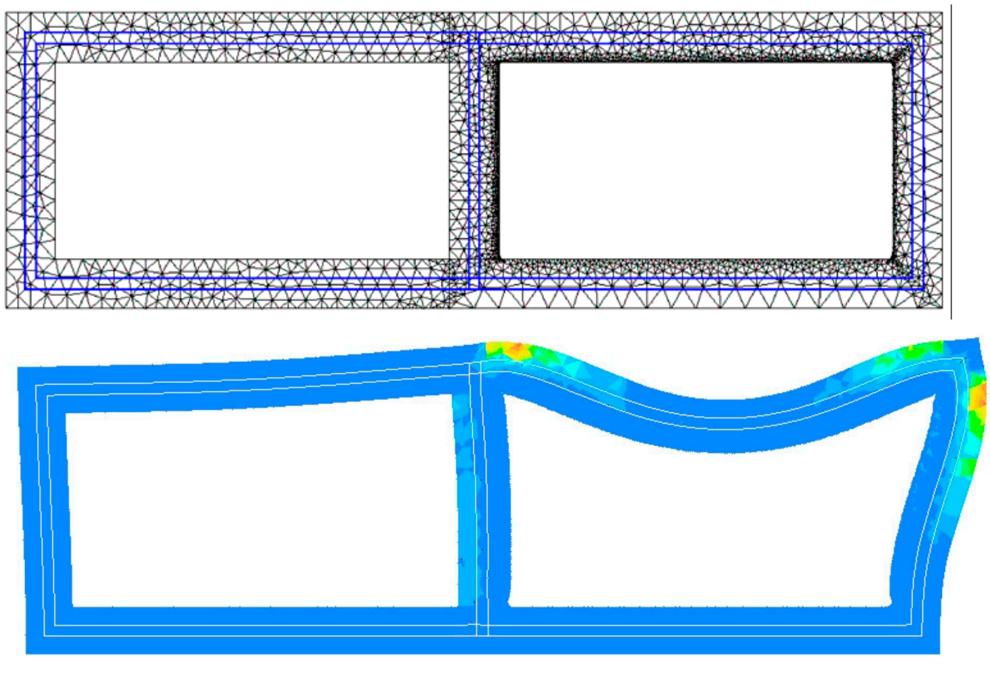

*Figure 6.1: finite element mesh and calculation result for the scaled tunnel segment*

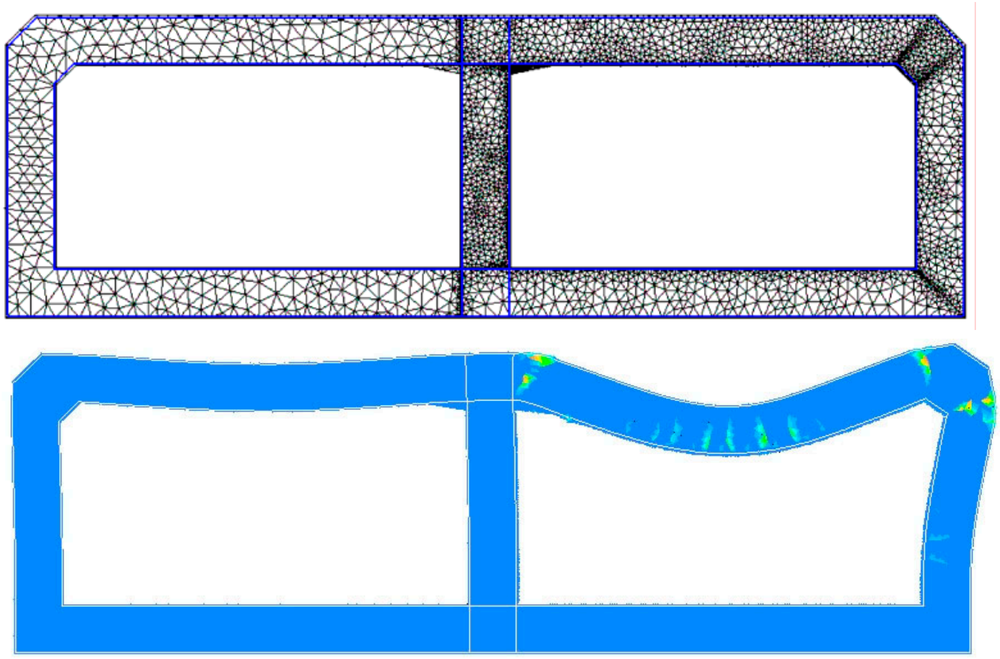

*Figure 6.2: finite element mesh and calculation result for the full scale tunnel*

The conclusions from the simulations as reported by Ben Nieman are as follows.

*Calculations using this finite element simulation show that cracking on the unheated side is a realistic phenomenon that may manifest itself during a fire in an immersed tunnel. Although certain simplifications have been made while modelling, a comparison with fire tests carried out at Efectis shows that the model has a good predictive value for the behaviour of these specimens. Although only a limited number of tests were performed, it seems reasonable to assume that the model also has a good predictive value for an actual size tunnel.*

*This means that serious crack widths on the unheated side will be reached during a fire, up to 1.8 mm after 40 minutes for an unprotected tunnel. As natural fires will typically have a longer duration, the resulting crack widths will evidently be even larger. In addition to a large crack in the roof adjacent to the middle wall, there will also be smaller cracks in the roof adjacent to the side wall (of the heated tube), and in the side wall just below the roof.*

*There are, in general, two ways of avoiding these cracks. Firstly, the crack initiation can be delayed by applying fire-resistant plates (which reduces the temperatures to which the concrete is exposed), or by increasing the tensile strength (although that is a less feasible option for actual tunnels). Secondly, one can accept that cracks will develop, but keep their size within acceptable limits, which can be achieved using reinforcement: similar to any other concrete construction, the size of cracks can be limited by increasing the reinforcement area on the side of the cracks. As this pertains to the unheated side, there are no temperature effects here.*

*However, the realisation that cracking may occur on the unheated side if this is not actively prevented is only a partial conclusion, as long as the question how damaging such a crack might be is not yet addressed. The structural integrity of the tunnel is not directly threatened by the occurrence of the cracks; even with crack widths as large as 1.8 mm the sag will be limited to 10 mm in the middle of the span, which is well within acceptable limits.*

*The durability of the heated region of the tunnel will be reduced during the crack, as the strength and stiffness of both the concrete and the reinforcing steel will decrease due to the high temperatures, and will not return upon cooling. However, this is a general risk associated with fire that has already been extensively described in other studies, and not a consequence of the cracks on the unheated side that are the focus of this master's thesis.*

*However, one particular structural aspect that is noticeably influenced by these cracks is the shear capacity at the location of the cracks. As cracks cannot transmit shear stresses, the shear capacity is reduced at the locations of the cracks, which coincide with the locations of the largest shear stresses. If shear reinforcement is applied this is not likely to become a problem, but it is important to keep the possibility of cracking on the unheated side in mind when designing shear reinforcement and chamfers in an immersed tunnel.*

*Beside this, it would seem that the main danger stemming from these cracks is a long-term one: the risk of corrosion of the reinforcement. How imminent this risk is, however, would be difficult to predict, as long as the cooling down phase is not analysed. The standard material models in DIANA do not facilitate the modelling of the cooling down phase, but with a user-supplied material model, this problem has been circumvented. The material model described in this master's thesis has therefore cleared the way to a proper modelling of the cooling down phase.*

Furthermore, the report gives recommendations regarding the numerical modelling, such as the incorporation of the behaviour during cooling down after a fire, further modelling of the cracked zone and further investigation of the shear capacity of the cracked cross section.

It is unknown if the crack width will increase or decrease during cooling down. The available material models are unsuitable to simulate this. Nevertheless, a justified question is whether a crack of 1.8 mm width might fill up with sand grains and therefore will be unable to slightly close during cooling down. Moreover, the calculated crack width of 1.8 mm occurs after 40 minutes. For numerical reasons it was not possible to continue the calculation for longer fire durations, but it is expected that the crack width will increase.

Even with fires of less extreme temperature or durations of less than 40 minutes, it is still possible that the cracks are there; the simulations show that the crack emerges at 17 minutes and starts to open up after 20-30 minutes of exposure to the RWS fire curve.

When applying fire protection, the time of the onset of the crack can be delayed significantly. The simulations by Nieman show that when a fire protection of 27.5 mm thick boards is used, the crack will occur after approximately 75 minutes.

The rate at which the crack widens increases when applying fire protection to the concrete. Assuming a fire duration of 120 minutes, this means that the unprotected crack will open up during about 100 minutes, whereas using 27.5 mm board protection, the crack will open up during only 45 minutes but at a slightly higher rate.

The model did not include the effect of PP-fibres, because this would require material models that would explicitly take into account moisture content and flow. However, the reduction of the crack width which was observed on the small scale tunnel segments may also have to do with the increase of ductility of the concrete. Such a ductile behaviour could be caused by fibres bridging the developing crack. However, with a crack width as large as 1.8 mm it is unsure if the “anti-spalling” type of PP-fibres would be able to bridge the crack and significantly contribute to the ductility of the concrete.

# 7 Repairability

Cold side cracks are likely to occur during a fire. These cracks cannot be seen from the inside of the tunnel, but they will extend all the way to the outer surface of the tunnel roof (or wall). Even though the deflection of the tunnel roof remains invisibly small, the width of the cracks will be significant. Given this information, the following questions must be answered:

Is it a problem if cold side cracking occurs in a tunnel roof during a fire?
- Is the shear capacity of a tunnel roof after a fire still sufficient?
- Is the durability of the tunnel roof after a fire still sufficiently guaranteed?

And if it is concluded that there is a problem,
- Can the cracks be detected and repaired with sufficient reliability?
- Or is it worthwhile to take measures to prevent the cold side cracking?

## 7.1 Shear capacity

The location of the cracks is exactly there where the shear forces are at their highest. Nevertheless, even a cracked cross section can still accommodate some shear force provided that there is sufficient reinforcement. It is recommended for tunnel owners to assess their tunnels with regard to shear resistance assuming a cracked cross section.

## 7.2 Durability

The crack width of 1.8 mm will leave the reinforcement directly exposed to the external environment. This means that ground water, or possibly salt sea water, will affect the reinforcement. As the shear capacity is strongly reduced, the reinforcement becomes even more important, so it is thinkable that after time the tunnel roof collapses.

## 7.3 Detection and repair of cracks

Deflection of the roof will be limited to roughly 10 mm. As probably no detailed measurements of the original shape of the tunnel exist, it will probably be impossible to observe or measure this deflection.

The cold side cracks cannot be seen from the inside of the tunnel. This means that they may be overlooked when assessing the damage. Even if the deflection of the roof is unnoticeably small, the crack width can be in the order of magnitude of 1.8 mm.

Detecting these cracks will not be easy. Dependent on the ductility of the concrete there may be one large crack or multiple smaller cracks. In any case, the shape of the cracks will be quite irregular. Moreover, the length of the cracks in longitudinal direction depends on the heated length of the tunnel. Even at some distance from the fire, the concrete structure will still be heated, even if the heating is less severe than directly near the fire. It is hard to predict over what length of the tunnel the cracks will be present. Therefore the number of cracks, the length of the cracks and their precise locations in the cross section are very hard to predict.

Injecting the cracks with a resin may be an option, but when the exact locations of the cracks are unknown this is probably not an easy task. Moreover, quality control of the injection is virtually impossible because the proper filling of the crack can only be observed from the outside of the tunnel.

## 7.4 Measures to prevent cracking

Basically there are two options to limit cold side cracking of the concrete:
- adding reinforcement, or
- (additional) thermal insulation.

Adding reinforcement near the outer surface of the tunnel roof may help to redistribute the forces, causing a fine crack pattern of multiple narrow cracks rather than one large crack. However, immersed tunnel roofs are normally quite heavily reinforced so adding extra reinforcement will not be an easy task. Moreover this is clearly not an option for existing tunnels.

Thermal insulation slows down the heating of the concrete, and therefore reduces the thermal gradient inside the roof. This means that the onset of the cold side crack is delayed. It is a matter of applying sufficient insulation layer thickness to make sure that the cold side cracks will be delayed beyond any reasonable fire duration. The proper amount of fire protection can be estimated using the model, but it is recommended to verify the effectiveness in a large scale fire test.

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