IMPROVING SAFETY AND DURABILITY OF CIVIL STRUCTURES

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Abstract. Society expects that failure of civil structures is extremely rare and relies on the care and expertise of the professionals involved in the design, construction and maintenance of structures. Analysis shows that human errors are a major source of structural failures. Relevant measures to improve the safety of civil structures include combating human error, applying hazard recognition methods, establishing organisational documents and performing adequate monitoring and maintenance of structures. Nowadays dealing with existing structures is a major engineering task. Safety evaluation of existing structures follows a stepwise procedure with an increasing degree of refinement of investigations. Target safety levels may be defined as a function of the hazard scenario and the characteristics of the structure under consideration. To improve durability of structures an original concept is to use Ultra-High Performance Fibre Reinforced Concrete (UHPFRC) to "harden" those zones of the structure that are exposed to severe environment and high mechanical loading. This conceptual idea combines efficiently protection and resistance properties of UHPFRC and significantly improves the structural performance. The concept is validated by means of a numerical simulation and applications.

Keywords: Structural safety, hazard scenarios, lessons from structural failures, human factors, safety evaluation of existing structures, target safety levels, ultra-high performance fibre reinforced concrete, durability, composite concrete construction, rehabilitation of concrete structures, bridge design, FE-analysis

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A. Ibrahimbegovic and M. Zlatar (eds.), *Damage Assessment and Reconstruction After War or Natural Disaster*,

1. Introduction

1.1. SAFETY OF CIVIL STRUCTURES

Society expects that the failure of civil structures is extremely rare and relies on the care and expertise of the professionals involved in the design, construction and maintenance of structures. This is in particular true for public technical systems such as transportation or energy supply systems and structures such as bridges.

Structural safety may be defined as follows: "*Adequate safety with respect to a hazard is ensured provided that the hazard is kept under control by appropriate measures or the risk is limited to an acceptable value. Absolute safety is not achievable.*" It is thus not the structure as such that is designated safe but rather the people, goods and the environment in its surroundings.

The continued use of existing structures is of great importance because the built environment is a huge economic and political asset, growing larger every year. Nowadays evaluation of the safety of existing structures is a major engineering task, and structural engineers are increasingly called upon to devise ways for extending the life of structures whilst observing tight cost constraints. Also, existing structures are expected to resist against accidental actions although they were not designed for.

Engineers may apply specific methods for evaluation in order to preserve structures and to reduce a client's expenditure. The ultimate goal is to limit construction intervention to a minimum, a goal that is clearly in agreement with the principles of sustainable development.

In Chapter 2, lessons from structural failures are presented first and measures to improve safety in the design, construction and maintenance of structures are deduced. Principles for the safety evaluation of existing structures and a procedure to determine target safety levels are outlined.

1.2. DURABIITY OF CIVIL STRUCTURES

Concrete structures show excellent performance in terms of structural behaviour and durability except for those zones that are exposed to severe environmental and mechanical loading. Rehabilitation of deteriorated concrete structures is a heavy burden also from the socio-economic viewpoint since it also leads to significant user costs. As a consequence, novel concepts for the rehabilitation of concrete structures must be developed. Sustainable concrete structures of the future will be those requiring just minimum interventions of only preventative maintenance with no or only little service disruptions.

Over the last 10 years, considerable efforts to improve the behaviour of cementitious materials by incorporating fibres have led to the emergence of Ultra-High Performance Fibre Reinforced Concretes (UHPFRC). These novel building materials provide the structural engineer with an unique combination of (1) extremely low permeability which largely prevents the ingress of detrimental substances such as water and chlorides and (2) very high strength, i.e., compressive strength higher than 150 MPa, tensile strength higher than 10 MPa and with considerable tensile strain hardening (up to more than 1‰ of strain) and softening behaviour (with fracture energy of more than 15,000 J/m^2). In addition, UHPFRC have excellent rheological properties in the fresh state allowing for easy casting of the self-compacting fresh material with conventional concreting equipment. Consequently, UHPFRC clearly have an improved resistance against severe environmental and mechanical loading thus providing significantly improved structural resistance and durability to concrete structures.

Chapter 3 presents an original concept of using UHPFRC for the improvement of concrete structures.

2. Lessons from Structural Failures

2.1. BRIDGE FAILURES IN SWITZERLAND

In Switzerland with a stock of about 25,000 bridges, no bridge collapse occurred during service since the failure of the steel railway bridge in Mönchenstein in 1891 that caused 73 deaths (Fig. 1a). As a consequence of this tragic accident, codes were introduced and systematic monitoring and maintenance of bridges has been performed since then in Switzerland. In several cases, partial structural failure could be prevented due to monitoring and subsequent immediate intervention.

In 1987, scour of a bridge pier on the Gotthard highway at Wassen led to significant economic loss for Switzerland and Europe because of the temporary closure of this important highway crossing the Alps (Fig. 1b). Fortunately there were no human casualties.

Two bridge failures occurred during construction in 1973 when the concrete deck slab of the composite bridge at Valangin (Fig. 1c) and the steel work of the composite bridge at Illarsaz were launched.

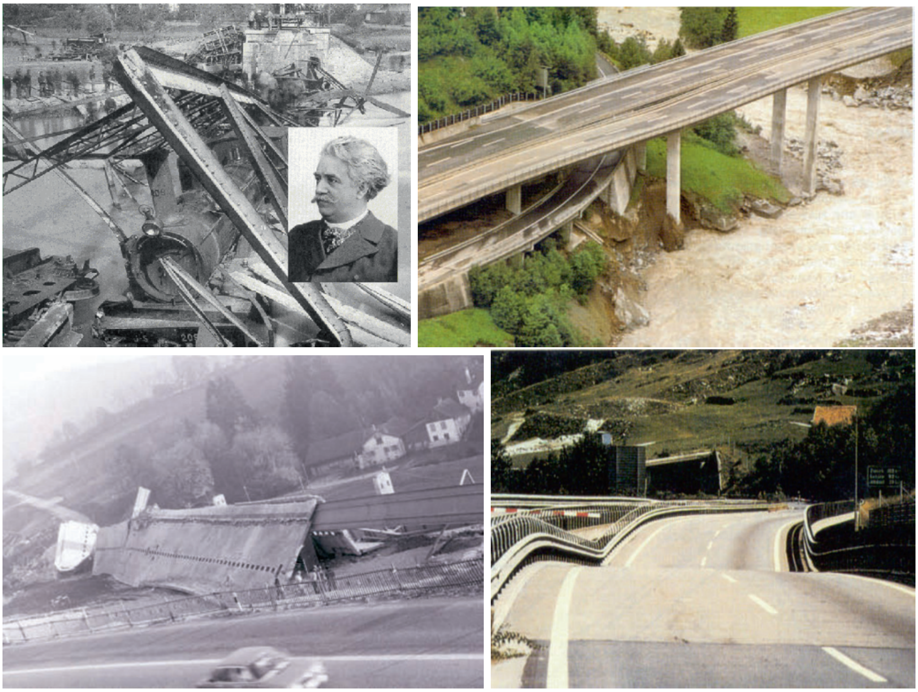

Figure 1. (**a**) Bridge collapse at Mönchenstein in 1891, (**b**) scour at the highway bridge at Wassen in 1987, and (**c**) failure during launching of the deck slab of the composite bridge at Valangin in 1973

2.2. BRIDGE FAILURES WORLDWIDE

A total of 138 cases of bridge failure worldwide have been analysed in Bailey et al. (2001). The following lessons can be learned from this study:

- Most of the bridge failures are due to human error on the part of the structural engineer or an inadequate or unplanned use of the bridge. As a consequence, human behaviour has to be "challenged" in order to reduce structural failures.
- Around 40% of all bridge accidents occurred during construction or during the first two years in service. This shows that more attention should be given to safety aspects relating to the construction phase. Efficient controls (quality assurance) during construction, a conscientious inspection of the structure before opening to service and an intensified monitoring during initial service life may help to prevent and detect defects leading to structural failure.
- Failures due to exceptional natural causes such as earthquake, wind or scour generally occur on bridges for which these hazards have been accepted, not recognised or not correctly considered due to lack of knowledge. Consequently, bridges with conceptual defects should be systematically identified and evaluated in order to improve the safety of a bridge stock.

- Failures due to corrosion or fatigue normally occur at an advanced age of the structure. This points to the significance of adequate monitoring and maintenance of the structure.
- Accepted risks and objectively unknown hazards represent only 8% of all bridge failures.
- No bridge failure could be traced back to items not or insufficiently covered by the codes of practices. This means that the safety level as implicitly applied is – as a rule – sufficiently high.

From these characteristics of bridge failures (not considering gross human errors), it can be deduced that inherently higher structural safety exists for bridges satisfying the following conditions:

- Conceptual design, dimensioning, detailing and construction of the bridge is in accordance with state-of-the-art knowledge and human factors have been considered.
- The bridge has shown a normal behaviour during the first years of service.
- The bridge is subjected to systematic and adequate monitoring and maintenance during its service life.

2.3. MEASURES DEDUCED FROM STRUCTURAL FAILURES

The results of the analysis of structural failures (as shown in Section 2.2 and other similar studies) challenge engineers to learn the lessons and to take the necessary measures. There are four domains where structural engineering has to improve:

2.3.1. *Hazard Recognition Methods*

Safety has a lot to do with the recognition of possible hazards. Once the potential hazards have been recognised, reducing their harmful effects is usually relatively easy. Not having recognised a hazard may lead to one of the worst experiences of an engineer. The objective is thus to recognise all possible hazards because only then a safe solution can be found. This is difficult to achieve.

Hazard recognition requires imagination and creativity from the engineer. There are techniques and methods that are helpful in trying to recognise possible hazards (Schneider, 1997):

- Chronological analysis: Step-by-step the process is considered beforehand (what, where, when will occur).
- Utilisation analysis: It is essential to analyse in advance the way the structure will be used. (what could go wrong? what could break down and become hazardous?)

- Influence analysis: which quantities influence the problem at hand (damaging influences in human activities and shortcomings)? which individually safe components of a situation can in combination become hazardous?
- Energy analysis: what are the energy potentials in the system?
- Material analysis: what properties of building materials can become hazardous (combustibility, explosiveness, toxicity, corrosion)?

Such strategies are used under various names such as Hazard and Operability Study (HAZOP), What-if Analyses or Failure Mode and Effect Analysis (FMEA).

2.3.2. *Organizational Documents*

A principal conclusion from the analysis of accidents is the necessity to establish organisational documents for the design, construction and maintenance of the structure. Preparation and updating of these documents requires a dialogue between all professionals involved in the design, construction and maintenance and in particular with the owner of the structure.

These organizational documents include a "utilization plan" and a "safety plan" ideally to be prepared for all structures in the initial planning phase. From these two basic documents a series of documents are derived which guide the design and construction phase as well as the service phase (utilisation and maintenance), i.e. during the complete life of the structure:

- The *utilization plan* specifies what is wanted. It lists the service states to be considered for the structure and defines the measures to ensure serviceability.
- The *safety plan* is developed on the basis of the utilisation plan and lists the *hazard scenarios* to be considered for the structure and defines the measures to guarantee adequate safety.

Hazard scenarios are critical situations and the conditions which might represent a hazard for a structure. Each hazard scenario is characterised by a predominant action and by one or more accompanying actions. The identification and evaluation of hazard scenarios represent the basis for the planning of measures to be taken to ensure adequate safety.

During the construction phase, co-operation between all those involved in the construction process should be improved and well-planned quality assurance procedures should be adopted.

Monitoring during the service phase of the structure provides the relevant information characterising structural behaviour in order to evaluate structural safety and to enable appropriate maintenance interventions.

Monitoring includes frequent or continuous, normally long-term, observation of structural conditions or actions by means of inspections and measurements. The objective is to detect abnormal structural behaviour or actions as early as possible; ideally before any damage process is initiated. Measured values may serve as "indicators", "warning signals" or "alerts". Maintenance is routine intervention to preserve adequate structural performance.

A monitoring and maintenance plan should be specified after the construction of a new structure and constantly updated during its service life depending on the utilisation plan and the results of monitoring or evaluation.

2.3.3. *Combating Human Error*

Human errors are clearly the main source of damage and structural failures. They can be combated at several levels (Schneider, 1997):

- *Subjectively unknown hazards* – by improving basic and continuing education and training and by publishing examples of bad experiences
- *Ignored hazards* – by clear allocation of responsibility and competence as well as by rigorously combating all forms of carelessness, negligence and ignorance
- *Unsuitable measures* – by improving expert knowledge, carefulness and overview with all those who plan the measures
- *Improper use of measures* – by requiring clear and unambiguous plans, basic documents and instruction, as well as by creating and maintaining effective control mechanisms
- *Objectively unknown hazards* – by fundamental research and systematic dissemination of experience

2.4. SAFETY EVALUATION OF EXISTING STRUCTURES

2.4.1. *Methodology*

The need to evaluate the safety of an existing structure can usually be related to doubt about the safety. The fundamental question is whether the structure is safe enough. If the answer is no, one of the following interventions has to be performed: demolish the structure and replace it by a safer one, strengthen the structure, ask for reduction of loads or reduce uncertainty by intensifying monitoring. If the answer is yes, then to do nothing and allow the continued operation of the structure may be the most appropriate action. However, intensifying the monitoring of the structure is sometimes beneficial.

Experience shows that a stepwise procedure in the evaluation of a structure is appropriate (ISO International Standard, 2001):

Step 1: *Objectives and hazard scenarios*

First, the objective of the evaluation must be clearly specified in terms of the future use of the structure (remaining service life). Hazard scenarios related to a change in structural conditions or actions should be specified in the safety plan in order to identify possible critical situations. Furthermore, accepted risks must be identified. As soon as there is some evidence of danger to humans or the environment, protective measures must be taken immediately.

Step 2: *Preliminary evaluation*

The preliminary evaluation consists of a study of documents, an inspection and preliminary structural safety checks to identify the critical deficiencies. The objective of Step 2 is to remove existing doubts using fairly simple methods which must allow proposals to be made for subsequent measures.

Current codes which have proven to provide adequate reliability over a long period of application may be used as a reference but not necessarily as the relevant criterion, and former codes that were valid at the time of construction of an existing structure should be used as informative documents only.

In studying the available documents, a deep insight should be gained into the situation when the structure was designed and built. These are indicators of quality referring to aspects such as design principles and methods, codes, construction methods and materials or working environment. Where there is uncertainty in the actions, action effects or properties of the structure, a detailed evaluation should be performed.

If the doubts that led to the evaluation cannot be overcome in the course of Step 2, further investigational steps must be undertaken in Step 3.

Step 3: *Detailed evaluation*

Detailed structural investigations and updating of information are typical of Step 3. Updating is based on prior information about the structure and on specific additional observations and measurements. In any case, the objective is to verify structural safety.

A detailed inspection of the structure or the structural element in question is extremely important to recognise typical hazard scenarios that could endanger the structure's remaining service life. Furthermore, any defects and damage due to excessive or unplanned loading must be detected.

By means of detailed structural analysis, reserves of strength in structural components may be identified and exploited using ultimate limit state concepts. Deterioration must be analysed as a time-dependent structural reliability problem.

The safety check of an existing structure should be carried out to ensure a target reliability level that reflects the required level of structural performance. The target reliability level may be determined taking into account the required performance level for the structure, the reference period and possible failure consequences.

The conclusion from the assessment shall withstand a plausibility check. In particular, discrepancies between the results of structural analysis (e.g. insufficient safety) and the real structural condition (e.g. no sign of distress or failure, satisfactory structural performance) must be explained.

2.5. TARGET SAFETY LEVELS FOR EXISTING STRUCTURES

2.5.1. *General Remarks and Concept*

Compared to the design of new bridges, there are two main reasons for treating safety of existing structures differently. Firstly, there are fewer hazards and less uncertainty once a structure has successfully entered service and performed satisfactorily. For example, 40% of bridge accidents occur during construction (see Section 2.2), and there is no need to cover this hazard when evaluating an existing bridge. Secondly, it costs more to increase the safety of an existing structure. As a consequence, target reliability levels for existing structures may be used and justified on the basis of socio-economic criteria, thus following a risk-based safety approach.

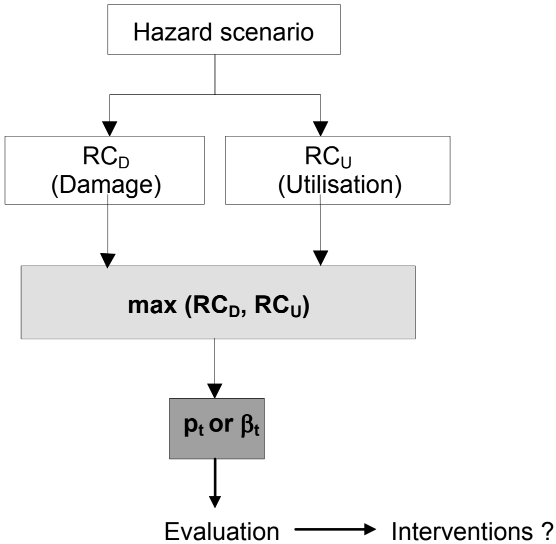

Figure 2. Approach to define the target safety level for a given hazard scenario (RC = Risk Category)

Taking account of the fact that structures are unique, the approach is to define target safety levels as a function of the hazard scenario rather than applying an uniform target safety level to all scenarios and structures. The motivation for this is to rationalize evaluation of existing structures with a view to avoiding interventions on structures that are already adequately safe. (This is often the case when design codes (for new structures) are simply applied to the evaluation of existing bridges.)

Such an approach to defining target safety levels is suggested in Bailey et al. (2001) involving the following steps (Fig. 2):

- Identification of hazard scenarios
- Definition of the consequences of a given hazard scenario with respect to damage and the economic importance of the structure as characterized by a risk category RC
- Selection of the target safety level p_t as a function of the magnitude of these consequences

It is important to note that the aim of this approach is not to reduce safety levels globally throughout the bridge network, but rather to target a uniform level of acceptable risk.

2.5.2. *Risk Categories*

Risk category for damage: The magnitude of damage can be described in terms of the number of casualties due to structure failure (Table 1). As a function of the hazard scenario and the failure mode, the number of casualties depends on the utilization, geometry and situation of the structure. There is a direct relation between the number of casualties and the mean daily use of the structure by persons.

TABLE 1. Risk categories for damage RC_D (As suggested in Bailey et al. (2001))

Probable number of casualties	Risk category for damage RC_D
<1	I
1	II
5	III
10	IV
50	V
100	VI
500	VII

Risk category for utilisation: A structure is an element of a given system (f.ex. transportation or supply system). The value of utilisation of a structure can be estimated by means of the costs incurred by its failure. In Table 2 risk categories are given as a function of the consequences of structure failure and the relative costs of safety measures. Consequences are defined by a ratio between failure costs C_{fail} (cost of accident, cost of re-construction of the bridge including user costs) and intervention cost C_{int} needed to prevent structure failure: $\rho = C_{fail}/C_{int}$. User costs depend on the importance of the system in terms of utilization intensity.

TABLE 2. Risk categories for utilisation RC_U (As suggested in Bailey et al. (2001) and adapted from JCSS Probabilistic Model Code)

Relative costs of safety measures	Consequences		
	Minor $\rho < 2$	Moderate $2 < \rho < 5$	Major $5 < \rho < 10$
Large	I	II	III
Normal	III	V	VI
Low	V	VI	VII

2.5.3. *Acceptable Risk Levels*

Acceptable risk levels may be assessed based on historical surveys of the risk associated with structure failures and the risk accepted by the public in daily activities, such as mountaineering and rail travel. These risks are then used to define an acceptable level of risk to be used for evaluating existing structures.

In terms of individual risk of death due to structure failure the study (Bailey et al., 2001) suggests:

- Lower limit: probability of 10^{-6} (deaths/habitant and year)
- Upper limit: probability of $3\ 10^{-4}$ (deaths/habitant and year) (corresponding to the risk of loss of life in a car accident)

2.5.4. *Target Safety Values*

Table 3 presents the target probability of failure p_t and the target reliability index β_t for each risk category. The target risk category is taken as maximum of the risk categories for damage RC_D and for utilisation RC_U (Fig. 2).

TABLE 3. Target probability and reliability indices as a function of the risk category (Bailey et al., 2001)

Risk category RC	Target probability of failure p_t	Target reliability β_t
I	10^{-3}	3.1
II	$5{\cdot}10^{-4}$	3.4
III	10^{-4}	3.7
IV	$5{\cdot}10^{-5}$	4.0
V	10^{-5}	4.2
VI	$5{\cdot}10^{-6}$	4.4
VII	10^{-6}	4.7

2.5.5. *Safety Check*

The target safety level is thus derived as a function of “external” (non-technical or intangible) parameters representing the value and importance of a structure.

This target safety level is then compared to the estimated safety (using engineering methods of calculation of failure probability) which is determined using “internal” (tangible) parameters describing the state of the structure.

In terms of probabilities the safety check is expressed as follows:

$$p_f = p_u \cdot \left(1 - p_{\det}\right) \leq p_t$$

with:

p_f : probability of failure of a structure or a structural element

p_u : probability of failure as calculated by structural analysis

$p_{\det}$: probability of detection of an extreme or unplanned load or a damage process decreasing the strength of a structural element

p_t : target probability of failure

This equation also shows that the probability of failure can be reduced by intensifying monitoring of a structural element and thus increasing the probability of the detection.

2.6. CONCLUSIONS

The analysis of structural failures provides a rich and valuable source from which relevant knowledge and measures to improve safety can be deduced. Human errors are clearly the main source of structural failures.

Relevant measures to improve the safety of civil structures include combating human error, applying hazard recognition methods, establishing organisational documents as well as systematic and adequate monitoring and maintenance of structures.

The safety evaluation of an existing structure should follow a stepwise procedure with an increasing degree of refinement of investigations.

For the evaluation of the structural safety of existing structures, target safety levels may be defined as a function of the hazard scenario and the characteristics of the structure under consideration. A methodology to define target safety levels is suggested in this chapter.

3. Improvement of the Durability of Concrete Structures using Ultra-High Performance Fibre Reinforced Concrete

3.1. CONCEPTUAL IDEA

This chapter presents an original concept for the improvement of concrete structures. The basic conceptual idea is to use UHPFRC only in those zones of the structure where the outstanding UHPFRC properties in terms of durability and strength are fully exploited; i.e. UHPFRC is used to “harden” the zones where the structure is exposed to severe environmental and high mechanical loading. All other parts of the structure remain in conventional structural concrete as these parts are subjected to relatively moderate exposure. Sustainable structures of the future will be those where the number and extent of interventions will be kept to the lowest possible minimum of only preventative maintenance without or only little disruptions of utilization.

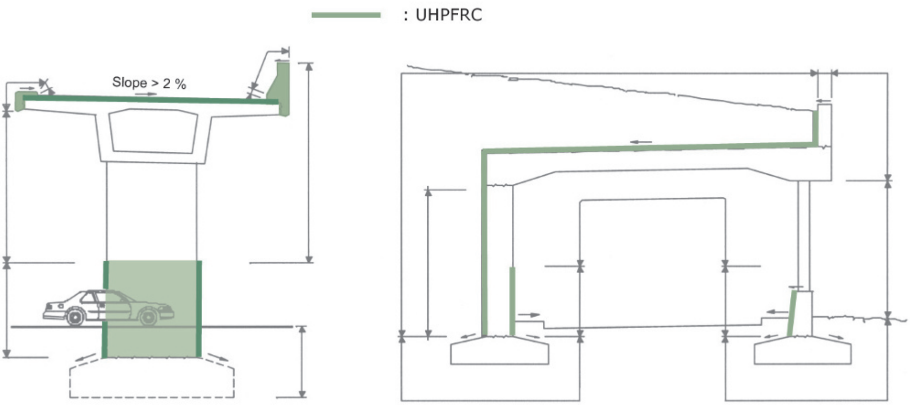

Figure 3. Concept of application of the local “hardening” of bridge superstructures with UHPFRC

This concept is applicable to new structures and for the rehabilitation of existing structures; it necessarily leads to composite structural elements combining conventional reinforced concrete and UHPFRC. The concept of application of UHPFRC is schematically illustrated on Fig. 3. An “everlasting winter coat” is applied on the bridge superstructure in zones of severe environmental and mechanical loads (exposure classes XD2, XD3).

Critical steps of the construction process such as application of waterproofing membranes or compaction by vibration can be prevented, and the associated sources of errors avoided. The construction process becomes then simpler, quicker, and more robust.

The waterproofing capabilities of the UHPFRC exempt from applying a waterproofing membrane. Thus, the bituminous concrete can be applied after only 8 days of moist curing of the UHPFRC. This constitutes a significant time saving with respect to the drying period of up to 3 weeks necessary prior to the application of a waterproofing membrane on a usual mortar or concrete.

The combination of the protective (P) and resistance (R) properties of UHPFRC with the mechanical performance of reinforcement bars (normal or high grade) provides a simple and efficient way of increasing the stiffness and load-carrying capacity with compact cross sections Fig. 4. Depending on the structural and material properties of the composite system, more or less pronounced built-in tensile stresses are induced in the UHPFRC due to restrained deformations at early age. This stress state needs to be analysed and evaluated (see Section 3.4).

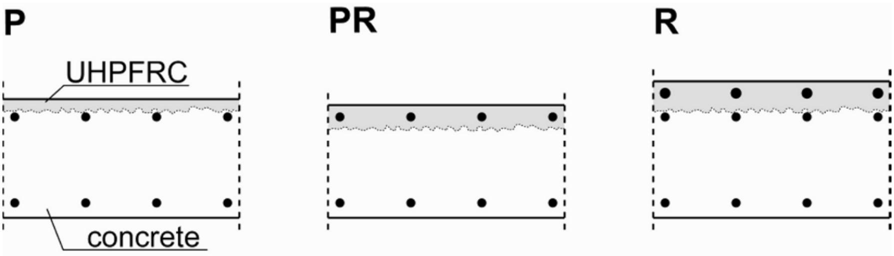

Figure 4. Basic configurations for composite structural elements combining UHPFRC and conventional structural concrete

The original conceptual idea (developed in 1999) has been investigated by means of extensive researches aimed at characterizing UHPFRC materials and the structural behaviour of composite structural members (Kamen et al., 2007, 2008; Charron et al., 2007; Habel et al., 2006a, b, c, d, 2007; Denarié and Brühwiler, 2006). The concept is well-suited for bridges and can also be implemented for buildings, galleries, tunnels or retaining walls.

3.2. PROPERTIES OF UHPFRC

3.2.1. *Tensile Behaviour of UHPFRC*

The uniaxial tensile behaviour of two different recipes of the UHPFRC CEMTEC$_{\text{multiscale}}$® type has been determined by means of a tensile test on unnotched dogbone specimens (Denarié and Brühwiler, 2006). The average curves from five tests for each material are represented on Fig. 5, showing the range of possible strain hardening responses. Both recipes are self-compacting.

Recipe CM0 is reinforced with a 468 kg/m^3 of a single type of 10 mm long steel fibres with an aspect ratio of 50. It has a water/binder ratio of 0.140, 1051 kg/m^3 cement, a fluid consistency (slump-flow = 700 mm) and is self-leveling.

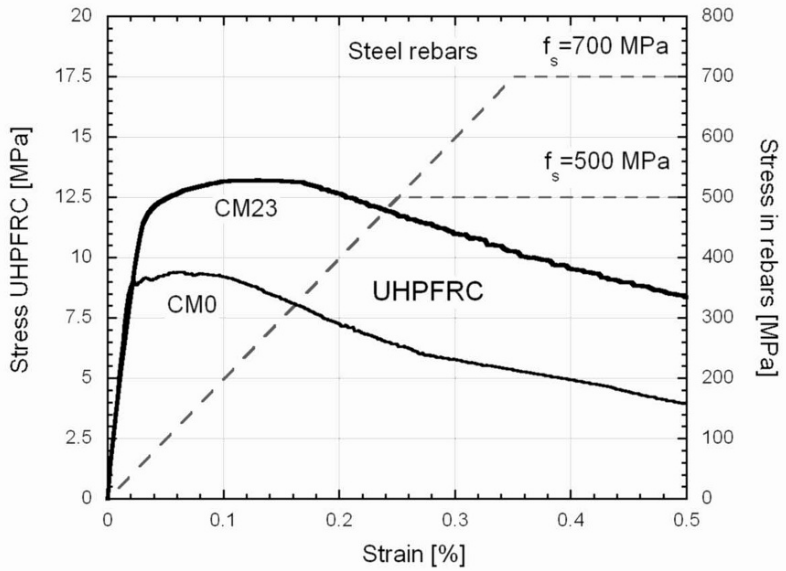

Figure 5. Tensile behaviour of two UHPFRC recipes, $\mathrm{CEMTEC}_{\mathrm{multiscale}}$®, unnotched tensile tests, fixed rigid boundary conditions, average curves at 28 days

Recipe CM23 has more binder (1,437 kg/m^3 cement) and a lower water-binder ratio (0.125). It is reinforced by a multilevel fibrous mix of macro steel fibres (10 mm long, aspect ratio 50) and microfibres (steel wool) with a total dosage of 705 kg/m^3. It can hold a slope of the substrate up to 2.5%. The effect of the addition of microfibres is revealed on Fig. 5 by three aspects:

1. The significant increase of the pseudo-elastic domain from 8 to above 11 MPa
2. The increase of the strain hardening domain
3. The increase of the load carrying capacity in the descending branch due to the indirect action of the microfibres on the progressive pull-out of the macro fibres

It is worth mentioning that the magnitude of strain hardening of UHPFRC such as $\mathrm{CEMTEC}_{\mathrm{multiscale}}$® (Rossi, 2002) falls into the range of the yield strain of construction steel, Fig. 5. This property opens up promising domains of combination of UHPFRC with reinforcement bars with high yield strength (700 MPa or above).

The fractured surface of a UHPFRC specimen after a tensile test shows numerous steel fibres, pulled out from the matrix. The work of pull-out of these numerous micro-reinforcements explains the extremely high specific work of fracture of UHPFRC (up to 30,000 J/m^2 compared to 200 J/m^2 for normal concrete). A significant part of the work of fracture of UHPFRC is dissipated in the bulk of the material, during the strain hardening phase, in the form of finely distributed, multiple cracks.

One should keep in mind that the mechanical response of fibrous composites such as UHPFRC is very much application dependent. Strong anisotropy effects can be induced by the casting procedure of the materials or the width and shape of the moulds and these effects have to be considered for the analysis of test results and for design (Wuest, 2007).

3.3. DESIGN EXAMPLE OF AN OVERPASS BRIDGE USING THE ORIGINAL CONCEPT

3.3.1. *Conceptual Design*

The above described conceptual idea is applied for the design and construction of the overpass bridge (Brühwiler et al., 2007). The principle is to use UHPFRC to "harden" only those zones where the structure is exposed to severe environmental action (direct water contact with deicing salts) and where high mechanical loads have to be taken up. These zones include the top surface of the deck slab, the kerb and sidewalk overlay elements and the zone above the middle support including a unique UHPFRC hinge. All other parts of the bridge structure remain in conventional reinforced and prestressed concrete as these parts are subjected to only moderate exposure. The developed bridge concept consists of a superstructure as a continuous two-span multi-girder system resting on a middle support and the abutments (Figs. 6 and 7).

The main girders are prefabricated in a construction plant and are prestressed and posttensioned in the longitudinal direction. After their alignment on the construction site, the space between the girders over the middle pier is filled out with UHPFRC creating at the same time an UHPFRC hinge over the finger piers (Detail 1, Fig. 8a). Then, a layer of UHPFRC is cast to connect the prefabricated girders along the longitudinal joint (Detail 2, Fig. 8b) and to provide a 3 cm thick (waterproofing) protection layer on the whole top surface of the deck slab. The kerb elements are made of prefabricated UHPFRC elements that are glued to the top surface. UHPFRC is thus applied both on the construction site and in the prefabrication plant by using different adapted mixes (SAMARIS, 2005).

The duration of the construction works shall be limited to a minimum in order to reduce disturbance for the road user. The total time of construction of the superstructure may be optimized and reduced to about 25–30 days. In addition, critical and time consuming steps of the construction process such as application of waterproofing membranes, on-site concreting or compaction by vibration are eliminated, and the associated sources of errors avoided. The construction process becomes then simpler, quicker and more robust with an optimal use of prefabrication.

The construction cost shall be similar or lower than for a conventional concrete bridge.

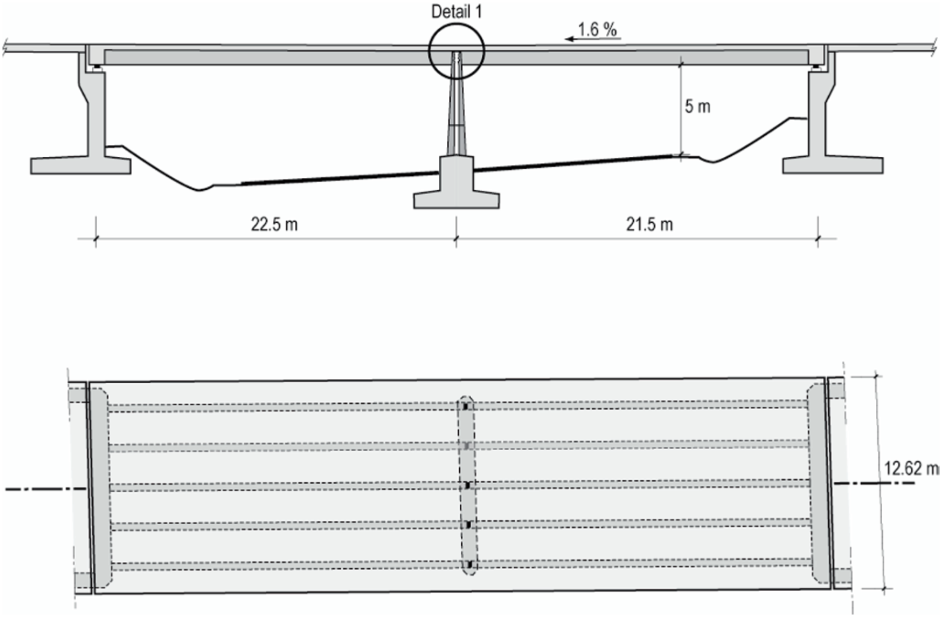

Figure 6. Elevation and plan view

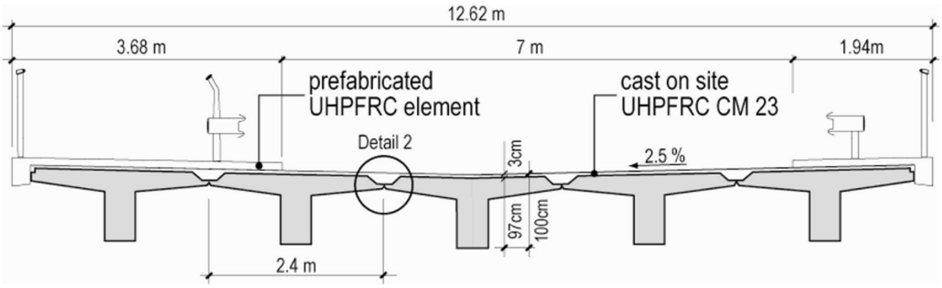

Figure 7. Typical cross section

The originality of Detail 1 (Fig. 8a) resides in the casting of the top part of the pier, hinge and connection between the prefabricated girders in one onsite casting sequence using UHPFRC. The 70 mm wide UHPFRC hinge is subjected to significant rotations requiring certain deformation capabilities of the UHPFRC. The top part of the middle support is a tension chord consisting of UHPFRC reinforced with steel reinforcing bars to take the tensile forces due to the bending moment over the support.

Detail 2 (Fig. 8b) provides a stiff load carrying connection between the prefabricated girders in the longitudinal direction. The 150 mm thick UHPFRC joint with steel reinforcement bars is highly resistant to account for concentrated wheel loads on the deck slab.

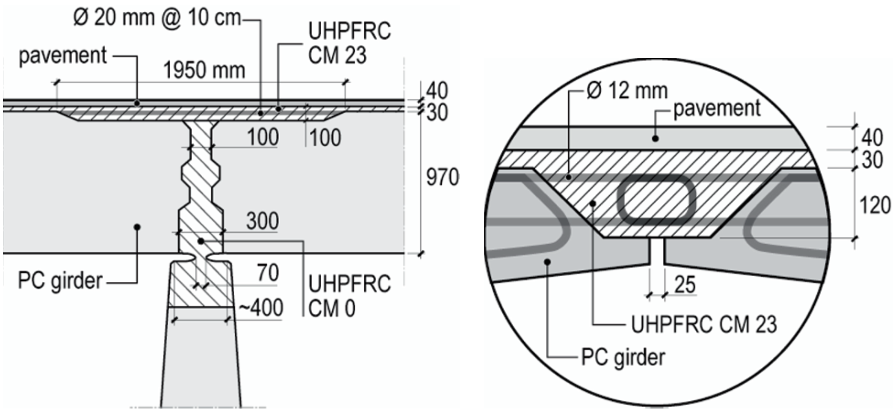

Figure 8. (**a**) Detail 1: middle support, and (**b**) Detail 2: longitudinal joint between prefabricated girders

Previous research indicated that a 30 mm thick UHPFRC layer provides the required mechanical performance and extremely low permeability. This UHPFRC protection layer contributes favourably to the load carrying behaviour of the deck slab in terms of stress membrane resisting against compression and tension forces without undergoing crack formation.

3.4. STRUCTURAL ANALYSIS OF A COMPOSITE BRIDGE GIRDER COMBINING UHPFRC AND REINFORCED CONCRETE

3.4.1. *Introduction*

This section shows findings of a FE-analysis of a conceptual bridge girder design combining a thin UHPFRC overlay with a reinforced concrete substructure (Fig. 7). Besides its load carrying contribution, the UHPFRC overlay replaces conventional waterproofing membrane and therefore has to remain in an impermeable state under service conditions and during the whole service life in order to protect the below conventional reinforced concrete structure effectively.

The present structural analysis focuses thus on the serviceability limit state of the structure.

3.4.2. *Constitutive Material Modeling of UHPFRC*

(a) Requirements
The UHPFRC designed for this application responds to the following general requirements:

- High compressive and tensile strengths
- Strain hardening and softening in tension
- Very low permeability
- Self-compacting fresh mix with the ability to be cast with a slope of 3%
- Low variability of mechanical properties

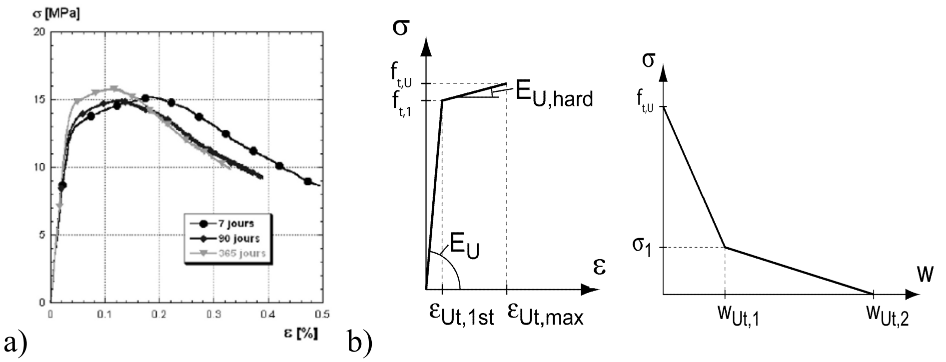

Figure 9. (**a**) Stress–strain diagram from experiments (Wuest, 2007) and (**b**) corresponding constitutive tensile law for FE-analysis input

(b) Tensile behaviour
High tensile strength as well as strain hardening and softening are characterising properties of UHPFRC. The uniaxial tensile behaviour was determined using dogbone specimens. The results of several experiments were averaged (Fig. 9a) and transformed into the constitutive material law for tension (Fig. 9b) as input for the FE-program.

(c) Viscoelastic behaviour
UHPFRC develops important shrinkage (mostly autogenous shrinkage) which leads to Eigenstresses in the composite element due to the restrained deformation conditions. Free shrinkage under drying conditions reaches up to 590 μm/m after 1 year with an evolution of two third of the value after 35 days (Kamen et al., 2007; Habel et al., 2006c). Induced stresses are partly balanced as a function of time by an important creep and relaxation capacity.

The high dosage of fibers prevents microcracking of the matrix and provides a high deformation capacity. Both effects are crucial for the proper working of the UHPFRC-layer regarding mechanical and physical requirements. Input data for the FE-analysis was deduced from comprehensive laboratory tests on the evolution of mechanical and physical properties depending on maturity (Kamen et al., 2007, 2008; Habel et al., 2006c, d).

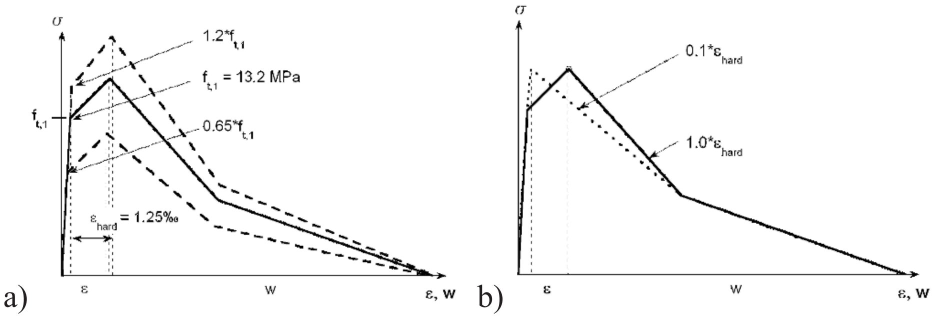

Figure 10. Variation of (**a**) tensile strength f_t and (**b**) deformation capacity ε_{hard}

(d) Variability of mechanical properties
An inherent property of fiber reinforced composites is the non-uniform fiber orientation and distribution depending on the mixing process, casting method and formwork boundaries (Wuest, 2007). This was taken into account in the FE-analysis by varying the values of tensile strength and deformation capacity (Fig. 10).

The tensile strength, defined as elastic limit of the material, was modified to 65% and 120% of the reference value $f_{t,1} = 13.2$ MPa (Fig. 9a), while the strain hardening, defined as the deformation between $f_{t,1}$ and the ultimate tensile strength $f_{t,U}$, was modified between 10% and 100% of the reference value $\varepsilon_{hard} = \varepsilon_{Ut,max} - \varepsilon_{Ut,1st} = 1.25‰$ (Fig. 9b). The ultimate tensile strength $f_{t,U}$ is considered to evolve with a constant factor of $1.25 * f_{t,1}$ in relation to the elastic limit.

3.4.3. *FE-Analysis*

(a) Description of the numerical tool
The FE-analysis was done with FEMASSE MLS (Roelfstra et al., 1994). This numerical tool allows to conduct comprehensive analyses including the coupling of age dependent thermal, hygral, chemical and mechanical properties.

In the given 2-D model the deformation in z direction (longitudinal sense of the bridge girder) was not restrained. The UHPFRC layer was applied to inert concrete, cured for 7 days and afterwards exposed to environmental conditions described by constant values at a temperature of 20°C. The numerical analysis starts at the instant of the UHPFRC overlay casting (time 0).

The structural analysis can be considered as representative regarding the observations made concerning the age-dependent variation of stresses and mechanical properties in the present structural element.

(b) Cross sectional model

The model represents an exemplary transversal cross section of the bridge girder near the support, showing five (prefabricated) T-beams in conventional prestressed concrete with the UHPFRC overlay (Fig. 11). The bottoms of the beams are vertically and horizontally fixed limiting the flexional deformability of the bridge deck to a minimum and increasing the degree of restraint to a maximum.

The loads are transferred laterally by the UHPFRC that also connects the longitudinal beams. The thickness of the layer is increased at the longitudinal joints to 15 cm instead of 3 cm on top of the T-beams.

In general, viscoelastic behaviour of the T-beams is beneficial for the stress evolution in the UHPFRC since it indirectly reduces the degree of restraint but this effect was not considered in the analysis.

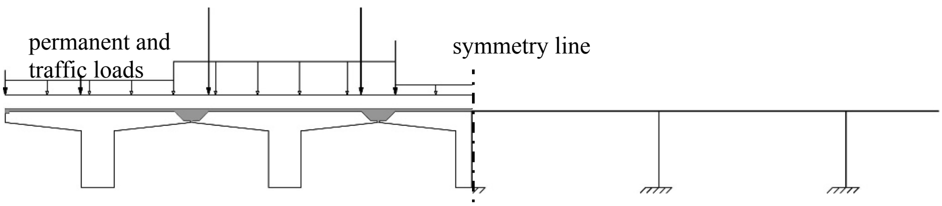

Figure 11. Transversal bridge section with UHPFRC layer, exemplary loading scheme and static system

(c) Load cases

The following load cases were considered in the modelling:

- Permanent loads due to self weight of the structure and the UHPFRC layer
- Permanent loads due to self weight of non-load bearing elements such as the curbs, crash barriers, railings and the asphalt layer, applied at 28 days
- Traffic loads at serviceability limit state including two traffic lanes according to current code provisions, applied at the most unfavourable position regarding stresses in the UHPFRC layer in the transversal sense

All external loads are superimposed to the continuous evolution of the mechanical and physical properties of UHPFRC such as Young's modulus, compressive and tensile strength, shrinkage and viscoelasticity.

3.4.4. *Results*

The results of the numerical simulation are presented exemplary for a typical reference point showing highest stresses:

(a) Restrained shrinkage
Restrained shrinkage is the load case the UHPFRC overlay is subjected to from the very beginning (after casting) and also the one that consumes an important part of the resistance capacity of the material. It is a deformation controlled loading process, and consequently, it is the deformation capacity of the material that predominantly replies to this load case (Fig. 12). The absolute value of the tensile strength is not of great importance here.

Depending on the level of the evolving tensile strength, shrinkage causes Eigenstresses that almost reach the elastic tensile strength $f_{t,1}$ before the external loads are applied. If the UHPFRC overlay is stressed beyond its elastic limit due to restrained shrinkage it enters into the hardening domain where it possesses an important deformation capacity, i.e. the stress increase at this stage will be very small.

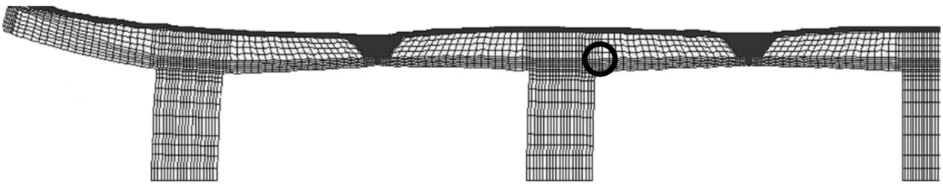

Figure 12. Deformed shape (not in scale) due to shrinkage of UHPFRC (without external loads) and location of the reference point

In this way, the strain hardening behaviour represents a significant stress release potential which is essential for the structural response of the overlay in terms of avoidance of macrocrack localisation and maintaining the low permeability of the UHPFRC layer.

(b) Permanent and traffic loads
Permanent and traffic loads are applied 28 days (672 h) after the application of UHPFRC. They represent a force controlled load case. The UHPFRC layer is subjected to an immediate stress increase which is superimposed to the stresses induced by restrained shrinkage.

Figure 13 shows the stress and strength evolution on the time axis until 42 days (1,000 h) for three cases. The lower dotted lines represent the elastic strength evolution $f_{t,1}$ whereas the upper dotted lines show the evolution of the ultimate strength $f_{t,U}$. The solid lines describe the stress evolution at the reference point. The step in the solid lines marks the point in time of external load application.

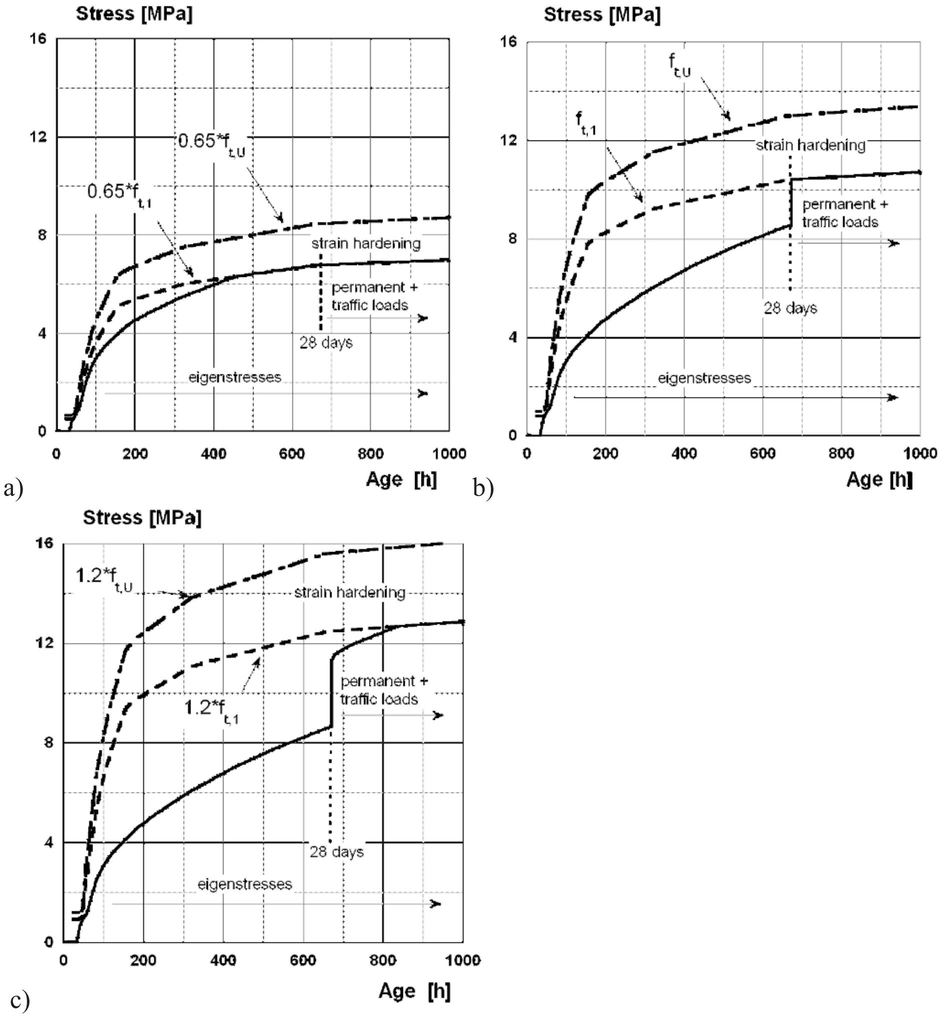

Figure 13. Evolution of stress and strength for (**a**) $0.65^*f_{t,1}$; (**b**) $1^*f_{t,1}$ and (**c**) $1.2^*f_{t,1}$

In case of $f_{t,1} = 0.65^*f_{t,1,ref}$ (Fig. 13a) the stresses due to restrained shrinkage reach the elastic limit before the external loads are applied. The application of these creates an inelastic response in the hardening domain. The important deformability of the strain hardening domain keeps the stress level in the UHPFRC layer very close to its elastic limit. The ongoing shrinkage does not significantly raise the stresses. The stress evolution closely follows the elastic strength evolution.

In case of $f_{t,1,ref}$ (Fig. 13b) the stress increase due to the external loads at 28 days is balanced partly by an elastic response and partly by the strain hardening of UHPFRC. The global behaviour is again similar. Once the stress level exceeds the elastic limit further stress increase is very small.

In case of $f_{t,1} = 1.2 * f_{t,1,\ ref}$ (Fig. 13c) external loads lead to a purely elastic response of the overlay. The loads induce a principal tensile stress of approximately 3 MPa at the reference point. Further stress increase is then induced by the continuing shrinkage until the stress level reaches the elastic limit and subsequently follows it.

In all the cases, it can be seen that once the material exceeds its elastic limit strength external loads and continuing shrinkage do not cause a significant further stress increase. The redistribution of loads and increased deformability due to the pronounced strain hardening behaviour and loss of stiffness prevent further stress increase in the UHPFRC layer. Therefore, it is unlikely that the tensile strength $f_{t,U}$ of the UHPFRC is reached. The UHPFRC layer thus remains at the initial stage of multiple microcracking without developing localised macrocracks. The material enters merely very little into the hardening domain, thus keeping its low permeability. It is not subjected to softening within the considered period of 1,000 h.

An exemplary simulation with traffic loads increased by a hypothetical factor of 3 shows that in fact the stress step continues significantly into the hardening domain if the level of loading is sufficiently high (Fig. 14a). Then the

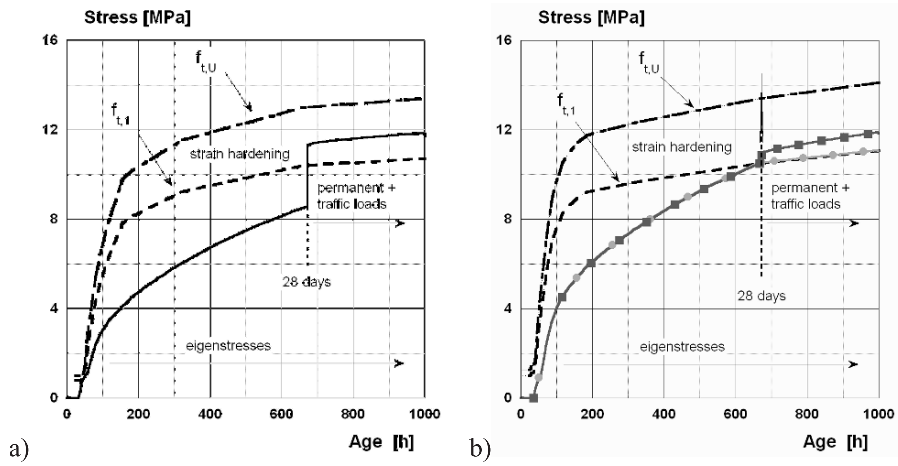

Figure 14. (**a**) Evolution of stress and strength for $1.0 * f_{t,1}$ and three times the external loads; (**b**) influence of ε_{hard} on the stress level for $\varepsilon_{hard} = 0.1 * \varepsilon_{hard,ref}$

stresses evolve parallel to the elastic tensile strength but at a higher level. UHPFRC seems to possess enormous reserves in a setup as described above to resist localised cracking even if its elastic tensile strength is exceeded.

In case the strain hardening capacity is reduced significantly the UHPFRC overlay obviously enters far into the hardening domain, (Fig. 14b). The two lines with markers show the stress evolution at a reference point for two materials with $0.1{*}\varepsilon_{hard,ref}$ (upper line with square markers) and full strain hardening capacity (lower line with round markers).

(c) Serviceability and waterproofing

Charron et al. (2007) have shown for a UHPFRC respecting the requirements given in 2.1 that the water permeability of UHPFRC remains low ($K_{w\ equiv.} < 2 \times 10^{-8}$ cm/s) until a tensile deformation of 1.3‰. This threshold deformation corresponds to a cumulated crack opening equal to 0.13 mm as compared to 0.05 mm for normal concrete.

Since the numerical results show that at all levels of tensile strength the principal stresses in the UHPFRC overlay do not significantly enter into the strain hardening domain, the proposed concept of an "impermeable" and waterproof UHPFRC layer is validated.

3.4.5. *Synthesis of Findings*

A structural analysis of a composite bridge girder combining reinforced concrete and UHPFRC at the serviceability limit state was performed. The structural response under combined loading due to restrained shrinkage and traffic loads was investigated. The obtained results show:

- Restrained shrinkage and external loads may generate stresses close to the elastic tensile strength in the UHPFRC overlay of the composite element with a high degree of restraint. The stresses then follow the age-dependent elastic strength evolution of UHPFRC.
- The evolution of applied stresses in the strain hardening domain is independent of the level of the elastic tensile strength. The loss of stiffness of the UHPFRC layer as it enters into the hardening domain causes a stress release and redistribution.
- The risk of transverse cracking of the UHPFRC layer in the presented structural configuration is unlikely due to the increased deformation capacity and significantly lower stiffness at strain hardening.
- Strain hardening is an essential property for the described type of application since it allows maintaining the low permeability of UHPFRC in its function as waterproofing layer.

3.5. APPLICATIONS

The original concept of application of UHPFRC for the improvement of structural concrete has been validated by means of applications. Four applications have been conducted until today which will be described in the following (Brühwiler and Denarié, 2008).

3.5.1. *Rehabilitation and Widening of a Road Bridge*

A short span road bridge with busy traffic has been rehabilitated and widened using UHPFRC. The entire deck surface of the bridge with a span of 10 m was rehabilitated in three steps during autumn 2004 (Fig. 15).

Firstly, the downstream kerb was replaced by a new prefabricated UHPFRC kerb on a new reinforced concrete beam which was necessary for the widening. Secondly, the chloride contaminated concrete of the upper surface of the bridge deck was replaced by 3 cm of UHPFRC in two consecutive steps such that one traffic lane could be maintained open. Thirdly, the concrete surface of the upstream kerb was replaced with 3 cm of UHPFRC.

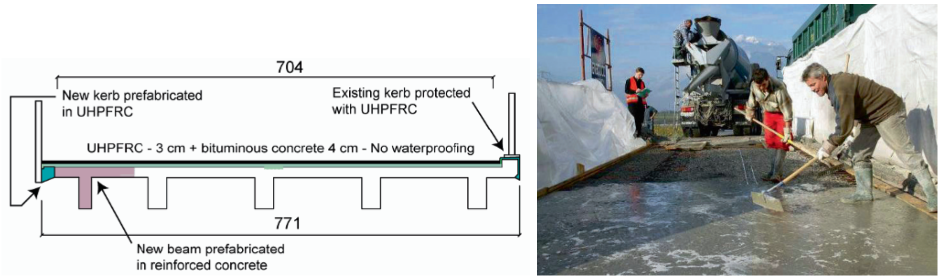

Figure 15. Bridge cross section after rehabilitation (dimensions in cm) and UHPFRC casting.

The fresh self-compacting UHPFRC material was prepared at a local concrete prefabrication plant with a standard mixer, brought to the site by a truck and then poured on the hydrojetted deck surface. The UHPFRC was easy to produce and place with standard tools and very robust and tolerant to the unavoidable particular site conditions. The bituminous pavement was applied on the UHPFRC surfaces after 8 days of moist curing, and the corresponding lane was reopened to traffic the next day. The bridge was fully reopened to traffic one month after the beginning of the construction work.

3.5.2. *UHPFRC Protection Layer on a Crash Barrier Wall*

A layer of UHPFRC has been applied in September 2006 to the concrete crash barrier walls of a highway bridge (Oesterlee et al., 2007). The main design requirement was to obtain long-term durable crash barrier walls since traffic interruption for future rehabilitation interventions are prohibitive due to the very high traffic volume on this highway. Long-term durability is obtained when transverse macro-cracks in the UHPFRC layer are absent and the permeability of UHPFRC layer to ingress of water and chloride ions is extremely low.

Figure 16 shows the crash barrier wall with a UHPFRC layer covering the areas subjected to splash exposure (Class XD3: reinforcement corrosion induced by chlorides). The rheological properties of UHPFRC were adapted for easy pouring into the 3 cm wide formwork to fill a height of 120 cm including a small horizontal part at the bottom of the wall that provides continuity with the conventional bridge deck with a waterproofing membrane.

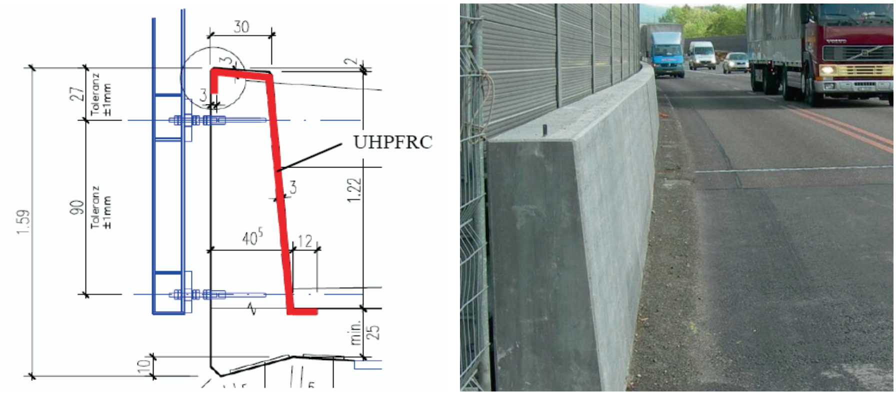

Figure 16. Typical cross section of the crash barrier wall and view after rehabilitation

Due to restrained early age deformation of the UHPFRC (mostly due to thermal and autogenous shrinkage) bonded to the existing reinforced concrete wall, an internal stress state is built up in the composite element including, in particular, tensile stresses in the UHPFRC layer. These tensile stresses, which can cause macrocrack formation, and the capacity of the UHPFRC to resist to these stresses were investigated by means of numerical analyses prior to the intervention.

The fresh self-compacting UHPFRC was fabricated in a conventional ready mix concrete plant, transported to the site by a truck and filled into the thin slot to realize the UHPFRC coating. The required mechanical properties and the protective function of the UHPFRC layer have been confirmed by in-situ air permeability tests and laboratory tests on specimens cast on site. Four months after application no crack could be found confirming the predictions made by the numerical simulations.

3.5.3. *Rehabilitation of a Bridge Pier using Prefabricated UHPFRC Shell Elements*

In this application, 4 cm thick UHPFRC shell elements have been prefabricated to form an outer protection shield for the existing 40 year-old reinforced concrete bridge pier which is located very close to a busy highway which makes it virtually not accessible for future maintenance interventions (Fig. 17).

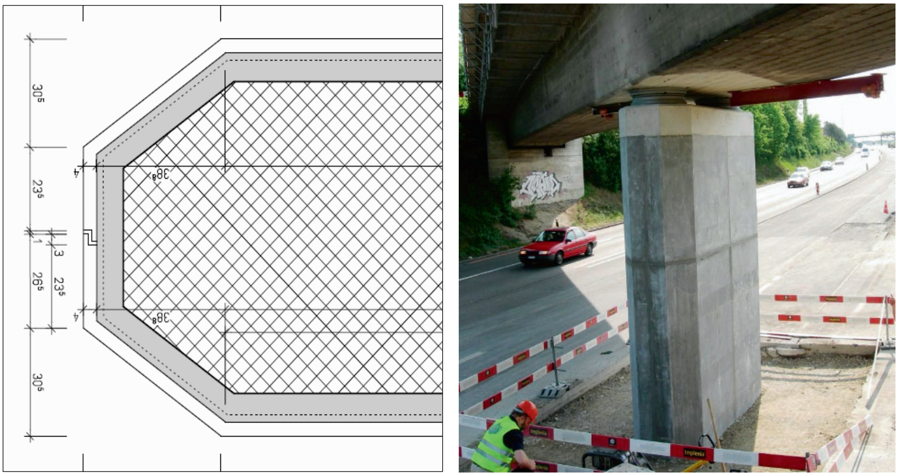

Figure 17. Cross section and general view of the rehabilitated bridge pier

In spring 2007, the UHPFRC elements (maximum element height of 4 m) were cast in a prefabrication plant, transported to the construction site and mounted, after removing of up to 10 cm of chloride contaminated concrete by hydrojetting. The joints between the different UHPFRC shell elements were glued using an epoxy resin. The remaining space between the UHPFRC elements and the existing reinforced concrete was filled with self-compacting mortar.

Long-term durability is expected since transverse cracks in the UHPFRC protection shield are absent and the permeability of UHPFRC for ingress of water and chloride ions is extremely low as confirmed by permeability tests.

3.5.4. *Strengthening of an Industrial Floor*

The 50 year-old drivable reinforced concrete floor of a fire brigade building had insufficient load carrying capacity in view of heavier future fire engines. The concept was to increase the load carrying capacity of the existing slab of 720 m^2 area by pouring a 4 cm thick UHPFRC layer on top of the existing RC slab, as a replacement of the existing cementitious non-load carrying overlay (Fig. 18). The UHPFRC layer leads to a thicker load carrying slab which provides (1) a better distribution of local wheel loads, (2) an increase in static height and (3) a layer of high strength material capable of resisting both compression and tension stresses.

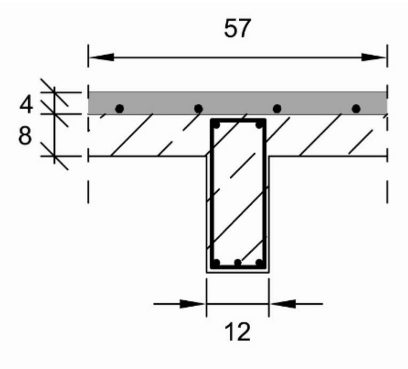

Figure 18. Cross section (dimensions in cm) with UHPFRC layer (in grey) and view of UHPFRC casting performed in autumn 2007

The UHPFRC was again fabricated in a local ready mix concrete plant and transported to the site by trucks. The excellent workability of the fresh self-compacting material allowed for easy casting. The use of the UHPFRC solution turned out to be very economic (compared to the conventional solution of slab demolition and reconstruction). This was also because the utilization of the fire workers building was only slightly restricted during the intervention, holding user costs down.

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