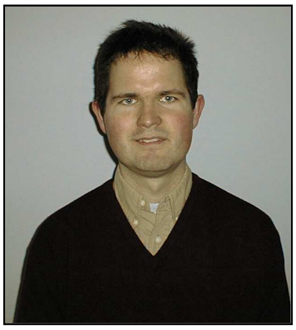
Experimental Investigation of Combined Thermal and Mechanical Behaviour of Danish and Swedish Concrete subject to High Temperatures



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ABSTRACT

A total of 60 high temperature tests have been carried out on three different concretes heated to 600 °C (one Swedish and two Danish concretes). Experimental evidence of thermal strains during heating with or without sustained loading is reported. The registered strains under transient temperature conditions are subdivided into purely mechanical strain, free thermal strain and thermo-mechanical strain. Furthermore, analytical models are proposed to predict the temperature dependency of the individual strain components. One important finding of the tests is that the Swedish concrete show only half of the free thermal strain that is observed for the Danish concretes at 600 °C. This difference is meant to be due to the differences in aggregate type.

Key words: concrete, high temperatures, thermal strain, transient creep, heating.

1. INTRODUCTION

Thermal exposure under fire is one of the most severe actions a concrete structure can be subjected to. The two main constituents of concrete react totally different against first time heating where the aggregate is expanding and the cement paste is shrinking under expulsion of moisture. The incompatible deformations may inflict severe damage to the concrete followed by degrading strength and stiffness. For a thorough description of the involved mechanisms the reader is referred to the literature, see e.g. [1-4].

In order to model the behaviour of a concrete structural member subject to heating, e.g. during a fire scenario, it is necessary to have some knowledge of the material response in the high temperature region of several hundred degrees. This response is complicated for the following reasons: (i) it is a combination of two opposite mechanisms - expansion of aggregate and shrinkage of cement paste – being experimentally difficult to separate; and (ii) the stress-strain response depends significantly on a combination of the stress history and the temperature history. Furthermore, the whole area of material testing under high temperatures suffers from a lack of test standards.

The present investigation is an experimental attempt to quantify the mechanical behaviour of typical Danish and Swedish concrete subject to combined thermal and mechanical actions. It has not been the scope of the investigation to give a full explanation of all the mechanisms and phenomena observed during the tests. The scope is rather to define a test programme that are fairly easy to carry out and furthermore, provides valuable/reliable information of the concrete behaviour that can be implemented in practical design of concrete members. The tests also provide information on the degree of experimental scatter to be expected.

The reported test programme is a part of a large project on environmentally friendly concrete technology called "Green Concrete". The concrete mixes being subject to high temperature testing have also been tested thoroughly with respect to mechanical properties, fresh properties and durability. The general conclusion of that project is that the green concretes basically behave similar to the conventional concretes (www.greenconcrete.dk).

2. EXPERIMENTAL PROGRAMME

High temperature tests are performed in a gas-fired furnace at the Fire Laboratory, Aalborg University. A test rig is established with test cylinders placed horizontally instrumented with quartz bars (Fig. 1). These quartz bars are lead through the furnace wall to the outside where they are connected to displacement transducers. Three quartz bars register the displacements of each end of the specimen. The measuring length is 400 mm.

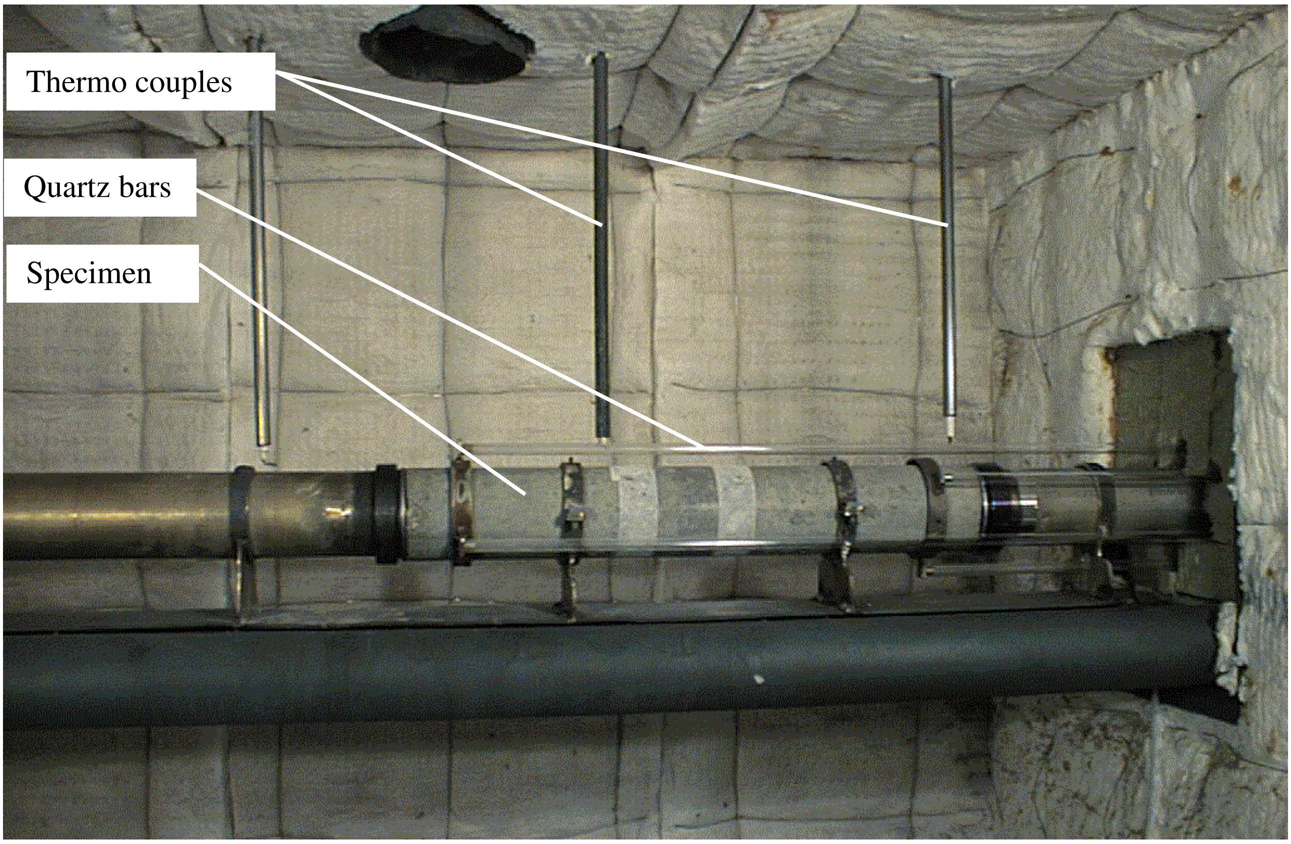


Figure 1 – Inside of furnace with test specimen. The dimensions of the test cylinders are diameter 83 mm and length 575 mm. Each specimen is equipped with two thermo couples. One is cast in the centre of the specimen the other is cast to a depth of approximately 10 mm.

2.1 Concrete mixes

Three different concretes are tested according to the programme given in the next section. One Swedish mix design and two Danish mixes are applied (Table 1).

Table 1 – Concrete mixes in kg/m³. Notes: [a] Danish rapid hardening cement (CEM I) for PR and P5 and Swedish Bygg C cement (CEM II) for P7. [b] Concrete slurry. [c] Fly ash from biological incineration. [d] Max. size 32 mm. [e] Max. size 25 mm. [f] Equiv. ratio based on reactivity factors of 0.5 for fly ash and 2 for silica. [g] Mean values measured on 100×200 mm cast cylinders.

Material	Concrete PR (Danish reference concrete)	Concrete P5 (Danish green concrete)	Concrete P7 (Swedish green concrete)
Cement[a]	170	168	194
Fly ash	60	60	-
Silica fume	14	13	-
Waste product	-	18[b]	79[c]
Water	143	156	167
Sand	725	692	1008
Coarse aggregate	1170[d]	1172[d]	904[e]
Plast. (Conplast 212)	0.75	0.71	-
(Peramin V)	-	-	0.45
Super plast. (Peramin F)	-	-	0.71
Equiv. w/c[f]	0.63	0.70	0.71
28 days strength[g]	35 MPa	32 MPa	29 MPa
Age at testing	3 – 6 months	4 – 6 months	5 – 8 months

The concretes are typical Danish/Swedish mix designs for indoors structural use (passive environmental exposure class). Due to the fact that the experiments are an integrated part of a large project on environmentally friendly concrete various waste products are reused in the mixes. Concrete PR is a Danish reference mix without any green modifications. Concrete P5 corresponds with PR substituting 2-3 % of the sand fraction with concrete slurry. Concrete P7 is a typical Swedish mix design where some of the cement has been replaced with fly ash from biological incineration.

The main difference between the three concretes lies in the types of aggregate. The volumetric content of aggregate is calculated to 73, 72 and 71 % for PR, P5 and P7, respectively, making the results directly comparable. In PR and P5 the sand is quartz sand and the coarse aggregate is uncrushed sea material, which may be flint, granite, limestone etc. The Danish sea material is known to be a very inhomogeneous material. No detailed mineralogical analysis of the aggregate has been performed. The Swedish concrete contains crushed rock material consisting of amfibolite, diabase and gneiss, i.e. siliceous aggregate.

After casting the specimens are air-cured in the laboratory wrapped in plastic. A month before testing the plastic for is stripped and the ends are saw-cut in order to obtain plane and parallel specimen ends. The tests are performed over a 3-month period after at least 3 months of curing.

2.2 Test execution

Three different types of experiments are performed (Fig. 2). Uniaxial compressive stress – strain curves are recorded at three different nominal temperature levels (200, 400 and 600 °C) beside room temperature. These stress – strain tests are performed without any load applied during heating and cooling. Furthermore, transient tests are performed where the specimen is subject to constant stress during heating and cooling and finally tests where the specimen elongation is restrained. Two specimens are repeated for each set of test conditions, giving about 20 specimens for each of the three concretes.

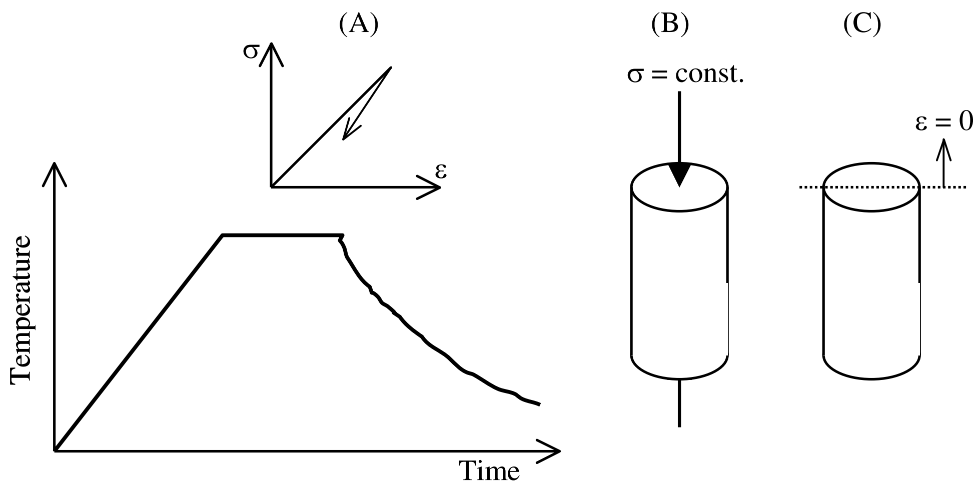


Figure 2 – Experiments with heating and loading of specimens. Test A: Stress – strain curves at constant elevated temperature levels. Test B: Heating and subsequent cooling under constant compressive stress (= 0, 11, 22, 44 % of cold strength). Test C: Restrained elongation.

The temperature of the furnace is recorded as the mean value of 3 thermo couples located just above the specimen (Fig. 1) together with the specimen temperature. In Fig 3 an example of a heating and cooling scenario is depicted. The heating rate is 3-4 °C per minute and it is maintained by manually controlling the gas temperature in the furnace. Turning off the burners and leaving the specimen inside the furnace provide the cooling process.

The temperature difference is seen to increase rapidly to a level of about 50 °C followed by a further increase to 75 °C due to the vaporisation and escape of free moisture from the specimen. This effect manifests itself by delaying the temperature increase within the central part. The cooling process reverses the temperature difference. It is seen how the specimen experiences a thermal chock within the first half-hour of cooling.

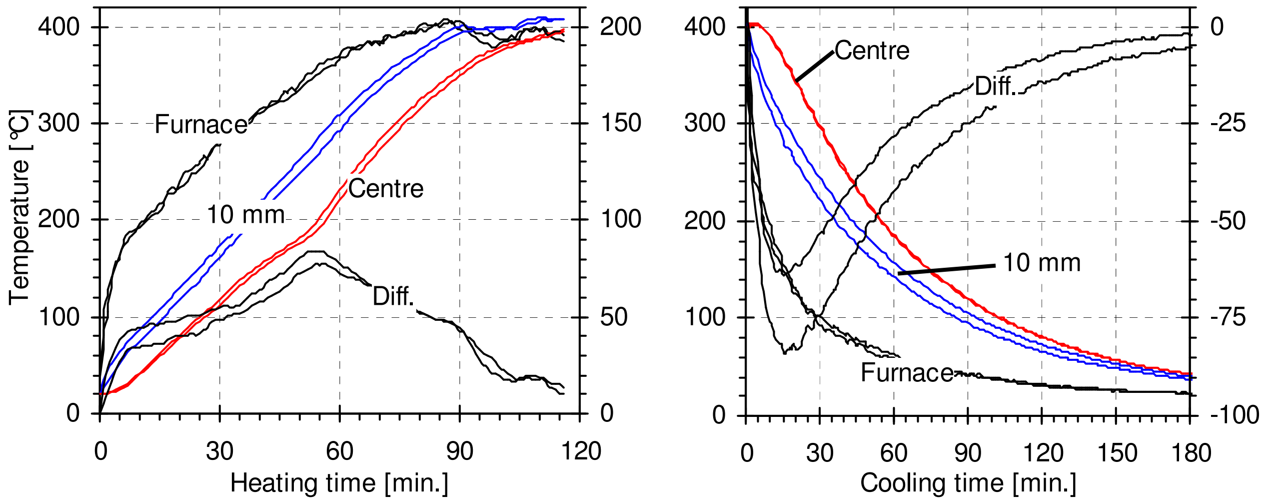


Figure 3 – Temperature recordings shown for two specimens heated to 400 °C with subsequent natural cooling. The temperature differences between centre and 10 mm depth are depicted on the right-hand axes.

3. EXPERIMENTAL RESULTS

3.1 Mechanical properties

The 28-days compressive strengths measured on standard cylinders are given in Table 1. The reference strength at room temperature, f_c^0, is also determined on cylinders saw-cut from the test specimens at the time of testing. The average strengths determined on these cylinders with diameter 83 mm and length 150 mm are 41, 39 and 30 MPa for PR, P5 and P7, respectively.

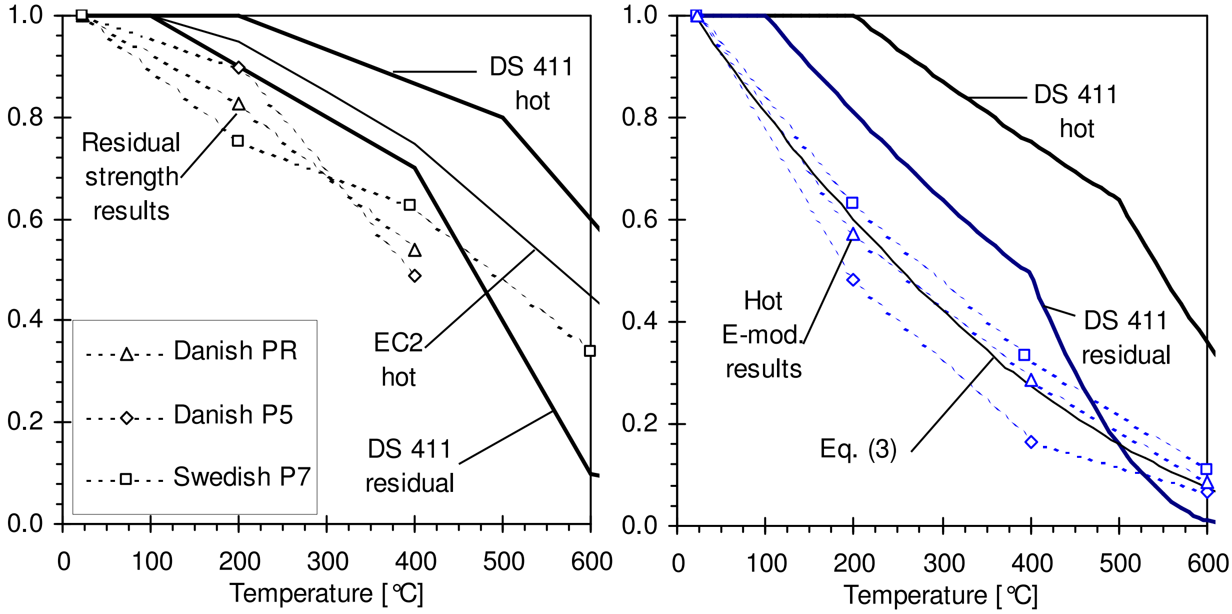


Figure 4 – Strength and stiffness reduction factors. Left: measured residual strengths normalised with respect to the cold strength f_c^0. Right: measured hot E-modulus normalised with respect to the cold reference value E_c^0. A reduction factor is defined as the ratio between the property measured after temperature exposure divided by its value at room temperature.

Test A (Fig. 2) is used to observe the effect of heating on the E-modulus. After reaching a certain maximum temperature the specimen is loaded up to about half its strength level and unloaded before it is left to cool back to room temperature. For reference the loading is also applied to specimens at room temperature. The reference E-modulus is determined to 35, 30 and 27 GPa for PR, P5 and P7, respectively.

After cooling the test specimens used in test A are saw-cut into short cylinders. Then after a couple of weeks in the laboratory they are tested for the residual compressive strength. It is assumed that the load cycle at high temperature does not influence the residual strength.

Figure 4 depicts how the residual strength and the hot E-modulus are found to decrease with the nominal maximum exposure temperature during heating. It is seen that for PR and P5 the residual strength after 600 °C is not determined because the specimens were too fragile to handle after cooling. The observed residual strength degradation is found to be slightly below the behaviour suggested by the Danish concrete code DS 411 [5].

Figure 4 also includes the hot strength reduction factor suggested by Eurocode 2 [6] for siliceous concrete together with the reduction factors given by the DS 411 [5] for strength and stiffness. DS 411 denotes the strength reduction factor by ξ_c and it defines the reduction factor on the E-modulus as $\xi_E = \xi_c^2$, a relationship that is originally proposed by Hertz [2].
Regarding the reduction factor on the E-modulus it is seen that the test results indicate a much more severe decrease than what is recommended in DS 411 for hot conditions. Note that DS 411 applies a convex relationship whereas the experimental evidence suggests a concave relationship, which is also supported experimentally in other tests [3,4].

3.2 Free thermal strains

During heating without external restraints the aggregate expands while the cement paste shrinks due to moisture being expelled (first the free evaporable water followed by chemically bound water). Thus, the thermal strains of concrete are based on two mechanisms of opposite sign. However, the thermal expansion of concrete is mainly governed by the properties of the aggregate and especially the content of quartz is important [1-4]. Furthermore, the magnitude of the thermal strains depends on the initial moisture content and the rate of heating. In [3] it is shown how a wet concrete sample shows smaller thermal strains than a dried-out sample due to the increased shrinkage. The same applies for a sample that is heated very slowly.

The dependency of the free thermal strains on the concrete temperature is depicted in Fig. 5 for heating and subsequent cooling. During heating the two Danish concretes PR and P5 behave almost identically showing much larger thermal strains than P7 especially after exceeding 400 °C. This significant difference in behaviour is assumingly due to the Danish aggregate not being as thermally stable as the Swedish aggregate. At 600 °C concrete P7 shows only half the thermal strain of PR and P5.

The fact that PR and P5 show very good experimental agreement in Fig. 5 indicates that the green modification of P5 (adding of concrete slurry, Table 1) does not alter its behaviour with regard to high temperatures.

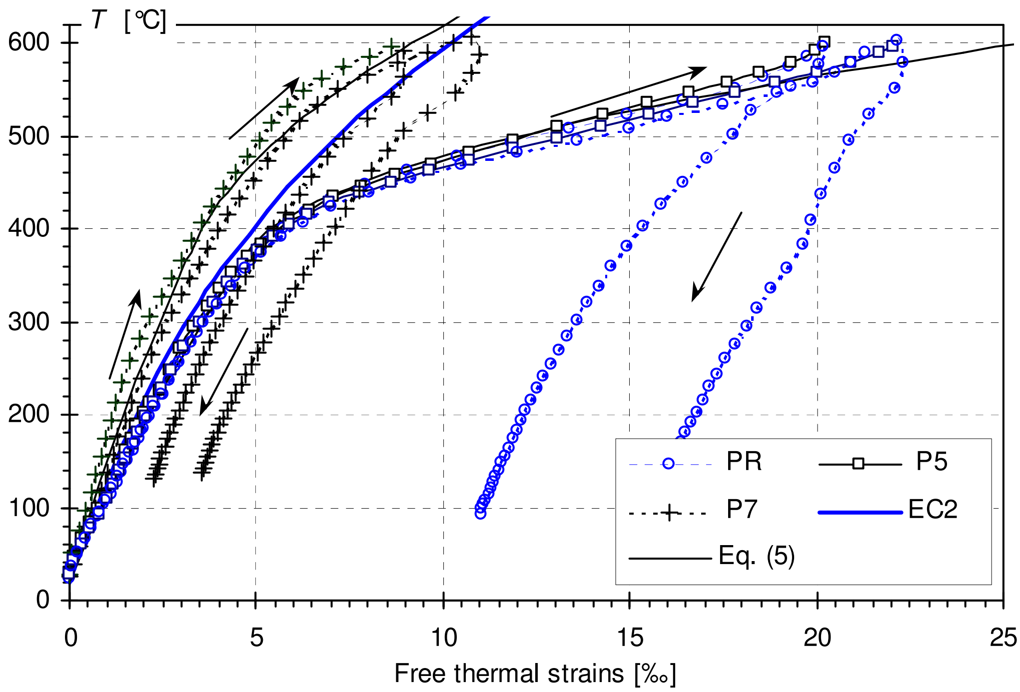
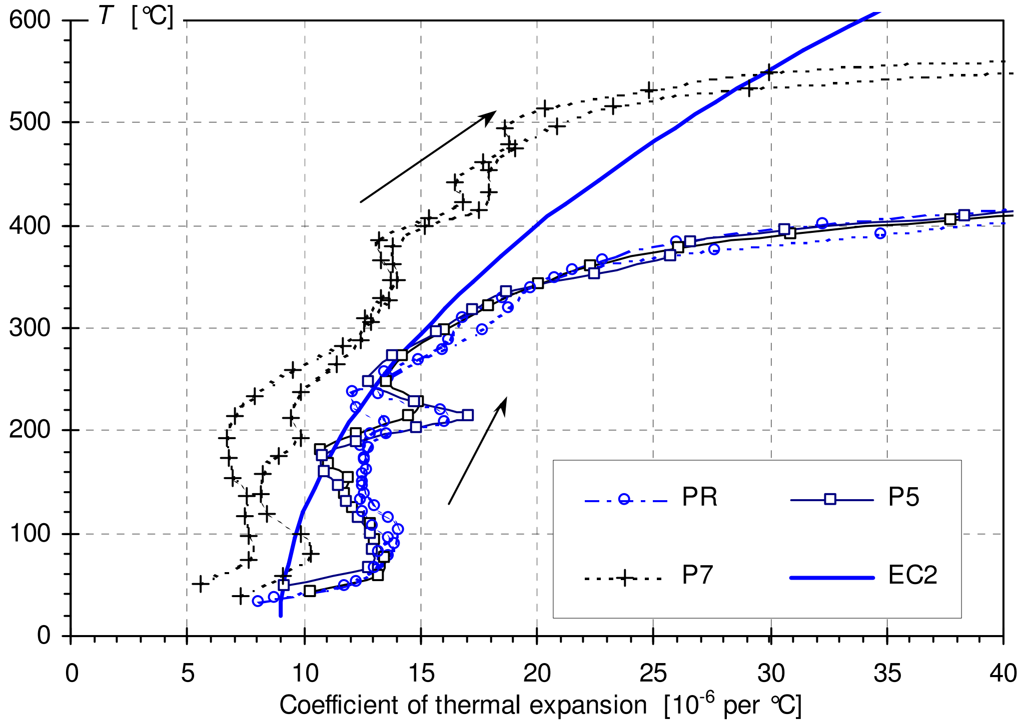


Figure 5 – Recorded thermal strains vs. temperature. The bold line depicts the function recommended in [6] for concrete with siliceous aggregates.

The P7 results are found to be in good agreement with those reported in [3] and the PR and P5 results are comparable in magnitude with results on gravel concrete reported in [1]. Figure 5 also contains a 3rd order polynomial recommended by Eurocode 2 [6]. This expression is seen to be slightly higher than P7 at all temperatures.

During cooling it is seen that the concretes behave almost identically corresponding to a thermal coefficient of expansion equal to approximately 15×10^{-6} per °C (Fig. 5). It is noted that the cooling process was not registered for concrete P5 by mistake.

After cooling back to room temperature the Danish concrete PR is seen to have a significant residual elongation due to cracks in the binder. The Swedish concrete P7 also showed some residual elongation about an order of magnitude smaller than PR.



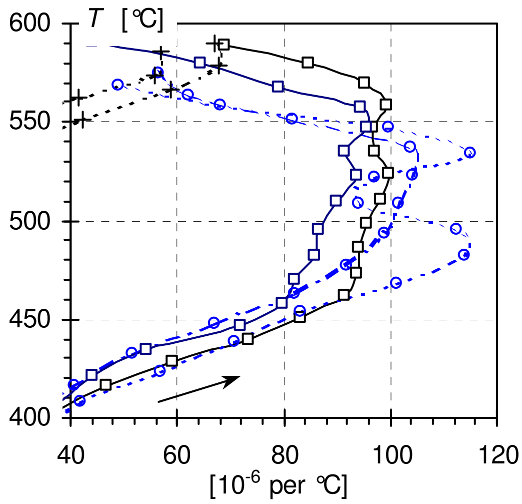


Figure 6 – Derivative of strain data in Fig. 5 with respect to temperature during heating. The final part of the curves is shown in the small diagram.

In Fig. 6 the thermal strain data are differentiated numerically with respect to temperature to obtain the coefficient of thermal expansion $\alpha = d\varepsilon_{th}/dT$. This diagram again demonstrates how the Swedish concrete shows significantly smaller expansion than the Danish concretes.

The significant increase in α found for P7 between 500 and 550 °C is probably due to inversion of quartz. For PR and P5 the coefficient of expansion changes from 10 to 15×10^{-6} per °C below 300 °C up to almost 100×10^{-6} per °C at 500 °C. Then the coefficient starts to decrease back to zero. However at this point the heating is stopped and the post 600 °C behaviour is not observed in the present investigation.

The variation of α in the temperature range 100 – 250 °C is due to the transport of free moisture that is taking place (cf. temperature recordings in Fig. 3). These variations are not detected during the cooling process.

3.3 Transient thermo-mechanical strains

General description

The term creep is used for time dependent deformations in concrete. Creep deformations typically take place over a time range of months and even years. An unsealed and unloaded test determines shrinkage and a sealed loaded test determines basic creep. However, if the creep test is carried out unsealed the total strain exceeds the sum of shrinkage and basic creep (Fig. 7). The difference is most often termed drying creep or load-induced shrinkage [7].

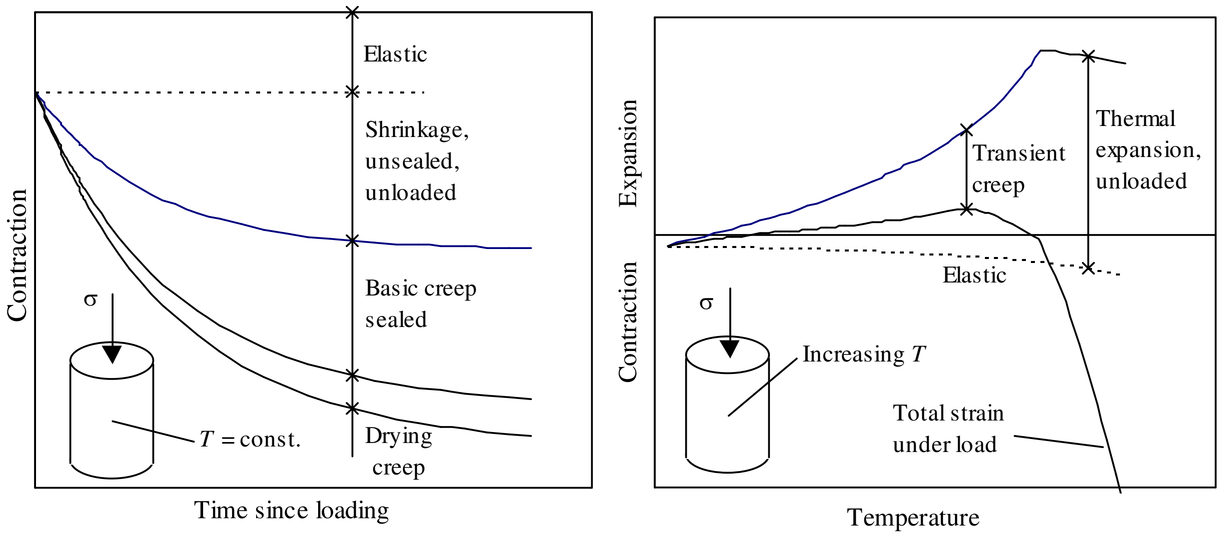


Figure 7 – Illustration of conventional creep strains (left) and transient thermal creep (right).

When concrete is heated there is a large difference whether the temperature conditions are stationary or transient. Under transient conditions the total strains are found not to consist of only free thermal strains and mechanical strains. The total strains also contain a component, which is often termed transient creep (Fig. 7). However, the term creep seems to be misleading since conventional creep also exists under isothermal conditions [3,4,8]. Furthermore, a transient-heating situation is most often relatively short-term and therefore the name thermo-mechanical strain is used instead of transient creep. The phenomenon is also sometimes termed load-induced thermal strain [1]. The thermo-mechanical strains only occur during heating and not during subsequent cooling as it are demonstrated in the following. Furthermore, thermo-mechanical strains are reported to occur only during first heating [1]. In the present section the results are briefly presented and in the next chapter they are further analysed.

Test results

Figures 8 and 9 depict the strain recordings during tests of type B. The strains do not include the mechanical strain from initial loading. Three compressive stress levels are applied beside the unstressed tests shown in the previous section (Fig. 5). The effect of combined heating and stress is clearly seen. The free thermal expansion results (0 % loading) are represented by solid lines based on the expressions presented in Section 4.2.

In order to demonstrate that thermo-mechanical strains only occur during heating, Fig. 8 also contains recordings from the cooling period. It is seen that the contraction that takes place during cooling is practically independent of the compressive stress level.

In all cases the heating is ended before the specimen fails in order to avoid damage to the quartz bars. It is seen that the compressive stress avoids the free thermal expansion to take place. The physical mechanisms behind this phenomenon are not fully understood. However, the effect is seated in the cement paste and therefore it is expected that the stress state enables the (negative) shrinkage strains to develop and counteracts internal cracking perpendicular to the load axis.

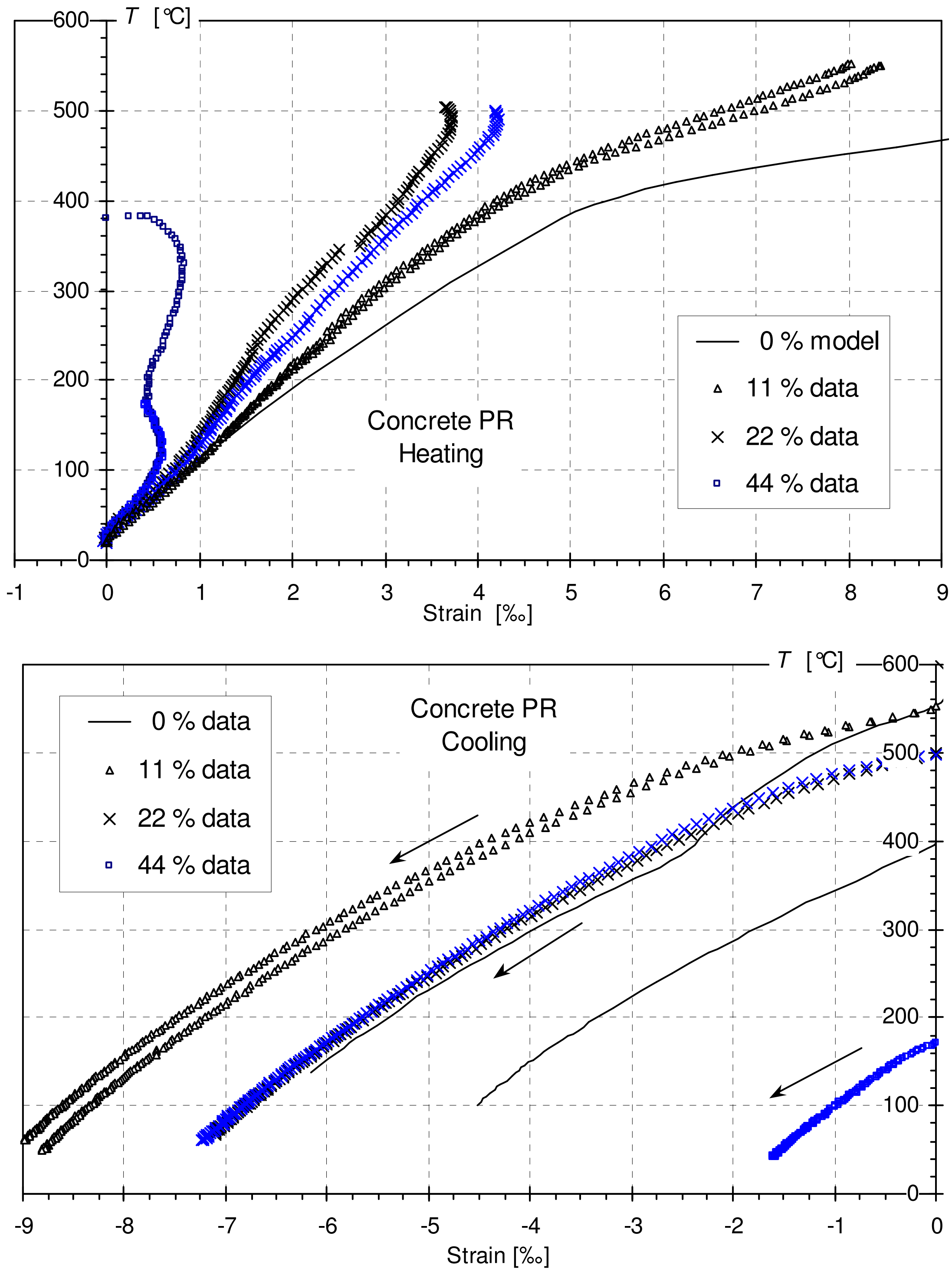
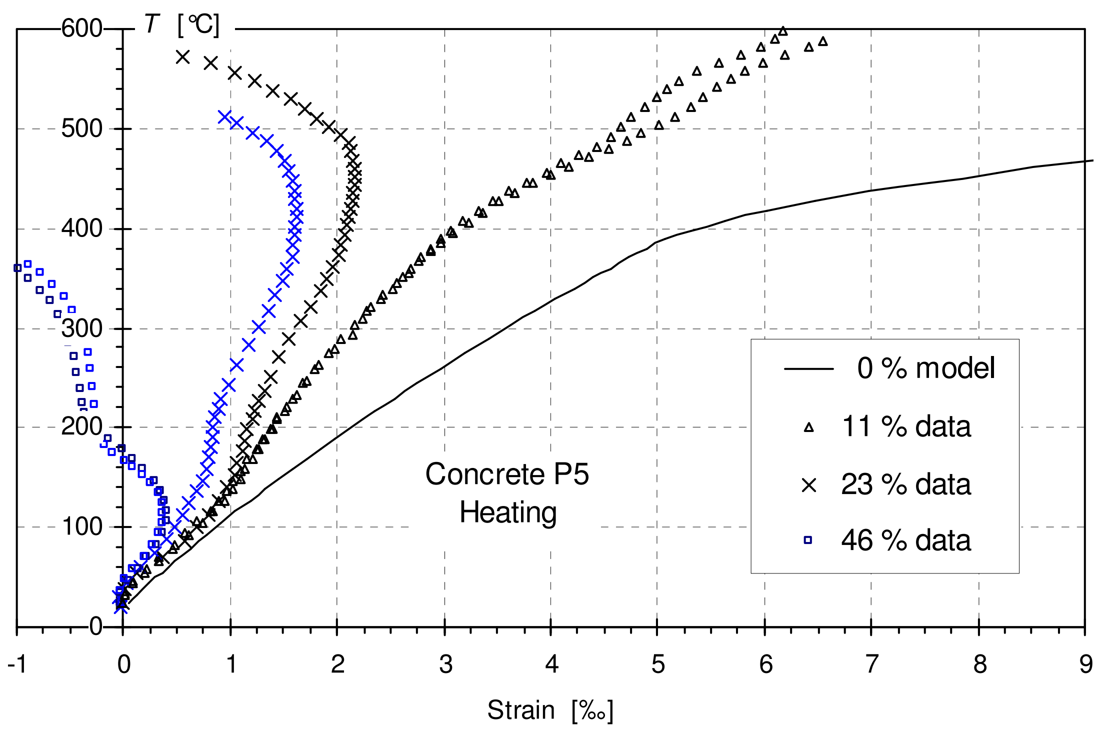


Figure 8 – Total strain data recorded during heating and cooling on PR. Positive strain is elongation. The legend numbers denote constant compressive stress in per cent of f_c^0.



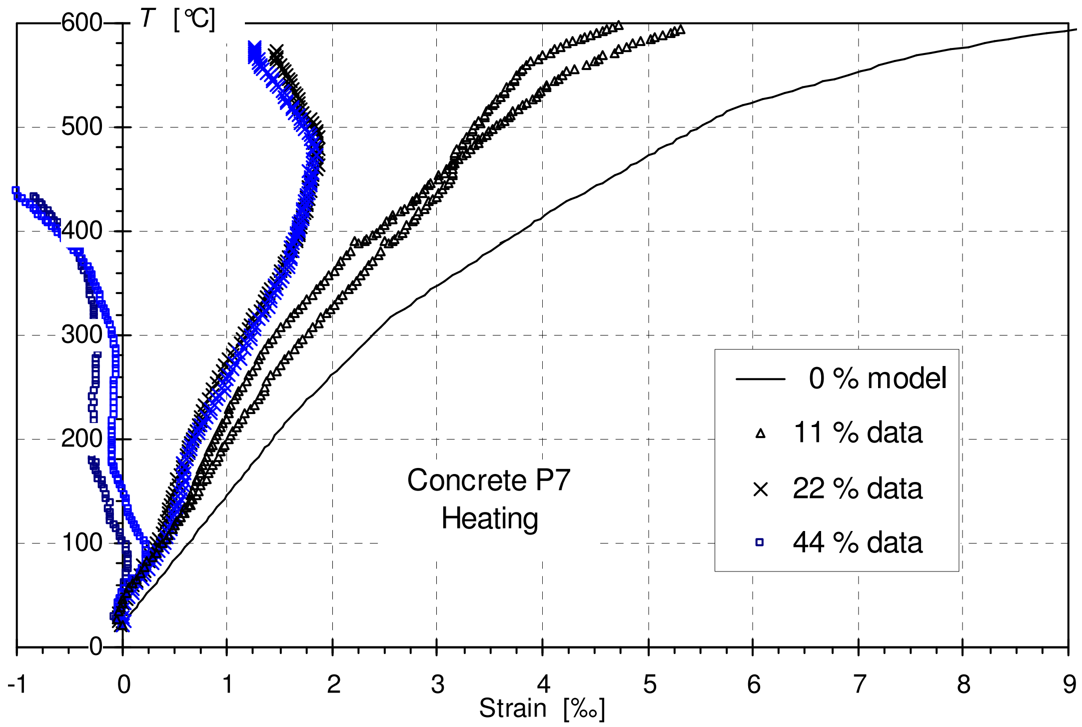


Figure 9 – Total strain data recorded during heating for P5 and P7. Positive strain is elongation. The legend numbers denote constant compressive stress in per cent of f_c^0.

4. THEORETICAL MODELING

In order to perform numerical calculations on the behaviour of concrete subject to varying high temperatures and stresses the constitutive relationship needs to be established. In the following this is done in the uniaxial case based on the test results described in the previous section.

The following formulation of transient strains under varying heating and loading is adopted:

$$\varepsilon_{tot}(T,\sigma) = \varepsilon_{el}(T,\sigma) + \varepsilon_{th}(T) + \varepsilon_{th}^{\sigma}(T,\sigma) \tag{1}$$

where the three components on the right-hand-side correspond to mechanical elastic strain, free thermal strain and combined thermo-mechanical strain, respectively. Temperature is T and stress is σ. Each of the three strain components is treated in the following sections. This formulation is in correspondence with the normal approach [2-4]. Note that (1) does not include conventional creep taking place under sustained loading and isothermal conditions. It is generally recognised that during the short-time duration of a high temperature test the conventional creep strain is negligible compared with the other strain components [3].

In the article stress is only taken as externally applied load. However, the presence of temperature gradients (Fig. 3) produces internal eigen-stresses, which are not considered here.

4.1 Model of mechanical strains

The mechanical strains are formulated by means of a temperature dependent elastic modulus

$$\varepsilon_{el}(T,\sigma) = \frac{\sigma}{E_c(T)} = \frac{\sigma}{E_c^0 \xi_E(T)} \tag{2}$$

where E_c = elastic modulus of concrete and $E_c^{\,0}$ = reference value at room temperature. This approach does not include plastic strains in order to keep the model simple. However, it is realised that in order to model high temperature response realistically the plastic strains need to be included. Several models are already suggested to include non-linear stress-strain behaviour at high temperatures [2-4].

It is proposed to apply a separate reduction factor on the E-modulus instead of connecting the reduction factors for strength and stiffness as it is done in DS 411 [5].

The test results from Fig. 4 are used to model the reduction factor on the E-modulus. The following parabolic expression is suggested to fit the test results:

$$\xi_E(T) = \left(1 - \frac{\Delta T}{T_0}\right)^2, \qquad \xi_E = \frac{E_c(T)}{E_c^0}, \qquad 0 \le \Delta T \le T_0 \tag{3}$$

where ΔT = heating temperature above room temperature (i.e. $\Delta T = T - 20$ °C) and T_0 is a model parameter determining the temperature rise where the reduction factor equals zero. In Fig. 4 a line with T_0 = 800 °C is depicted for illustration purposes. A more general model could easily be formulated if there is the need for it. For instance the power 2 could be treated as a model parameter along with T_0.

The model in (3) does not include any influence from compressive stresses during the heating. It is generally acknowledged that stresses during heating increases the stiffness as well as the strength of the concrete due to the fact that the stresses inhibit the development of cracking. In Fig. 10 the temperature influence on the elastic modulus is illustrated for specimens heated under constant compressive stress [4]. It is seen how the parabolic model in (3) is plausible to predict the reduction factor. The effect of stress level is seen to be most significant at high temperatures. Furthermore, a straight line from the starting point to the end point of (3) is seen to provide a reasonable model for this stress effect. However, the amount of test data is insufficient to give a more detailed model for the stiffening effect of sustained loading under heating.

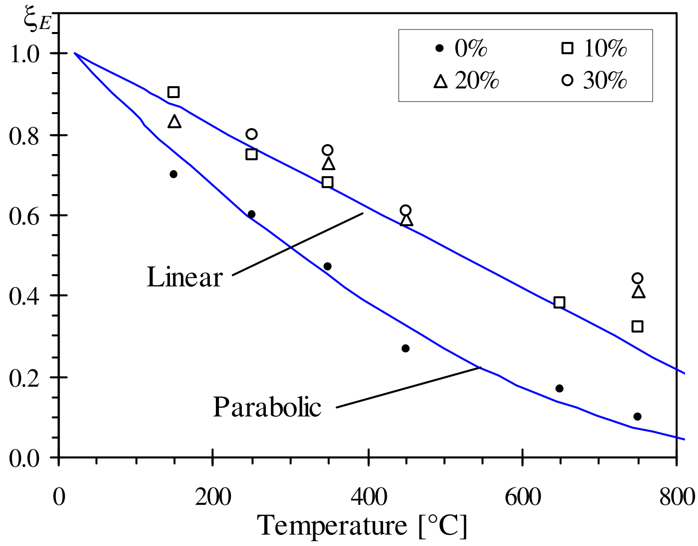


Figure 10 – Reduction factor for E-modulus for normal concrete. Data taken from [4]. The legend numbers denote compressive stress applied during heating in per cent of f_c^0. The parabolic curve is given by (3) with T_0 = 1000 °C and the linear curve connects the end points of the parabola.

4.2 Model of free thermal strains

The free thermal strains depend solely on the temperature:

$$\varepsilon_{th}(T) = \int \alpha(T)\, dT \tag{4}$$

where α = coefficient of thermal expansion and the integration is performed over the temperature range in question. Typically the coefficient of thermal expansion is approximately 10×10^{-6} per °C at room temperature for normal concrete. However, as the temperature increases the coefficient also increases (Fig. 6).

In order to model the abrupt change in behaviour observed during the experiments a bi-linear model of the coefficient of thermal expansion is applied. By integration this results in thermal strains following a parabolic function of temperature:

$$\varepsilon_{th}\big(\theta(T)\big) = \begin{cases} A\theta^2 + B\theta, & 0 \le \theta \le \theta^* \\ C(\theta-\theta^*)^2 + A(2\theta-\theta^*)\theta^* + B\theta, & \theta^* < \theta \end{cases} \tag{5}$$

The three parameters *A*, *B* and *C* together with the transition temperature θ* determine the thermal strain – temperature relationship. The temperature is normalised in order to simplify the notation:

$$\theta = \frac{\Delta T}{100^\circ C} \geq 0 \tag{6}$$

where it is recalled that ΔT is the temperature rise above room temperature.

In Fig. 5 the parameters of Table 2 are applied in (5). It is noted that the analytical expression should only be used within the temperature range where it is experimentally verified.

Table 2 – Parameters for free thermal strain function in (5). Transition temperature = 380 °C.

Parameters	*A*	*B*	*C*	θ*
Concrete	[‰]	[‰]	[‰]	[-]
PR and P5	0.1	1.0	3.5	3.6
P7	0.06	0.7	0.7	3.6

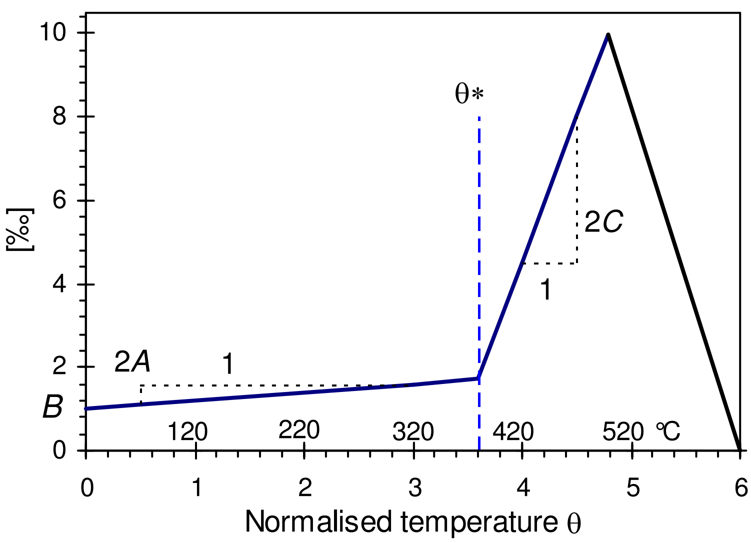


Figure 11 – Illustration of derivative $d\varepsilon_{th}/d\theta$ vs. normalised temperature. Based on parameters for PR and P5.

In Fig. 11 the influence of the parameters in (5) is illustrated. *B* governs the initial value of the coefficient of expansion at room temperature while *A* and *C* govern the slopes of the bi-linear curve. Furthermore, an upper limit corresponding to $\alpha=100\times10^{-6}$ per °C is applied in accordance with the experimental observations in Fig. 6, followed by a decreasing coefficient of thermal expansion. Note that the coefficient of thermal expansion is not determined experimentally beyond 600 °C. However tests shows that the coefficient of expansion reduces to zero after 600 – 700 °C [1-4,6].

4.3 Model of thermo-mechanical strains

The thermo-mechanical strain is assumed to be proportional with the stress level:

$$\varepsilon_{th}^{\sigma}(T,\sigma)=\frac{\sigma}{f_c^0}\tilde{\varepsilon}(T) \tag{7}$$

where it should be noted that $\tilde{\varepsilon}$ is zero during cooling and under isothermal conditions. Thus, the thermo-mechanical strains that develop during heating are considered irreversible. By inserting (2) and (7) in (1) the load dependent strains are formulated as

$$\varepsilon_{el}+\varepsilon_{th}^{\sigma}=\frac{\sigma}{f_c^0}\left(\tilde{\varepsilon}+\frac{f_c^0}{\xi_E E_c^0}\right)=\frac{\sigma}{f_c^0}\varepsilon^{\sigma}=\varepsilon_{tot}-\varepsilon_{th} \tag{8}$$

where $\tilde{\varepsilon}$ = normalised thermo-mechanical strain and ε^{σ} = normalised load dependent strain.

Since the total strain measurements obtained from a type B test (Figs. 8 and 9) are obtained under constant stress conditions they may be used to quantify ε^{σ} by means of (8). In Fig. 12 the test data are converted into ε^{σ} values with ε_{th} given analytically in (5).

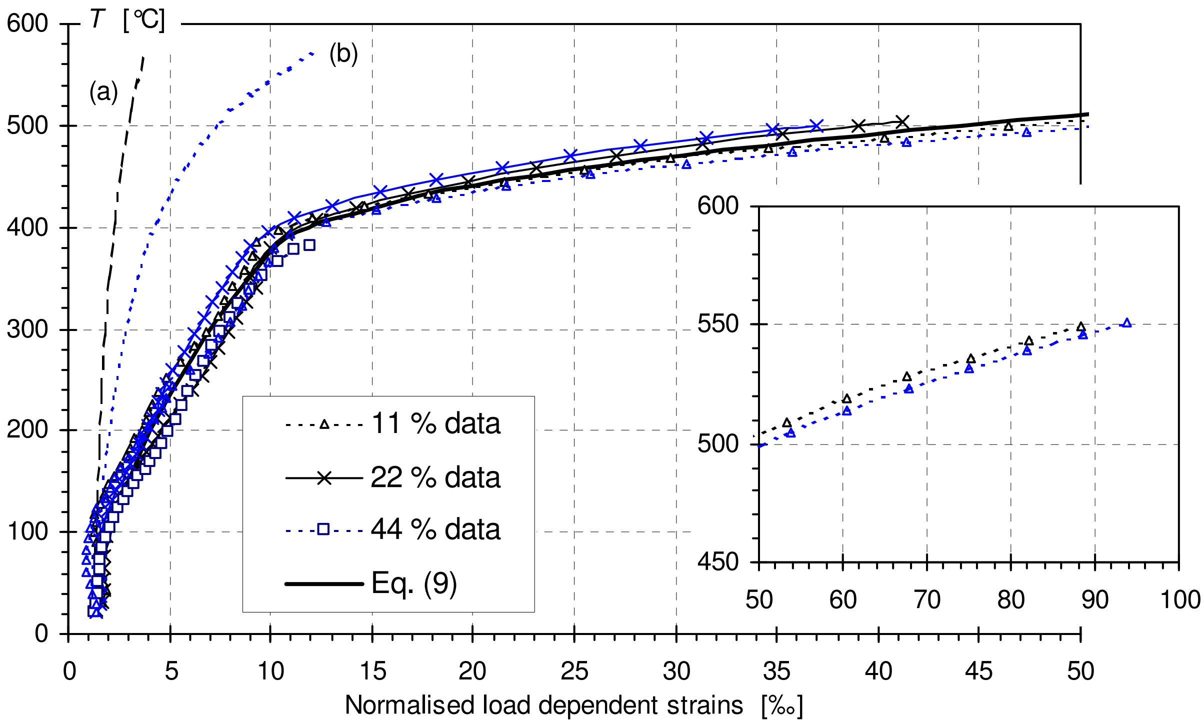
The broken lines in Fig. 12 depict the mechanical part of the strains. Line a is based on a linearly decreasing E-modulus (Fig. 10) and line b is based on the parabolic decreasing E-modulus governed by (3). Thereby, line a includes the strengthening effect of sustained stress during heating while line b neglects this effect. The importance of including the correct temperature degrading effect on the E-modulus is clearly seen especially when the temperature exceeds 400 °C. The physically correct behaviour is probably lying somewhere in between line a and b.

The difference between the data symbols and the dotted line in Fig. 12 is equal to the normalised thermo-mechanical strain $\tilde{\varepsilon}$ in (7). However, instead of performing a subdivision of the load dependent strains into mechanical and thermo-mechanical strains they are kept together in the following description.

The shape of the curves in Fig. 12 generally follows 3 different slopes:

- Below 100 °C the load dependent strain is almost fully elastic and the thermo-mechanical strains are of minor importance.
- From 100 to 400 °C the thermo-mechanical strains start to increase significantly, which is associated with the escape of free and chemically bound moisture from the cement paste.
- Above 400 °C the thermo-mechanical strains increase even faster until the specimen experiences a compressive failure. This behaviour is connected with the formation of cracks within the cement paste. This behaviour is most clearly observed for PR and P5.

By comparison it is found that the test data in Fig. 12 for P7 is in agreement with the Swedish tests performed by Anderberg & Thelandersson [3,8]. Anderberg & Thelandersson suggested the thermo-mechanical strain to be proportional with the thermal strain, i.e. $\tilde{\varepsilon}=k\varepsilon_{th}$. This model has been used extensively in practical applications because of its simplicity; however, the proportionality factor is found to depend on the concrete and the aggregate in question. Normally $1.5 < k < 2.5$ seems plausible.



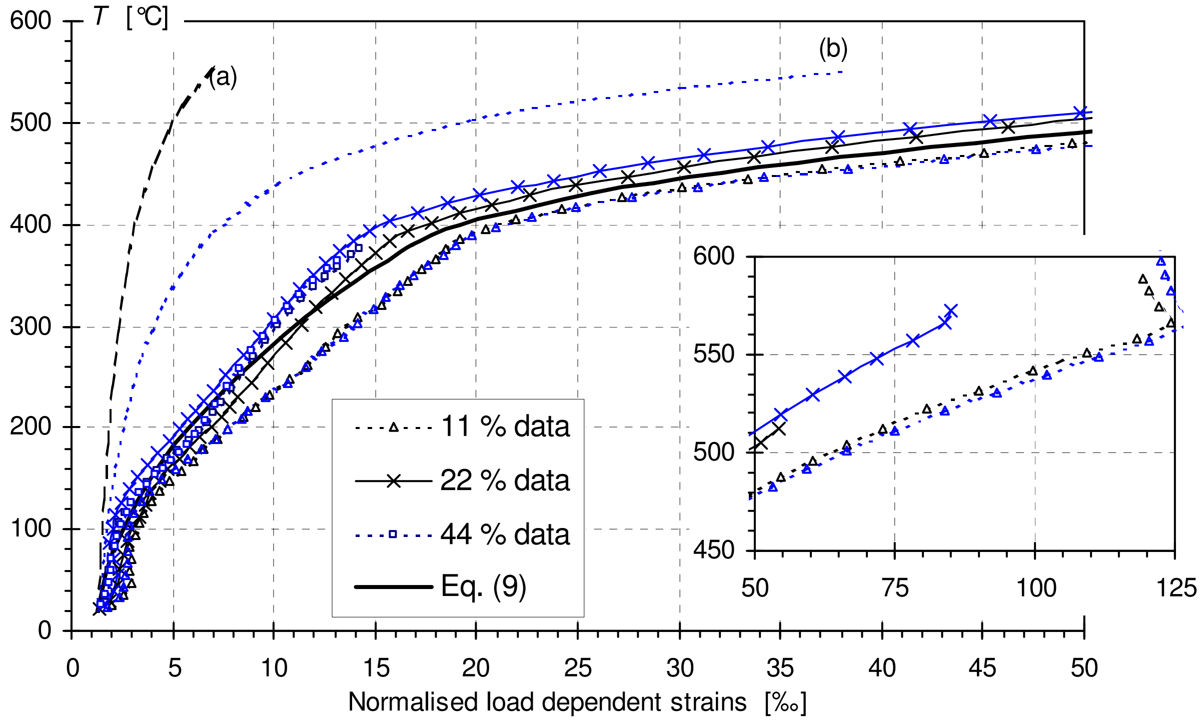


Figure 12 – Normalised load dependent strains ε^{σ} vs. temperature for PR (top) and P5 (bottom) under heating and constant stress (type B tests). The inserted diagrams depict the measurements beyond 50 ‰.

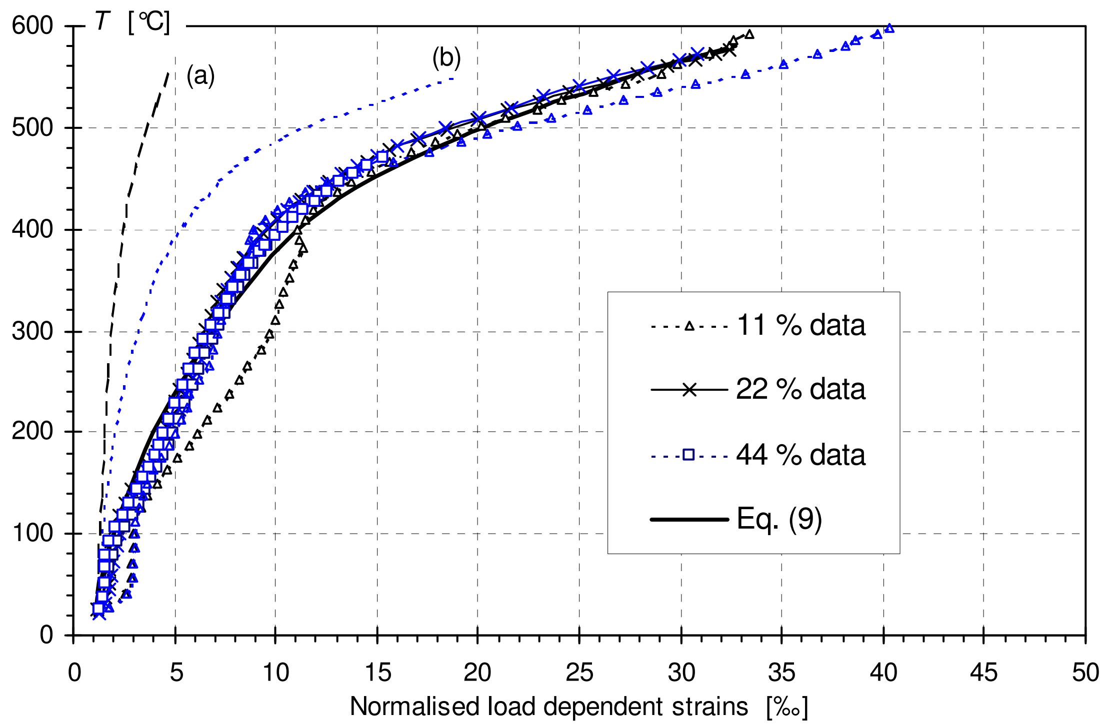


Figure 12 – …continued. Normalised load dependent strains for P7.

In the present article the normalised load dependent strains during heating are modelled by means of an expression similar to (5):

$$\varepsilon^{\sigma} = \tilde{\varepsilon} + \frac{f_c^0}{\xi_E E_c^0} = \frac{f_c^0}{E_c^0} + \begin{cases} A_1\theta^2 + B_1\theta, & 0 \le \theta \le \theta^* \\ C_1(\theta-\theta^*)^2 + A_1(2\theta-\theta^*)\theta^* + B_1\theta, & \theta^* < \theta \end{cases} \qquad (9)$$

where subscript 1 have been added to the three parameters in order to separate them from the thermal strain parameters. In Table 3 the parameters are given corresponding to the lines depicted in Fig. 12. It is seen that P7 generally experiences the smallest thermo-mechanical strains followed by PR and P5. Furthermore, the test results show a rather large scatter especially at high temperatures. Therefore, the suggested model should be used with care and not taken as an exact description of the behaviour.

Table 3 – Parameters for normalized load dependent strains modeled in (9).

Parameters Concrete	A_1 [‰]	B_1 [‰]	C_1 [‰]	θ* [-]
PR	0.5	0.7	20.0	
P5	1.0	0.7	20.0	3.6
P7	0.5	0.7	3.5	

4.4 Verification of model through restrained deformation tests

In order to illustrate the effect of the various strain components under heating, tests of type C have been performed (Fig. 2). A test specimen is heated while its end planes are fully restrained. Thus, compressive stresses build up in the specimen under heating. These stresses may be calculated according to the following equation where $\varepsilon_{tot} = 0$ is applied in (8):

$$\frac{\sigma}{f_c^0} = -\frac{\varepsilon_{th}}{\varepsilon^\sigma} \tag{10}$$

The thermal strains modelled in (5) together with the normalised load dependent strains modelled in (9) are applied with parameters given in Table 2 and 3, respectively.

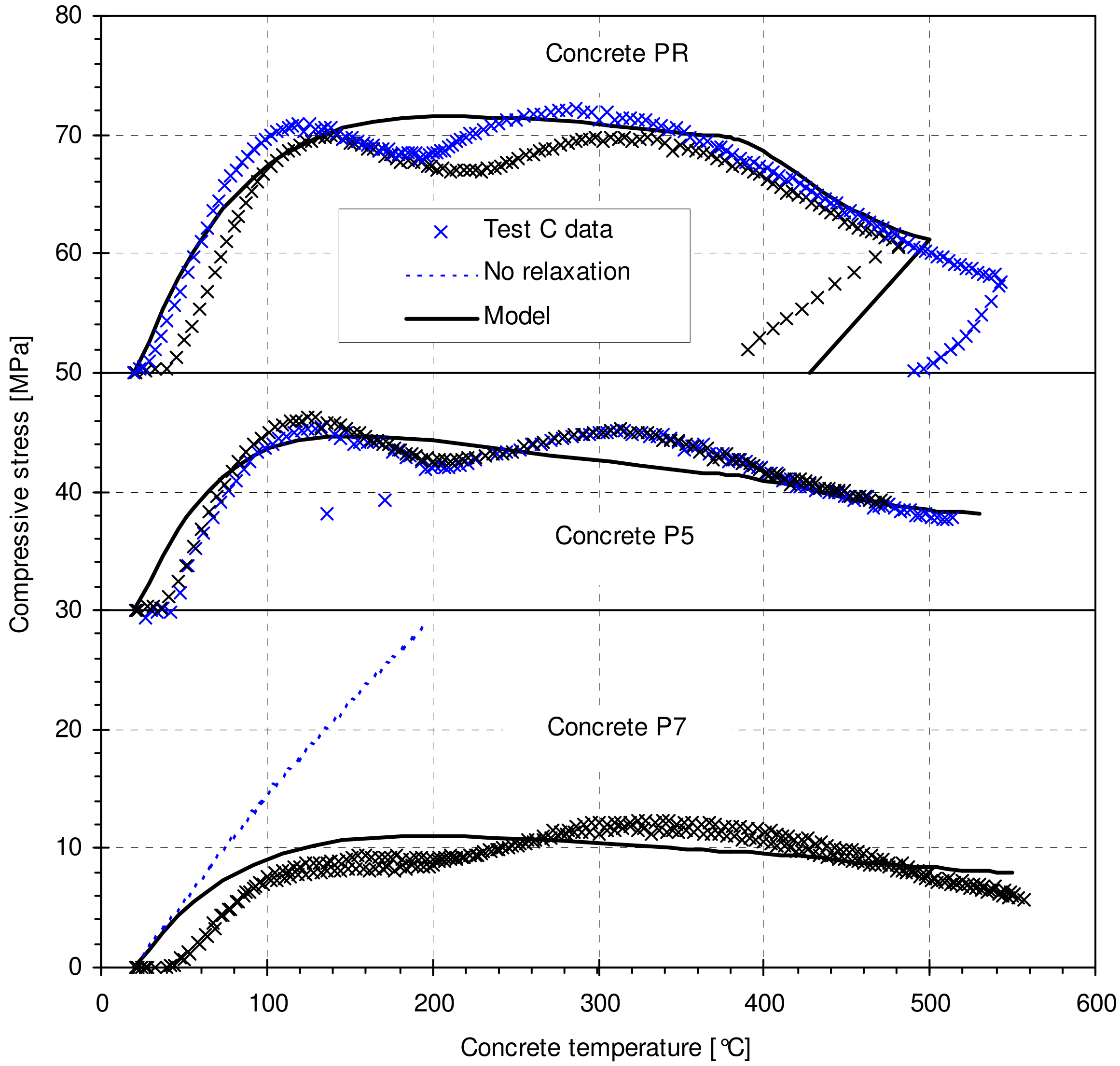


Figure 13 – Restraining stresses during type C test. Note that PR and P5 are shifted along stress axis in order to separate the curves. For PR the stresses are also registered during cooling. Two tests are performed for each concrete.

In Fig. 13 the measured restraining stresses are depicted together with model calculations based on (10). The model calculations are found to be in good agreement with the measurements, finding the correct order of magnitude for the stresses. However, the model cannot predict the local stress peaks that are due to small variations in the coefficient of thermal expansion and in the thermo-mechanical strain. These variations that are mainly caused by vapour transport are not included in the proposed analytical model.

The dotted line in Fig. 13 illustrates a situation neglecting the thermo-mechanical strains, i.e. $\tilde{\varepsilon} = 0$ in (10), i.e. the specimen feels no stress relaxation. It is clearly seen that neglecting the thermo-mechanical strains leads to erroneous results.

During cooling the specimen contracts without any recovery of the thermo mechanical strains and after almost 100 °C temperature drop the specimen is stress-free. In Fig. 13 the stress loss during cooling is calculated for PR based on the hot E-modulus at 500 °C (Fig. 4) and a constant coefficient of thermal strain equal to 15×10^{-6} per °C (Fig. 5). The correspondence with the test data is seen to be satisfactory.

If the specimen is restrained in tension as well as in compression the cooling process produces tensile stresses. This situation is often the case when concrete members are subject to fire and at the same time restrained against deformations due to adjoining structural members. See [9] for an example of this build up of tensile stresses during cooling.

5. CONCLUSIONS

The high temperature tests have resulted in the following main conclusions on the tested types of concrete:

1. The residual strengths are found to be in reasonable agreement with the normal behaviour reported in the literature and in the fire design codes.
2. The E-modulus determined at elevated temperatures is found to decrease almost identically as a function of temperature for the tested concretes. A new analytical proposal for the stiffness reduction factor as a function of temperature is given.
3. The stiffness reduction factor applied in the Danish concrete code DS 411 is found to overestimate the E-modulus significantly compared with the test results. It is recommended that the reduction factors for strength and stiffness should be separated.
4. For the Danish concretes the free thermal expansion under heating and cooling is found to be much higher than for the Swedish concrete.
5. This divergence between the Danish and the Swedish concretes may have several reasons: (i) it is possible that the behaviour is connected solely with differences in aggregate thermal behaviour; (ii) part of the difference may be associated with the Danish cement paste drying out and reversing its deformations from shrinkage to expansion when temperatures exceed 400 °C, or (iii) the difference is associated with internal damage/micro cracking.
6. In order to fully understand the origin of the above-mentioned differences in thermal deformations a more detailed test programme is needed, including separate investigations of aggregate, cement paste, microscopic testing, etc.
7. Tests with simultaneous loading and heating are used to quantify the magnitude of the thermo-mechanical strains (transient creep). It is experimentally confirmed that thermo-mechanical strains are irreversible during cooling.

8. The thermo-mechanical strains of the Swedish concrete are found to be in agreement with other test results reported in the literature. The Danish concretes show larger thermo-mechanical strains, which is partly attributed to the fact that the sustained load inhibits the internal crack formation during heating.
9. New analytical expressions are proposed for the free thermal expansion and the thermo-mechanical strain as a function of temperature. The validity of the proposed analytical models is found to be satisfactory by comparing with restrained tests under heating.

ACKNOWLEDGEMENTS

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Summary of Nordic Mini-Seminar on Concrete and Fire

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ABSTRACT

The paper summarises the presentations and discussions during the two-days seminar. A total of 16 participants from Denmark, Norway and Sweden plus a guest from Northern Ireland attended the event at Danish Technological Institute. The main topics of the presentations were explosive spalling, material modelling under high temperatures, structural modelling and experimental aspects of fire testing. Especially the aspects of explosive spalling attracted discussions on how to standardise testing and conditioning of specimens.

Key words: fire, high temperatures, thermal strains, explosive spalling, vapour pressure, mechanical and thermal properties.

1. INTRODUCTION

Following the previous Nordic Mini-Seminar on Fire Resistance of Concrete [1] organised by SINTEF in 1989 a follow-up was planned and carried out at the Danish Technological Institute on 22/23 May 2003. A total of 16 participants attended the Mini-Seminar sub-divided into:

- 9 participants from Denmark (of which 2 were presenting final year student projects). Representing the Danish Technological Institute (DTI), the Danish Institute of Fire Technology (DIFT), the Technical University of Denmark (BYG-DTU) and Aalborg University (AaU).
- 4 participants from Sweden. Representing Lund Institute of Technology (LTH) and the Swedish National Testing and Research Institute (SP).
- 2 participants from Norway. Representing SINTEF and a private branch organisation for concrete manufacturers (BMB).
- 1 guest speaker invited from FireSERT, University of Ulster, Northern Ireland.

The Mini-Seminar consisted of presentations from the various participants. These presentations were kept in an informal atmosphere in order to accommodate for discussions and questions from the audience. Thus, the initial time schedule of the sessions was overruled early on during the event.

The presenters had prepared short papers on their subjects. These papers are collected in a report from the Danish Technological Institute [2], which may be acquired via the following web site www.danishtechnology.dk/building. The reference list contains the title of each paper [3-15].

The number of participants was equal to that of 1989 in Trondheim but on that occasion the Norwegian delegation numbered 10 persons. Unfortunately no Finnish or Icelandic participants were present.

The main scope of the Mini-Seminar is to enable researchers on the subject to gather and present their results and opinions for an audience of experts. Furthermore, the Mini-Seminar serves as a perfect opportunity to make contacts across the Nordic borders and to improve the collaboration between research institutions. It is my hope that this event will lead to new research projects on concrete and fire.

Dr. Nadjai from FireSERT gave an opening lecture on behalf of Prof. Shields who founded FireSERT through the 1980ies [3]. The reason for inviting Dr. Nadjai is that the Danish Technological Institute is collaborating with FireSERT on an EU-application for funding under FP6. FireSERT is an abbreviation of Fire Safety Engineering Research and Technology, providing courses in fire safety engineering and carrying out fire related research. In 2002 new lab facilities were opened, based on a £5.7 M grant. More details are found in [3].

Dr. Nadjai outlined the facilities and the research groups at FireSERT. It is clear that FireSERT is a research institution successful in attracting research funding and also successful in providing courses to consulting engineers, fire fighters, etc.

In the following some of the topics discussed at the Mini-Seminar are summarised. For more details the reader is referred to the individual papers which gives further literature references.

2. EXPLOSIVE SPALLING

The topic of explosive spalling of concrete covers under rapid heating is still giving cause for discussions and new experimental evidence was presented at the Mini-Seminar [4,5,8,13,14]. It is generally accepted that spalling is governed by moisture content (curing conditions), heating rate, concrete permeability and mechanical stresses. Furthermore, the beneficial effect of pp-fibres is generally acknowledged even though the needed dosage of fibres is depending on several factors. It is also unclear whether the fibre dosage has a local optimum or whether its spalling performance increases with the amount of pp-fibres.

At the Technical University of Denmark (BYG-DTU) a new test set-up is under development to screen various concretes to assess their risk of explosive spalling. The test exposes a standard cylinder to compressive ring stresses together with rapid heating of the cylinder end. Results from this type of test [5,13] show that the set-up still needs to be calibrated in order to demonstrate its repeatability and consistency. Furthermore, the discussion revealed whether a plausible spalling test needs to be conducted on structural elements or on small specimens. SP showed interest in the test method for reference purposes.

A Swedish research project to assess the risk of spalling of Swedish SCC, partly funded by a concrete element manufacturer, was presented by Dr. Persson and Dr. Boström [4,14]. It is experimentally demonstrated that conventional concrete exhibits less explosive spalling than SCC (under identical water-cement ratio). The amount of spalling material by weight differs by a factor 4-5.

One objective of the project is to quantify the amount of pp-fibres necessary to add to the SCC-mix so that the spalling performance is similar to that types of conventional concrete. Tests are reported on 16 different concretes, varying the pp-fibre content, w/c-ratio, cement and filler type. It is concluded in [4] that the spalling is very sensitive to the amount of lime filler.

Finally, it was stated that one of the outcomes of the Swedish project on SCC was that the pre-fab manufacturer had stopped using SCC in fire exposed structural elements.

Dr. Hammer from SINTEF reported the Norwegian experiences with HPC with lightweight aggregate subject to hydrocarbon fire [8]. The research dates back to the 1980ies, however, it seems that the same subjects are still being discussed some 15 years later.

3. MATERIAL PROPERTIES

Concrete properties such as thermal conductivity, thermal expansion, transient creep and mechanical strength and stiffness are treated in [4,9,11,12].

The presentations from SP focused on the thermal properties needed to model the temperature development within a solid concrete cross-section. A new technique to obtain the thermal conductivity, the heat capacity and the thermal diffusivity was presented [11] under the name Transient Plane Source. It was demonstrated how test results using this technique could predict the temperature dependency of these thermal properties. Furthermore, Dr. Wickström from SP presented numerical temperature calculations emphasising the importance of including the energy consumption from water evaporation giving a time delay in the temperature increase (this presentation was not accompanied by a paper).

Investigations on the decreasing strength and stiffness were presented in [4,9] for SCC and pumice concrete respectively. The results on strength degradation were consistent with earlier experimental investigations and the structural concrete codes. However, [9] stated that the Danish code of practice (DS 411:1999) seems to overpredict the E-modulus as a function of temperature.

New test results on thermal deformations of concrete were presented in [12]. These tests are to the author's knowledge the first of its kind performed in Denmark. The thermal expansion of Danish concrete based on uncrushed sea material and quartz sand, showed significantly higher expansion than similar Swedish concrete that was based on crushed siliceous aggregate. The difference at 600 °C was reported to be 100 %. This difference was attributed to the aggregate type but it could also be connected with the use of flyash and microsillica in Danish concrete.

4. STRUCTURAL MODELLING

Test results and calculation models were presented regarding various structural elements such as walls, slabs, etc. [6,7,15].

In [6,7] full scale tests performed at the Danish Institute of Fire Technology were presented the former on normal concrete and the latter on lightweight concrete based on expanded clay aggregate.

A Danish test programme performed in the 1990ies [6] on prefab concrete elements demonstrated how difficult it is to predict the buckling behaviour of bearing walls and columns. Due to the influence of thermal deformations the deflection of the wall is mainly towards the fire exposure (bowing). However, depending on the load eccentricity the deflection history may change which Dr. Hertz demonstrated [7] by means of experimental measurements.

Based on the tests on lightweight concrete a design tool has been developed at the Technical University of Denmark for the design of carrying capacity of walls and slabs under fire load [7]. The manufacturers of expanded clay concrete financially supported this investigation.

In his closing lecture [15] Dr. Nadjai presented both tests and calculations on concrete block masonry walls loaded axially.

5. RESEARCH NEEDS AND POSSIBILITIES

As a direct representative for the Norwegian concrete and masonry manufacturers Dr. Vik presented BMB as an umbrella organisation to promote the use of concrete and masonry in Norway [10]. BMB was created more than 20 years ago to mitigate the competition from the Norwegian timber industry. They have been involved in various research projects comparing the fire safety of concrete and masonry walls compared with lightweight timber walls and active fire protection (sprinkling).

It was agreed under the following discussion that in order to attract funding for fire research it is necessary to include the end-users of the research (the concrete industry, the consulting engineers, the building owners, etc.). Therefore, BMB is an important partner as a representative of the industry.

Several times during the Mini-Seminar it was argued whether the introduction of Eurocode 1992-1-2 on fire design of concrete is a step in the right direction in order to create consensus on the topic. The author stated that the structural fire design is complicated by the fact that Eurocode includes partial coefficients in its tabulated values. Thus, the fire design loading becomes illogical and the system looses its transparency because each country introduces its partial coefficients through the national application documents. N. Andersen from the Danish Institute of Fire Technology stated the same thing in his presentation [6] and Dr. Hertz pointed out that the Eurocode seems to lack consistency with general calculation methods on the carrying capacity under fire load.

A summary of the research needs may be formulated as:

- Explosive spalling. Development of proper test methods with sufficient level of simplicity and at the same time being structurally plausible.
- Better calculation models for predicting carrying capacity of walls and columns including effect of bowing/deflection, support conditions etc.
- Better knowledge of modern concrete types thermal expansion and creep behaviour. To be used as input to structural calculations. Especially the effects of adding flyash, microsilica and chemical admixtures need to be clarified.

There seems to be a broad agreement that more research should be initiated on concrete and fire for instance through funding from the Nordic Industrial Fund.

By the end of the Mini-Seminar it was decided that SP in Sweden should arrange the next event in 2-3 years time.

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Modelling a Cement Manufacturing Process to Study Possible Impacts of Alternative Fuels – Part A

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Abstract

Energy costs and environmental standards have encouraged cement manufacturers world-wide to evaluate to what extent conventional fuels can be replaced by different alternative fuels, *i.e.* processed waste materials. Also Finnsementti decided to take an important step towards using alternative fuels. In order to make a proper choice from a wide range of alternative fuels it was decided that a commercial modelling tool should be used to model the four-stage pre-heater plus kiln system for a full-scale cement plant (daily clinker production ~ 2150 tons) that uses, for example, scrap tires as secondary fuel.

Modelling work was started with Aspen Plus program meaning that the process equipment were described as inter-connected blocks simulating the set-up at the plant. Also steady-state mass and energy balances and thermodynamic equilibrium for the chemical species were calculated. By comparing the calculated results with existing process values, the model is used to optimise process control and the use of alternative fuels while maintaining clinker product quality. It is also used to predict the possible changes in the combustion, pre-calcining and clinker formation processes. The waste tyre fuel for example also acts as a raw material in the clinker manufacturing process. Calculations with different descriptions of the clinker chemistry were made and evaluated against real process data where possible. Some results are given in this paper.

Key words: Modelling, Cement production, Alternative fuels

1. INTRODUCTION

Alternative fuels offer cement industry the opportunity to reduce fossil-fuel derived CO_2 emissions, reduce fuel costs and contribute to the processing of waste streams at a time of increased pressure on land filling of wastes. The question of what could happen to a cement clinker production process after a fuel switch may partly be answered by computer simulation, using a model or combination of models that take into account process chemistry, fluid dynamics and thermodynamics.

Most of the recent studies on modelling cement manufacturing reported in the literature are based on computational fluid dynamics (CFD) and they mainly study aerodynamic behaviour of particles in the preheating system, the shape and temperature of the flame in the combustion zone, coal combustion itself as well as oxygen enrichment in the burning zone and not so much the thermodynamics and clinker chemistry taking place in the kiln system [1,2,3]. This paper describes the simulation of a cement clinker production plant using the commercial software Aspen Plus®, with a strong focus on clinker chemistry and thermodynamics in the rotary kiln and the effect of alternative fuels on material flows, emissions and product quality.

2 CEMENT MANUFACTURING AND KILN SYSTEM CHEMISTRY

Cement manufacturing consists of raw meal grinding, blending, pre-calcining, clinker burning and cement grinding. In short, limestone and other materials containing calcium, silicon, aluminium and iron oxides are crushed and milled into a raw meal. This raw meal is blended and then heated in the pre-heating system (cyclones) to start the dissociation of calcium carbonate to oxide. The meal goes further into the kiln for heating and reaction between calcium oxide and other elements to form calcium silicates and aluminates at a temperature up to 1450 ^oC: the so-called clinker burning. The cyclone system is attached to the rotary kiln by a riser duct. Secondary fuel is fed to the riser duct, whiles the main fuel mixture, coal/petcoke, is used to fire the kiln. Reaction products leave the kiln as a nodular material called clinker. The clinker will be interground with gypsum and other materials to cement [4]. Figure 1 shows a simplified flowsheet presenting the cement manufacturing process.

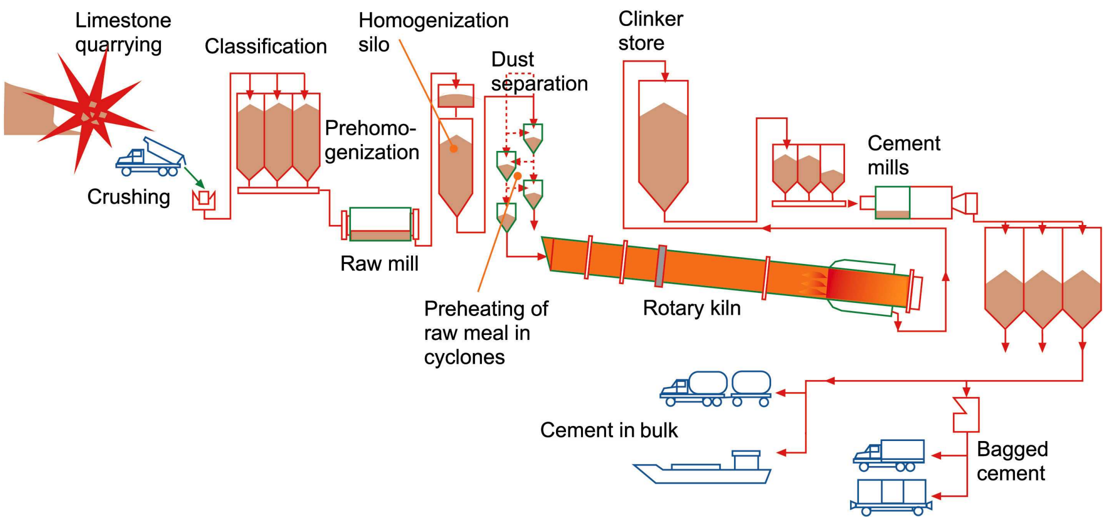

Figure 1. Cement manufacturing from the quarrying of limestone to the bagging of cement.

The chemical reactions that occur in the kiln are illustrated in Figure 2 [8]. Reading the picture from the left-hand side the temperature is increased when going from the meal feed to the burning zone in the rotary kiln. The most important oxides that participate in the reactions are $CaCO_3$, SiO_2, Al_2O_3 and Fe_2O_3. Up to about 700°C water is removed from the meal. In the preheating section (700-900°C) calcination as well as an initial combination of alumina, ferric oxide and silica with lime takes place according to Figure 2. However, due to the short residence time in the pre-calciner section (*i.e.* several seconds), this initial chemistry does not occur, but compounds such as spurrite, $2(CaO)_2*SiO_2*CaCO_3$, and sulphate spurrite $2(CaO)_2*SiO_2*CaSO_4$ are formed. From 900°C to 1200°C belite, C_2S (= $2CaO*SiO_2$), forms, partly from spurrite decomposition. Above 1250°C a liquid phase appears and this promotes the reaction between belite and free lime to form alite, C_3S (= $3CaO*SiO_2$). During the cooling stage the molten phase forms C_3A, tri calcium aluminate (= $3CaO*Al_2O_3$) and if the cooling is slow alite may dissolve back into the liquid phase and appear as secondary belite [8].

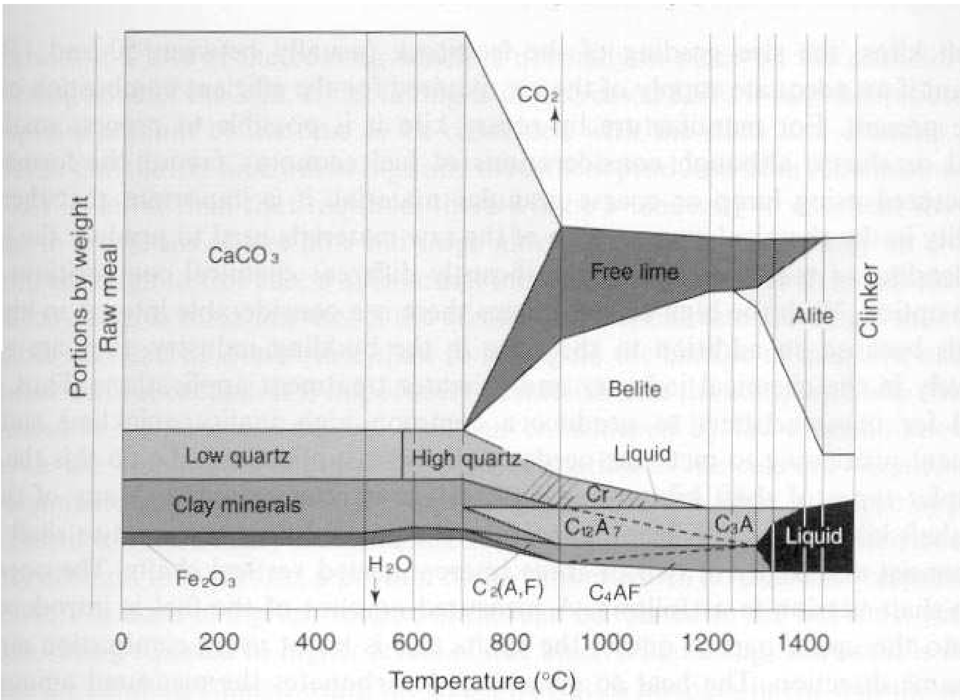

Figure 2. A schematic view of the clinker formation reactions [8].

The main reactions in clinker formation are as follows:

1. $CaCO_3 \rightarrow CaO + CO_2$
2. $2CaO + SiO_2 \rightarrow C_2S$ ($2CaO*SiO_2$)
3. $3CaO + Al_2O_3 \rightarrow C_3A$ ($3CaO*Al_2O_3$)
4. $4CaO + Al_2O_3 + Fe_2O_3 \rightarrow C_4AF$ ($4CaO*Al_2O_3*Fe_2O_3$)
5. $CaO + C_2S \rightarrow C_3S$ ($3CaO*SiO_2$)

Clearly, it is important that the combustion process and the composition of the meals fed to the kiln system is well understood. Usually the production of clinker is done in such a way that one type of clinker allows the plant operators to manufacture several well-defined types of cement that comply with the physical demands as specified in the cement standards.

3 THE PROCESS MODEL

The process model (set up under Aspen Plus® 10.2) [6] consists basically of mass balances for compounds and reactors. Incoming and outgoing compound flows are studied to get the chemical processes and their changes in the kiln system under better control [5]. At the same time, the calcium silicate system typical for cement manufacture is studied more closely including alkali species, chlorides and heavy metals.

One goal of the modelling work is to find out how well a certain type of reactor or separator can describe a certain function in the process. The final target is to be able to describe the behaviour of the kiln process in a realistic way, when the fuel is fully or partly replaced by an alternative fuel. The outcome could possibly be called "quality control" giving vital information about the chemical changes caused by the alternative fuel, the heat transmission in the kiln, and possibly also other parameters controlling the clinker burning process. It is important to quantify how these affect the clinker composition. Figure 3 shows a flowsheet chart of the process model. The material streams from one reactor or block to another, the in- and out- going flows are shown, as are also different (alternative) fuel feeding locations. The feed of alternative fuels is possible to both the rotary kilns burning zone and to the pre-heating system, in this case meaning the riser duct with the pre-calciner cyclones. Several fuel-blocks are attached to both pre-calciner and burning zone in order to make it easier to simulate different fuel mixes. Chemical analyses are needed both for the kiln feed and the fuels that are intended to be tested with this model.

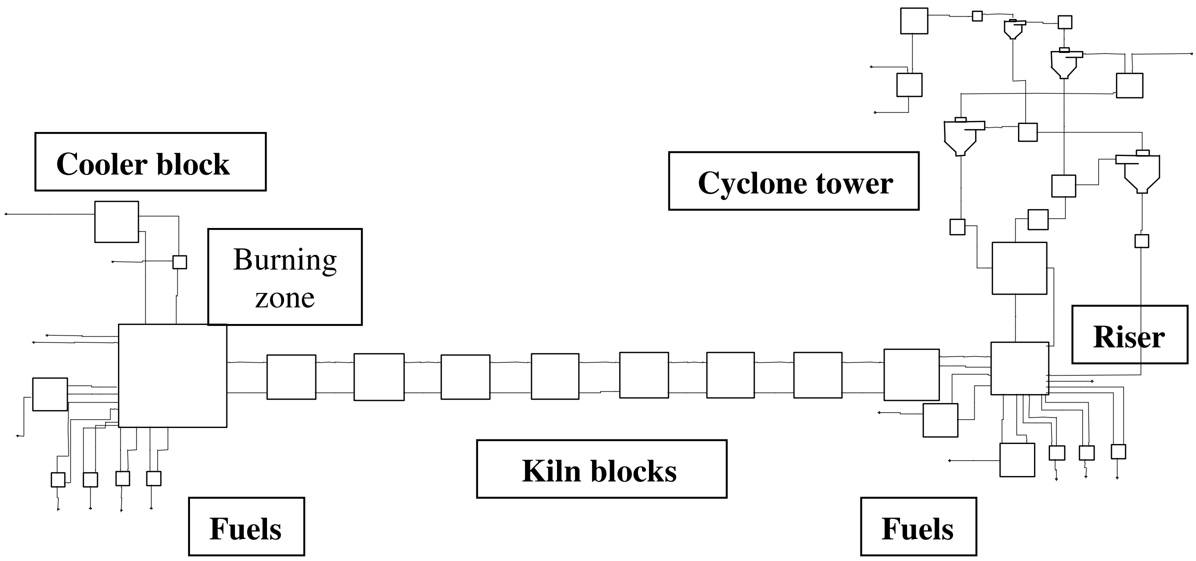

Figure 3. Birds' eye of a process flowsheet model in Aspen Plus® showing the structure of the model and streams to and from the "blocks".

To be able to build a simulation model, detailed knowledge of the process is necessary and a lot of information on the design and operation of the cement plant at Parainen is used in the modelling work [5]:

- Temperatures and pressures at various locations and the incoming mass streams
- The dimensions and operational parameters of the cyclones, defining their grade efficiency performance
- Chemical composition and heating values of the incoming raw meal, the primary and secondary fuel, with particle size distributions for all these materials

- Incoming mass flows of raw meal, primary and secondary fuel and combustion air

Several things could not be implemented straightforward into the model, as an example the calcination of the raw meal taking place in the pre-heating system, cyclones [5]:

- In the heat exchange section, the cyclones act as separators in which also the calcination reaction $CaCO_3(s) \rightarrow CaO(s) + CO_2$ is taking place. This was modelled as a cyclone separator with a perfectly stirred reactor at the inlet in which the calcination reaction can take place, depending on temperature and CO_2 partial pressure, see Figure 4.

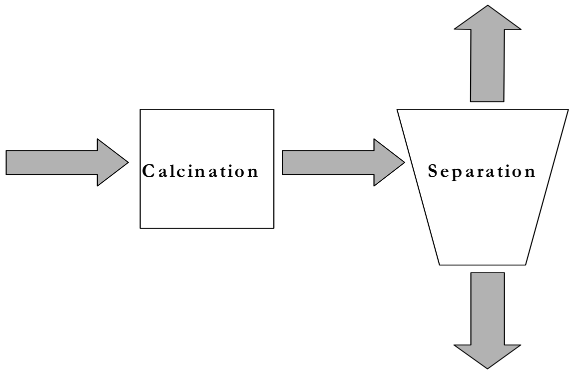

Figure 4. Modelling block for a cyclone in which separation as well as limestone calcination takes place.

The blocks are chosen in such a way that the chemistry in the different parts of the process can be specified as realistic as possible (*e.g.,* equilibrium or non-equilibrium reactors), in a user-friendly way. After having attached the various modelling blocks to each other, representing different pieces of equipment in the clinker manufacturing process, the model is rather much in accordance to the flowsheet of the real process. One of the differences is the modelling block for the pre-calciner cyclones, as mentioned above. Also processes like separation or splitting of streams are modelled with specified blocks, while the flowsheet of the process only shows "pipelines" directing the flows to different pieces of equipment.

All the participating components are specified and listed on the same data sheet. In addition, the particle size distribution must be defined for all streams where solids are involved. Proximate fuel analyses are listed, as well as the elemental analysis data for the fuels. Pieces of the summary sheets with the input data and how the possible results may look are shown in Tables 1 and 2.

Table 1. An example of the Aspen Plus® input data used in the simulation (numbers reduced to one decimal).

	Primary air	Primary coal	Secondary coal	Raw meal
Temperature C	20	70	70	70
Pressure bar	1	1	1	1
Mass VFrac	1	0	0	0
Mass SFrac	0	1	1	1
*** ALL PHASES ***				
Mass Flow kg/hr	59184.0	9500	1200	69970
Volume Flow m^3/hr	50000	6.85	0.87	23.04
Enthalpy MMkcal/hr	-0.07	0.65	0.08	-208.5
Density kg/m^3	1.18	1386.8	1386.8	3037.4

When it comes to the kiln system chemistry Figure 2 does not show the effect of sulphur and alkali on the melt formation in the kiln, neither does it show the substitution reactions that will occur when alkali, sulphur or trace elements are interchanged with, for example, the elements Ca, Si or Al. Free lime is present as well as periclase (MgO) and anhydrite ($CaSO_4$). The formation of these phases will be taken into account in the modelling work at a later stage.

Also when it comes to the chemical components it was necessary to make several changes to be able to specify the reactions related to clinker formation [7,8]:

- Iron containing silicates, which are formed during clinker production (such as C_4AF,= $4CaO*Al_2O_3*Fe_2O_3$) are not included in the standard databases of the modelling software. Therefore a separate databank was created and used in the calculations, which contained C_4AF, NAS_6 (= $Na_2O*Al_2O_3*6SiO_2$) and KAS_6 (= $K_2O*Al_2O_3*6SiO_2$) [9].

There is still work to be done with the created "inhouse" data bank. Other chemical compounds related to clinker formation will be added, such as for spurrite, if thermodynamic data for these can be found. Below, results calculated with the modelling softwares' standard database are compared with results calculated with the additional databank included.

Table 2. Results (Aspen Plus® layout reduced to one decimal) from one of several calculations done to balance up the kiln system (results are not verified).

	Gas, burning zone	Clinker	ESP, dust	ESP, gas	Material stream from cyclones	Dust stream from cyclones
Temperature C	2000	275	160	160	200	950
Pressure bar	0.997	1	0.8	0.8	0.945	0.983
Mass VFrac	1	0	0	1	0.941	0
Mass SFrac	0	1	1	< 0.001	0.059	1
*** ALL PHASES ***						
Mass Flow kg/hr	81077.8	86217.9	8749.9	139479.3	148229.2	37000.7
Volume Flow m^3/hr	521242.8	11.3	4.0	187409.2	173239.4	9.2
Enthalpy MMkcal/hr	-5.1	-255.2	-23.9	-160.0	-182.4	-104.1
Density kg/m^3	0.16	7619.8	2182.1	0.7	0.9	4009.7
Mass Flow kg/hr						
N_2	55631.3			64768.8	64768.8	
O_2	89.4					
H_2O	3922.9			4637.9	4637.9	
NO_2	0.01					
NO	86.7					
S	0.06					
SO_2	243.6			0.005	0.005	
SO_3	0.03					
H_2	24.05			2.5	2.5	
Cl_2	Trace					
HCl	9.7			21.3	21.3	
C			559.1	0.001	559.1	144.3
CO	3512.9			0.9	0.9	
CO_2	17557.2			70047.9	70047.9	
$CaCO_3$			6116.3	0.006	6116.3	
CaO						7586.8
SiO_2			1130.7	0.001	1130.7	
Al_2O_3			249.2	< 0.001	249.2	
Ca_3SiO_5		72571.4				
Ca_2SiO_4						23485.9
$Ca_3Al_2O_6$		10633.7				4785.6
$CaSO_4$			344.6	< 0.001	344.6	
$C1_2A_7$		382.5				
Na_2O						
K_2O						
NaCl				Trace	Trace	
KCl				Trace	Trace	
Na_2SO_4			98.o	< 0.001	98.0	
K_2SO_4			121.4	< 0.001	121.4	52.0
Na						
K						
Fe_2O_3		1657.7	130.6	< 0.001	130.6	946.2
Ash		972.6				

4 RESULTS AND DISCUSSION

Calculations were made to test how well simulation results compare with the clinker quality data reported by the production team. In these calculations both the primary and the secondary fuels are coal. Calculations were done with both the softwares' own thermodynamic databank and in combination with an "inhouse" data bank. In Figure 5 we can see the alite values for different cases. Figure 6 shows the belite content.

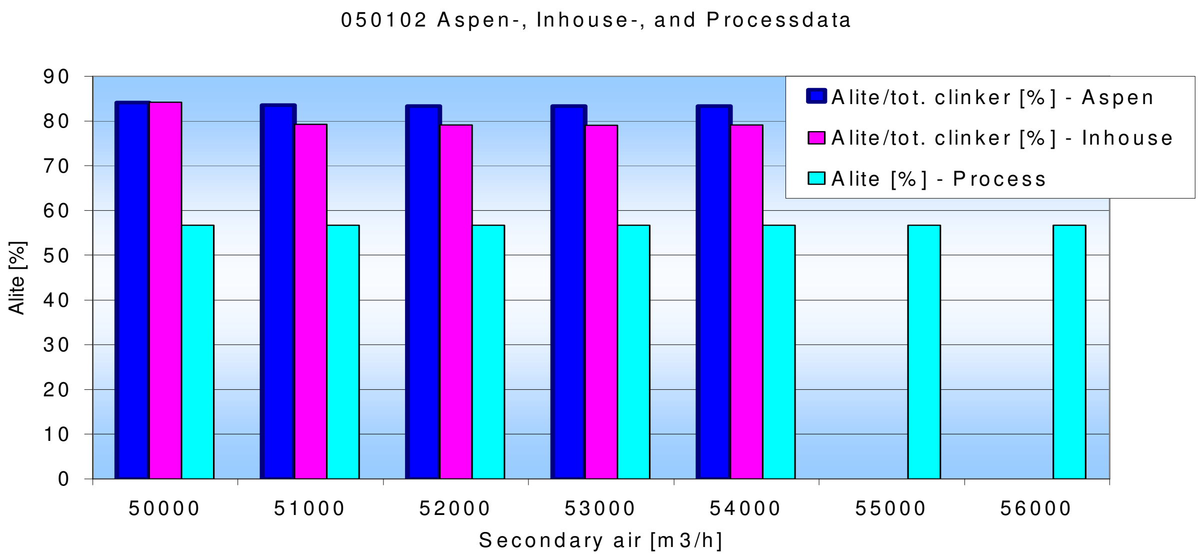

Figure 5. The content of alite in the clinker, process values compared with calculated results. The x- axis shows the amount of combustion air (secondary air) fed to the burning zone.

It has to be noted that variations in the feed of combustion air (secondary air) changes the formation of clinker minerals, as also was expected. As the alite content goes down, the belite content grows rapidly. The balance can be found somewhere between the amounts 54000 and 55000 m^3/h of secondary air.

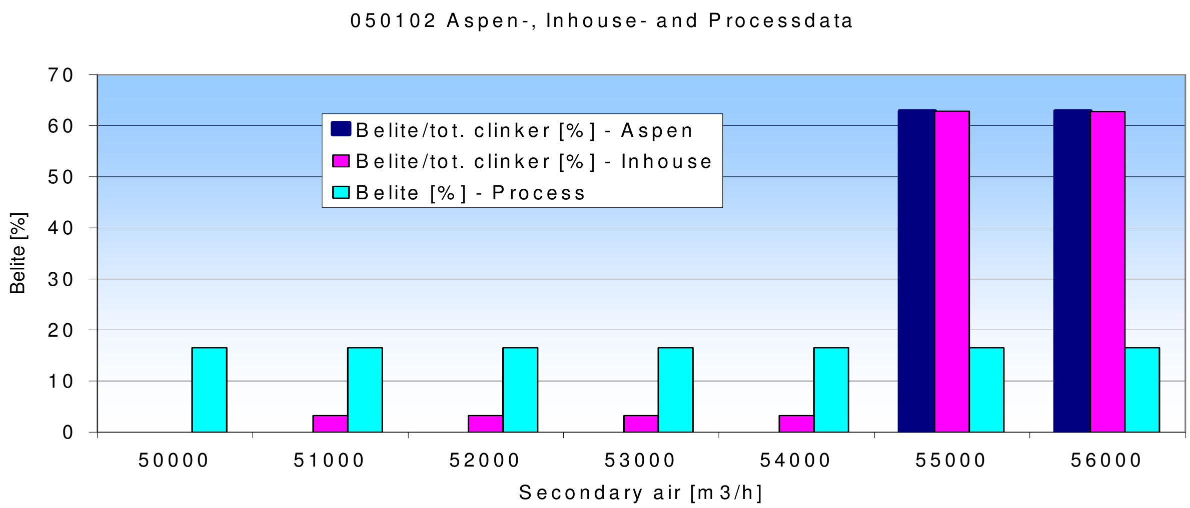

Figure 6. The content of belite in the clinker, process values compared with calculated results.

To be able to figure out what causes the changes in the interval 54000-55000 m³/h secondary air, also a graph for the oxygen content in the flue gases after the burning zone, was plotted. A notable increase in the oxygen amount can bee seen. If too large amounts of secondary air are fed to the burning zone it interrupts the clinker formation. The flame becomes unstable, the burning zone is cooled and a lot of dust is in circulation in the kiln and pre-calciner system. The variations in oxygen excess are shown in Figure 7.

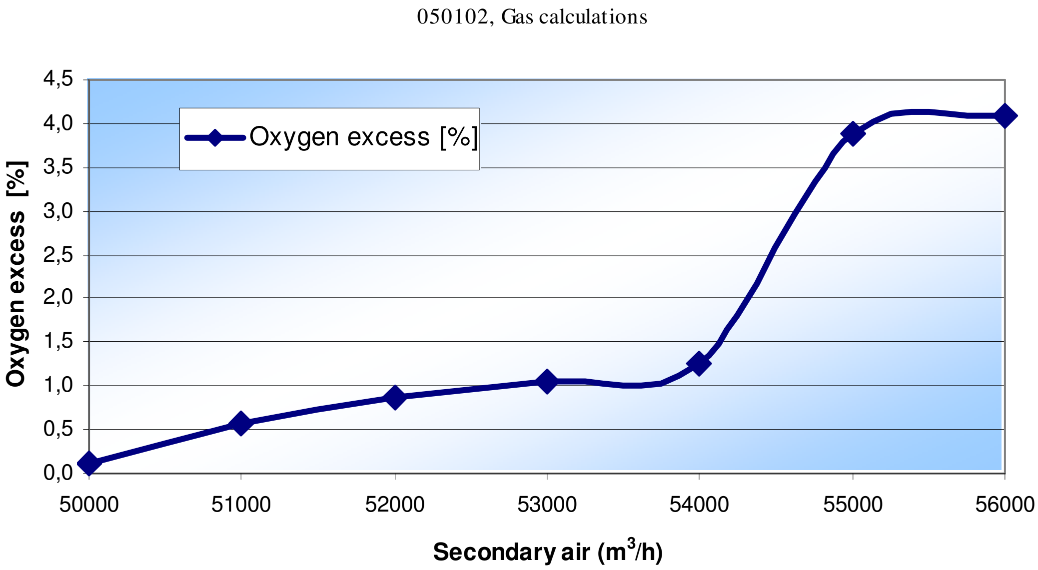

Figure 7. The amount of oxygen excess after burning zone plotted against the amount of secondary air.

The demand of combustion air in the burning zone appears to have a large influence on the results. As mentioned earlier there is a clear connection between the use of alternative fuels and the behaviour of the kiln process. The required air amounts will change significantly when changing from a fossil fuel to an alternative fuel, which may require changes to the process equipment. The model described above is a powerful tool when selecting an alternative fuel for a cement kiln [10].

5 CONCLUSIONS

If there are doubts as to whether one should change fuels in the rotary kiln system of a cement plant it may be of use to simulate the possible cases beforehand and obtain information on how serious the resulting changes might be. This can also be used to check whether the equipment is suitable and flexible enough for the new fuel combinations.

There are clear differences between the calculated values for clinker compositions and the production values. This is due to the changes in the "description" of the clinker chemistry, and because not all the participating elements are listed in the output of the simulations. Although the results are improved by using the “inhouse” data, further adjustment will be needed. It is also important to balance the fuel and air amounts against the kiln feed to optimise clinker quality.

Part B of this paper is scheduled for publication in year 2004. It will present calculations with various alternative fuels.

ACKNOWLEDGEMENTS

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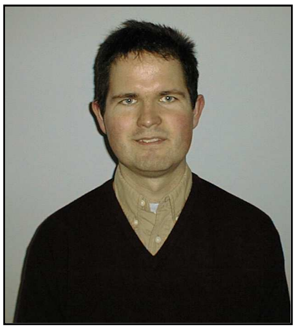
Experimental Investigation of Combined Thermal and Mechanical Behaviour of Danish and Swedish Concrete subject to High Temperatures



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ABSTRACT

A total of 60 high temperature tests have been carried out on three different concretes heated to 600 °C (one Swedish and two Danish concretes). Experimental evidence of thermal strains during heating with or without sustained loading is reported. The registered strains under transient temperature conditions are subdivided into purely mechanical strain, free thermal strain and thermo-mechanical strain. Furthermore, analytical models are proposed to predict the temperature dependency of the individual strain components. One important finding of the tests is that the Swedish concrete show only half of the free thermal strain that is observed for the Danish concretes at 600 °C. This difference is meant to be due to the differences in aggregate type.

Key words: concrete, high temperatures, thermal strain, transient creep, heating.

1. INTRODUCTION

Thermal exposure under fire is one of the most severe actions a concrete structure can be subjected to. The two main constituents of concrete react totally different against first time heating where the aggregate is expanding and the cement paste is shrinking under expulsion of moisture. The incompatible deformations may inflict severe damage to the concrete followed by degrading strength and stiffness. For a thorough description of the involved mechanisms the reader is referred to the literature, see e.g. [1-4].

In order to model the behaviour of a concrete structural member subject to heating, e.g. during a fire scenario, it is necessary to have some knowledge of the material response in the high temperature region of several hundred degrees. This response is complicated for the following reasons: (i) it is a combination of two opposite mechanisms - expansion of aggregate and shrinkage of cement paste – being experimentally difficult to separate; and (ii) the stress-strain response depends significantly on a combination of the stress history and the temperature history. Furthermore, the whole area of material testing under high temperatures suffers from a lack of test standards.

The present investigation is an experimental attempt to quantify the mechanical behaviour of typical Danish and Swedish concrete subject to combined thermal and mechanical actions. It has not been the scope of the investigation to give a full explanation of all the mechanisms and phenomena observed during the tests. The scope is rather to define a test programme that are fairly easy to carry out and furthermore, provides valuable/reliable information of the concrete behaviour that can be implemented in practical design of concrete members. The tests also provide information on the degree of experimental scatter to be expected.

The reported test programme is a part of a large project on environmentally friendly concrete technology called “Green Concrete”. The concrete mixes being subject to high temperature testing have also been tested thoroughly with respect to mechanical properties, fresh properties and durability. The general conclusion of that project is that the green concretes basically behave similar to the conventional concretes (www.greenconcrete.dk).

2. EXPERIMENTAL PROGRAMME

High temperature tests are performed in a gas-fired furnace at the Fire Laboratory, Aalborg University. A test rig is established with test cylinders placed horizontally instrumented with quartz bars (Fig. 1). These quartz bars are lead through the furnace wall to the outside where they are connected to displacement transducers. Three quartz bars register the displacements of each end of the specimen. The measuring length is 400 mm.

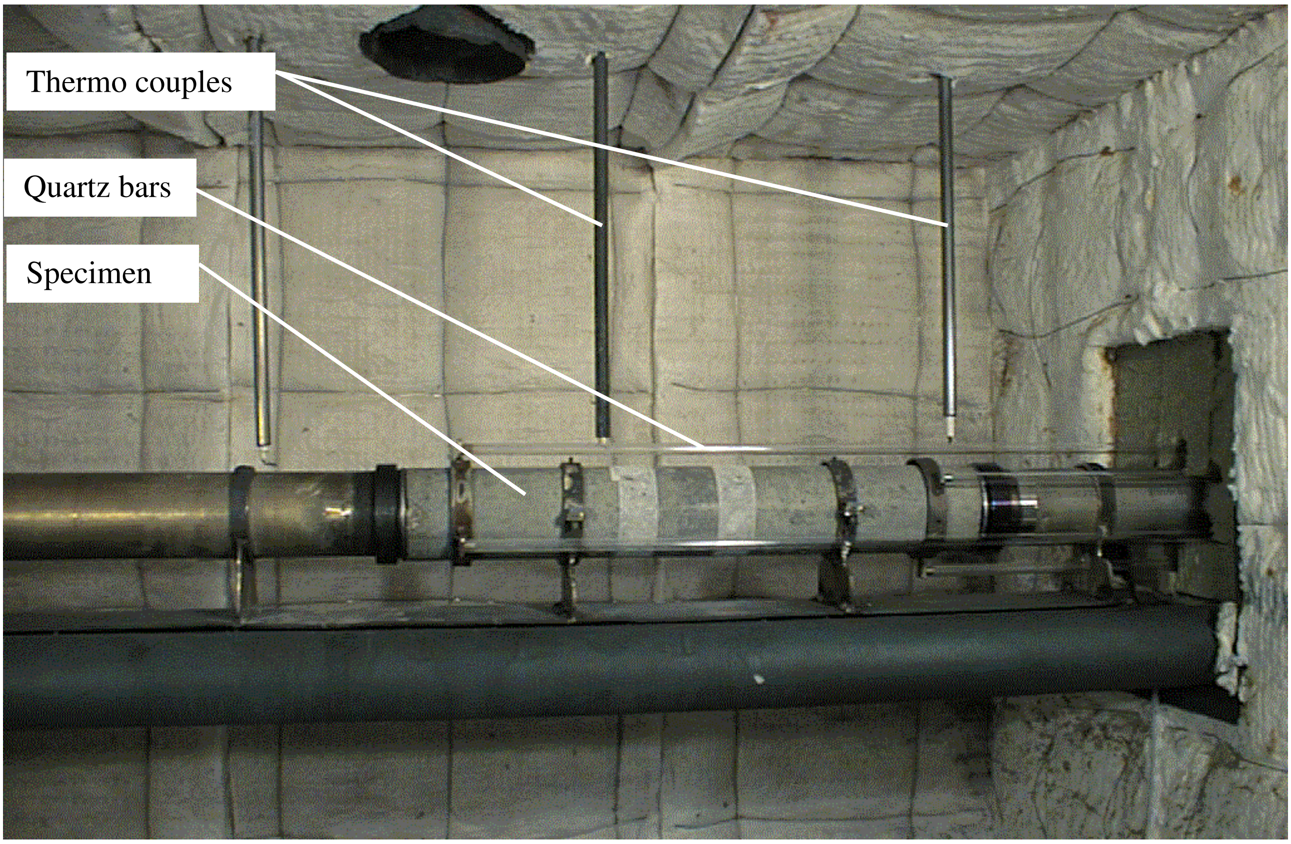


Figure 1 – Inside of furnace with test specimen. The dimensions of the test cylinders are diameter 83 mm and length 575 mm. Each specimen is equipped with two thermo couples. One is cast in the centre of the specimen the other is cast to a depth of approximately 10 mm.

2.1 Concrete mixes

Three different concretes are tested according to the programme given in the next section. One Swedish mix design and two Danish mixes are applied (Table 1).

Table 1 – Concrete mixes in kg/m^3. Notes: [a] Danish rapid hardening cement (CEM I) for PR and P5 and Swedish Bygg C cement (CEM II) for P7. [b] Concrete slurry. [c] Fly ash from biological incineration. [d] Max. size 32 mm. [e] Max. size 25 mm. [f] Equiv. ratio based on reactivity factors of 0.5 for fly ash and 2 for silica. [g] Mean values measured on 100×200 mm cast cylinders.

Material	Concrete PR (Danish reference concrete)	Concrete P5 (Danish green concrete)	Concrete P7 (Swedish green concrete)
Cement[a]	170	168	194
Fly ash	60	60	-
Silica fume	14	13	-
Waste product	-	18[b]	79[c]
Water	143	156	167
Sand	725	692	1008
Coarse aggregate	1170[d]	1172[d]	904[e]
Plast. (Conplast 212)	0.75	0.71	-
(Peramin V)	-	-	0.45
Super plast. (Peramin F)	-	-	0.71
Equiv. w/c[f]	0.63	0.70	0.71
28 days strength[g]	35 MPa	32 MPa	29 MPa
Age at testing	3 – 6 months	4 – 6 months	5 – 8 months

The concretes are typical Danish/Swedish mix designs for indoors structural use (passive environmental exposure class). Due to the fact that the experiments are an integrated part of a large project on environmentally friendly concrete various waste products are reused in the mixes. Concrete PR is a Danish reference mix without any green modifications. Concrete P5 corresponds with PR substituting 2-3 % of the sand fraction with concrete slurry. Concrete P7 is a typical Swedish mix design where some of the cement has been replaced with fly ash from biological incineration.

The main difference between the three concretes lies in the types of aggregate. The volumetric content of aggregate is calculated to 73, 72 and 71 % for PR, P5 and P7, respectively, making the results directly comparable. In PR and P5 the sand is quartz sand and the coarse aggregate is uncrushed sea material, which may be flint, granite, limestone etc. The Danish sea material is known to be a very inhomogeneous material. No detailed mineralogical analysis of the aggregate has been performed. The Swedish concrete contains crushed rock material consisting of amfibolite, diabase and gneiss, i.e. siliceous aggregate.

After casting the specimens are air-cured in the laboratory wrapped in plastic. A month before testing the plastic for is stripped and the ends are saw-cut in order to obtain plane and parallel specimen ends. The tests are performed over a 3-month period after at least 3 months of curing.

2.2 Test execution

Three different types of experiments are performed (Fig. 2). Uniaxial compressive stress – strain curves are recorded at three different nominal temperature levels (200, 400 and 600 °C) beside room temperature. These stress – strain tests are performed without any load applied during heating and cooling. Furthermore, transient tests are performed where the specimen is subject to constant stress during heating and cooling and finally tests where the specimen elongation is restrained. Two specimens are repeated for each set of test conditions, giving about 20 specimens for each of the three concretes.

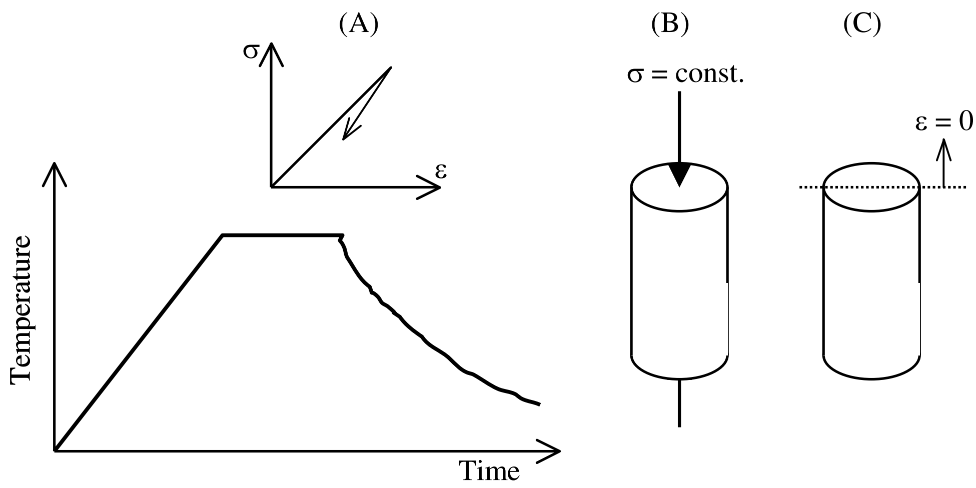


Figure 2 – Experiments with heating and loading of specimens. Test A: Stress – strain curves at constant elevated temperature levels. Test B: Heating and subsequent cooling under constant compressive stress (= 0, 11, 22, 44 % of cold strength). Test C: Restrained elongation.

The temperature of the furnace is recorded as the mean value of 3 thermo couples located just above the specimen (Fig. 1) together with the specimen temperature. In Fig 3 an example of a heating and cooling scenario is depicted. The heating rate is 3-4 °C per minute and it is maintained by manually controlling the gas temperature in the furnace. Turning off the burners and leaving the specimen inside the furnace provide the cooling process.

The temperature difference is seen to increase rapidly to a level of about 50 °C followed by a further increase to 75 °C due to the vaporisation and escape of free moisture from the specimen. This effect manifests itself by delaying the temperature increase within the central part. The cooling process reverses the temperature difference. It is seen how the specimen experiences a thermal chock within the first half-hour of cooling.

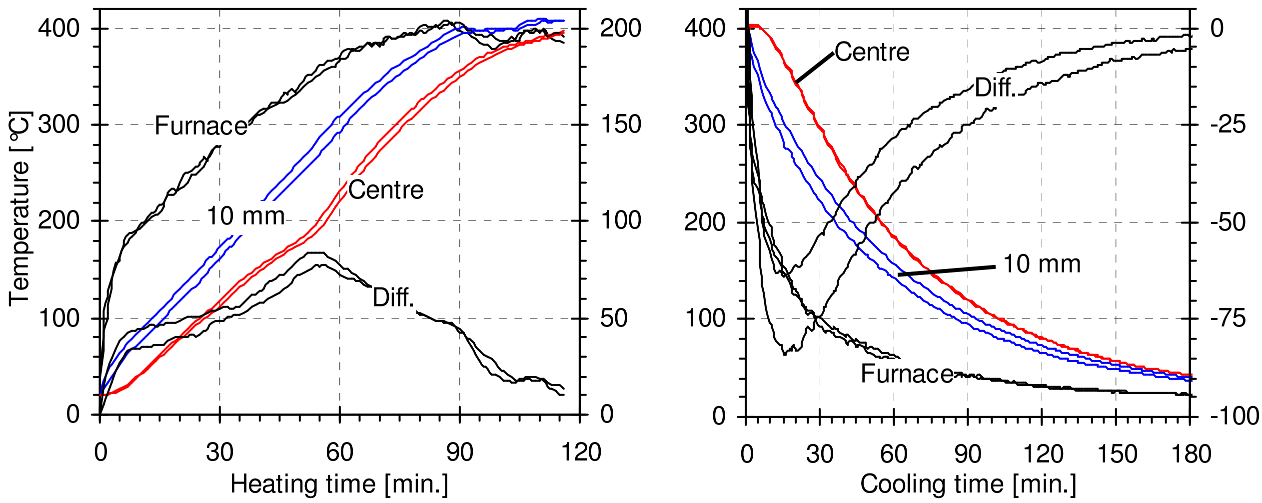


Figure 3 – Temperature recordings shown for two specimens heated to 400 °C with subsequent natural cooling. The temperature differences between centre and 10 mm depth are depicted on the right-hand axes.

3. EXPERIMENTAL RESULTS

3.1 Mechanical properties

The 28-days compressive strengths measured on standard cylinders are given in Table 1. The reference strength at room temperature, f_c^0, is also determined on cylinders saw-cut from the test specimens at the time of testing. The average strengths determined on these cylinders with diameter 83 mm and length 150 mm are 41, 39 and 30 MPa for PR, P5 and P7, respectively.

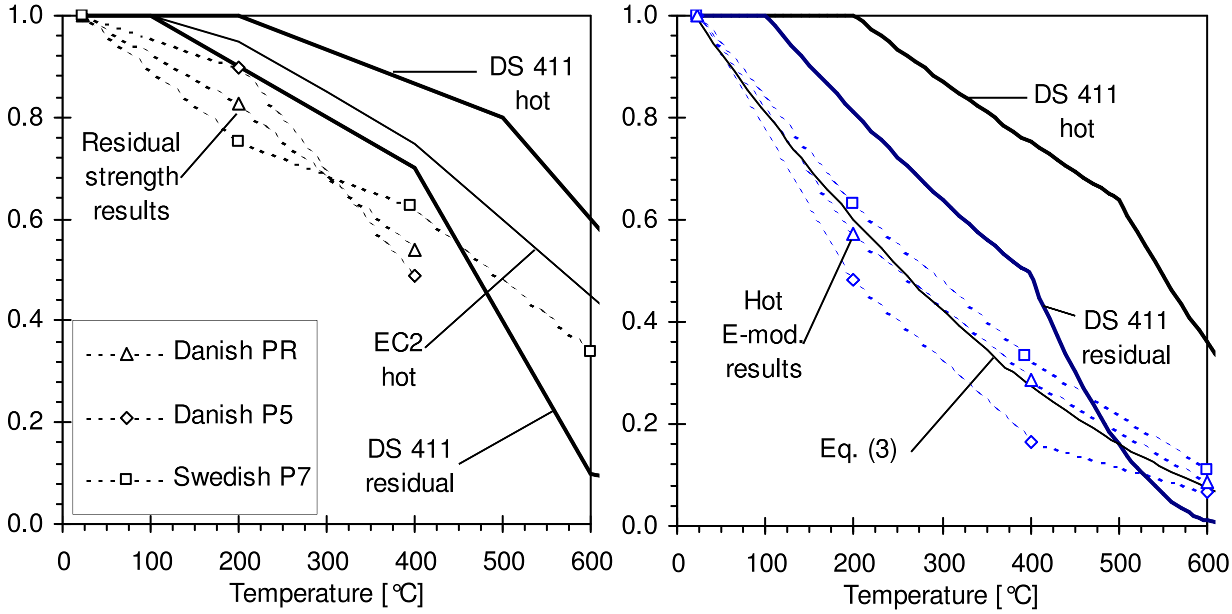


Figure 4 – Strength and stiffness reduction factors. Left: measured residual strengths normalised with respect to the cold strength f_c^0. Right: measured hot E-modulus normalised with respect to the cold reference value E_c^0. A reduction factor is defined as the ratio between the property measured after temperature exposure divided by its value at room temperature.

Test A (Fig. 2) is used to observe the effect of heating on the E-modulus. After reaching a certain maximum temperature the specimen is loaded up to about half its strength level and unloaded before it is left to cool back to room temperature. For reference the loading is also applied to specimens at room temperature. The reference E-modulus is determined to 35, 30 and 27 GPa for PR, P5 and P7, respectively.

After cooling the test specimens used in test A are saw-cut into short cylinders. Then after a couple of weeks in the laboratory they are tested for the residual compressive strength. It is assumed that the load cycle at high temperature does not influence the residual strength.

Figure 4 depicts how the residual strength and the hot E-modulus are found to decrease with the nominal maximum exposure temperature during heating. It is seen that for PR and P5 the residual strength after 600 °C is not determined because the specimens were too fragile to handle after cooling. The observed residual strength degradation is found to be slightly below the behaviour suggested by the Danish concrete code DS 411 [5].

Figure 4 also includes the hot strength reduction factor suggested by Eurocode 2 [6] for siliceous concrete together with the reduction factors given by the DS 411 [5] for strength and stiffness. DS 411 denotes the strength reduction factor by ξ_c and it defines the reduction factor on the E-modulus as $\xi_E = \xi_c^2$, a relationship that is originally proposed by Hertz [2].
Regarding the reduction factor on the E-modulus it is seen that the test results indicate a much more severe decrease than what is recommended in DS 411 for hot conditions. Note that DS 411 applies a convex relationship whereas the experimental evidence suggests a concave relationship, which is also supported experimentally in other tests [3,4].

3.2 Free thermal strains

During heating without external restraints the aggregate expands while the cement paste shrinks due to moisture being expelled (first the free evaporable water followed by chemically bound water). Thus, the thermal strains of concrete are based on two mechanisms of opposite sign. However, the thermal expansion of concrete is mainly governed by the properties of the aggregate and especially the content of quartz is important [1-4]. Furthermore, the magnitude of the thermal strains depends on the initial moisture content and the rate of heating. In [3] it is shown how a wet concrete sample shows smaller thermal strains than a dried-out sample due to the increased shrinkage. The same applies for a sample that is heated very slowly.

The dependency of the free thermal strains on the concrete temperature is depicted in Fig. 5 for heating and subsequent cooling. During heating the two Danish concretes PR and P5 behave almost identically showing much larger thermal strains than P7 especially after exceeding 400 °C. This significant difference in behaviour is assumingly due to the Danish aggregate not being as thermally stable as the Swedish aggregate. At 600 °C concrete P7 shows only half the thermal strain of PR and P5.

The fact that PR and P5 show very good experimental agreement in Fig. 5 indicates that the green modification of P5 (adding of concrete slurry, Table 1) does not alter its behaviour with regard to high temperatures.

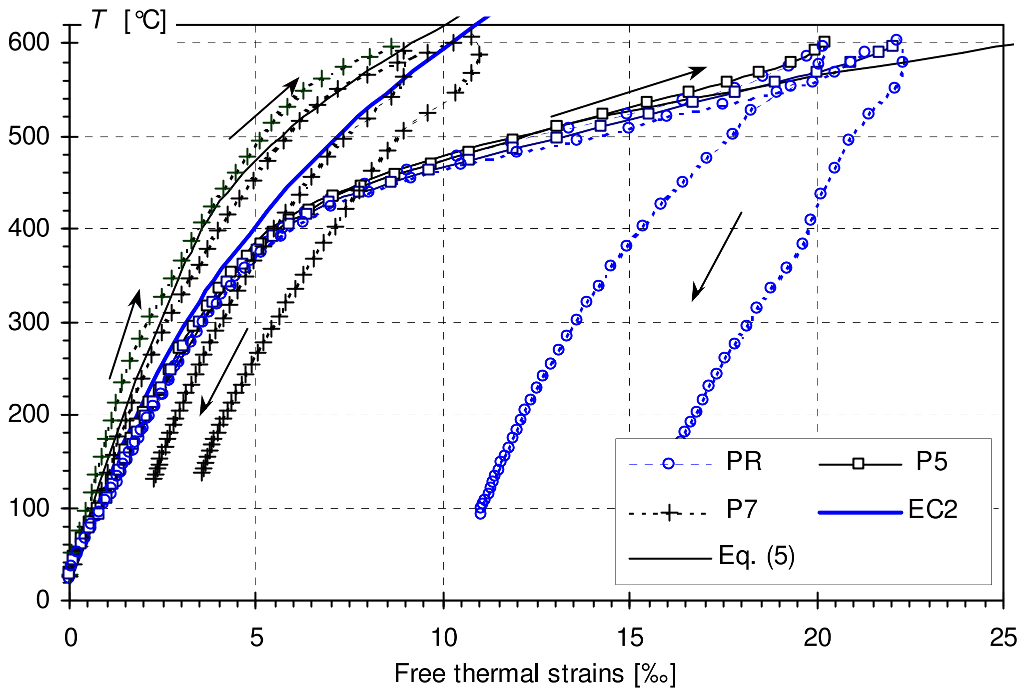
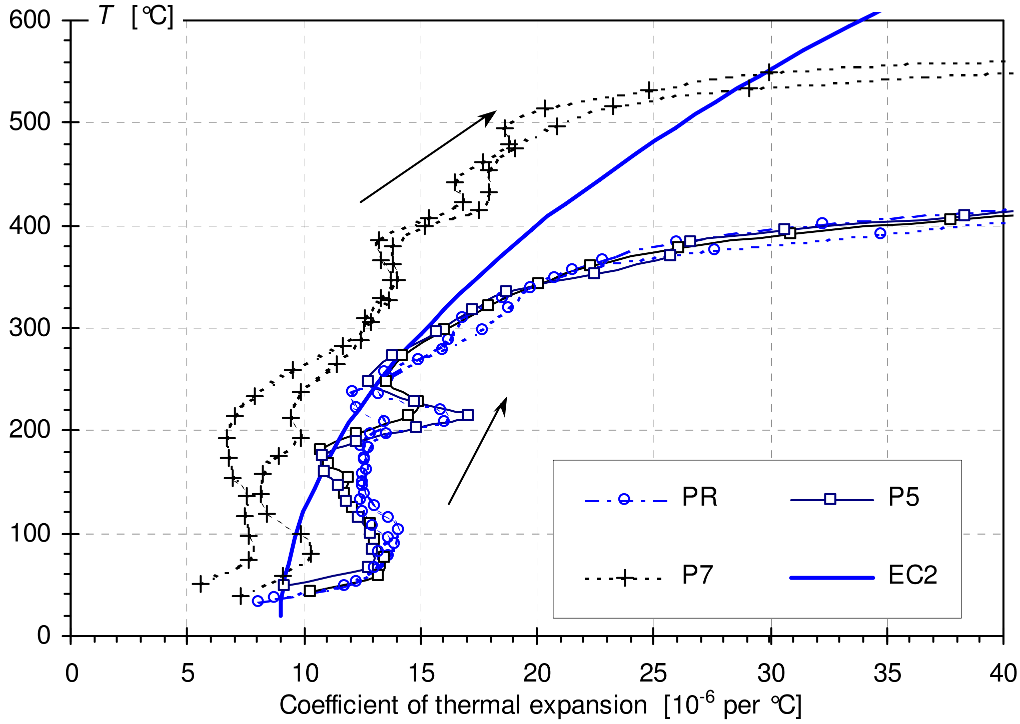


Figure 5 – Recorded thermal strains vs. temperature. The bold line depicts the function recommended in [6] for concrete with siliceous aggregates.

The P7 results are found to be in good agreement with those reported in [3] and the PR and P5 results are comparable in magnitude with results on gravel concrete reported in [1]. Figure 5 also contains a 3rd order polynomial recommended by Eurocode 2 [6]. This expression is seen to be slightly higher than P7 at all temperatures.

During cooling it is seen that the concretes behave almost identically corresponding to a thermal coefficient of expansion equal to approximately 15×10^{-6} per °C (Fig. 5). It is noted that the cooling process was not registered for concrete P5 by mistake.

After cooling back to room temperature the Danish concrete PR is seen to have a significant residual elongation due to cracks in the binder. The Swedish concrete P7 also showed some residual elongation about an order of magnitude smaller than PR.



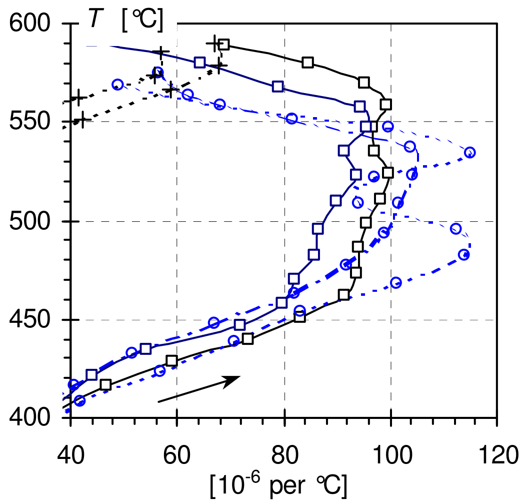


Figure 6 – Derivative of strain data in Fig. 5 with respect to temperature during heating. The final part of the curves is shown in the small diagram.

In Fig. 6 the thermal strain data are differentiated numerically with respect to temperature to obtain the coefficient of thermal expansion $\alpha = d\varepsilon_{th}/dT$. This diagram again demonstrates how the Swedish concrete shows significantly smaller expansion than the Danish concretes.

The significant increase in α found for P7 between 500 and 550 °C is probably due to inversion of quartz. For PR and P5 the coefficient of expansion changes from 10 to 15×10^{-6} per °C below 300 °C up to almost 100×10^{-6} per °C at 500 °C. Then the coefficient starts to decrease back to zero. However at this point the heating is stopped and the post 600 °C behaviour is not observed in the present investigation.

The variation of α in the temperature range 100 – 250 °C is due to the transport of free moisture that is taking place (cf. temperature recordings in Fig. 3). These variations are not detected during the cooling process.

3.3 Transient thermo-mechanical strains

General description
The term creep is used for time dependent deformations in concrete. Creep deformations typically take place over a time range of months and even years. An unsealed and unloaded test determines shrinkage and a sealed loaded test determines basic creep. However, if the creep test is carried out unsealed the total strain exceeds the sum of shrinkage and basic creep (Fig. 7). The difference is most often termed drying creep or load-induced shrinkage [7].

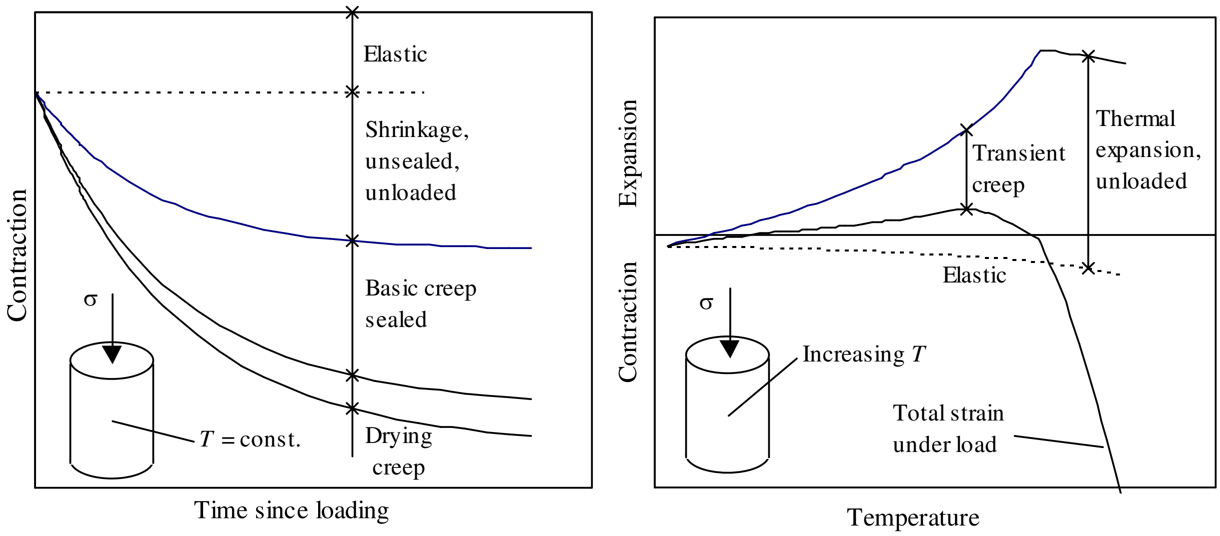


Figure 7 – Illustration of conventional creep strains (left) and transient thermal creep (right).

When concrete is heated there is a large difference whether the temperature conditions are stationary or transient. Under transient conditions the total strains are found not to consist of only free thermal strains and mechanical strains. The total strains also contain a component, which is often termed transient creep (Fig. 7). However, the term creep seems to be misleading since conventional creep also exists under isothermal conditions [3,4,8]. Furthermore, a transient-heating situation is most often relatively short-term and therefore the name thermo-mechanical strain is used instead of transient creep. The phenomenon is also sometimes termed load-induced thermal strain [1]. The thermo-mechanical strains only occur during heating and not during subsequent cooling as it are demonstrated in the following. Furthermore, thermo-mechanical strains are reported to occur only during first heating [1]. In the present section the results are briefly presented and in the next chapter they are further analysed.

Test results
Figures 8 and 9 depict the strain recordings during tests of type B. The strains do not include the mechanical strain from initial loading. Three compressive stress levels are applied beside the unstressed tests shown in the previous section (Fig. 5). The effect of combined heating and stress is clearly seen. The free thermal expansion results (0 % loading) are represented by solid lines based on the expressions presented in Section 4.2.

In order to demonstrate that thermo-mechanical strains only occur during heating, Fig. 8 also contains recordings from the cooling period. It is seen that the contraction that takes place during cooling is practically independent of the compressive stress level.

In all cases the heating is ended before the specimen fails in order to avoid damage to the quartz bars. It is seen that the compressive stress avoids the free thermal expansion to take place. The physical mechanisms behind this phenomenon are not fully understood. However, the effect is seated in the cement paste and therefore it is expected that the stress state enables the (negative) shrinkage strains to develop and counteracts internal cracking perpendicular to the load axis.

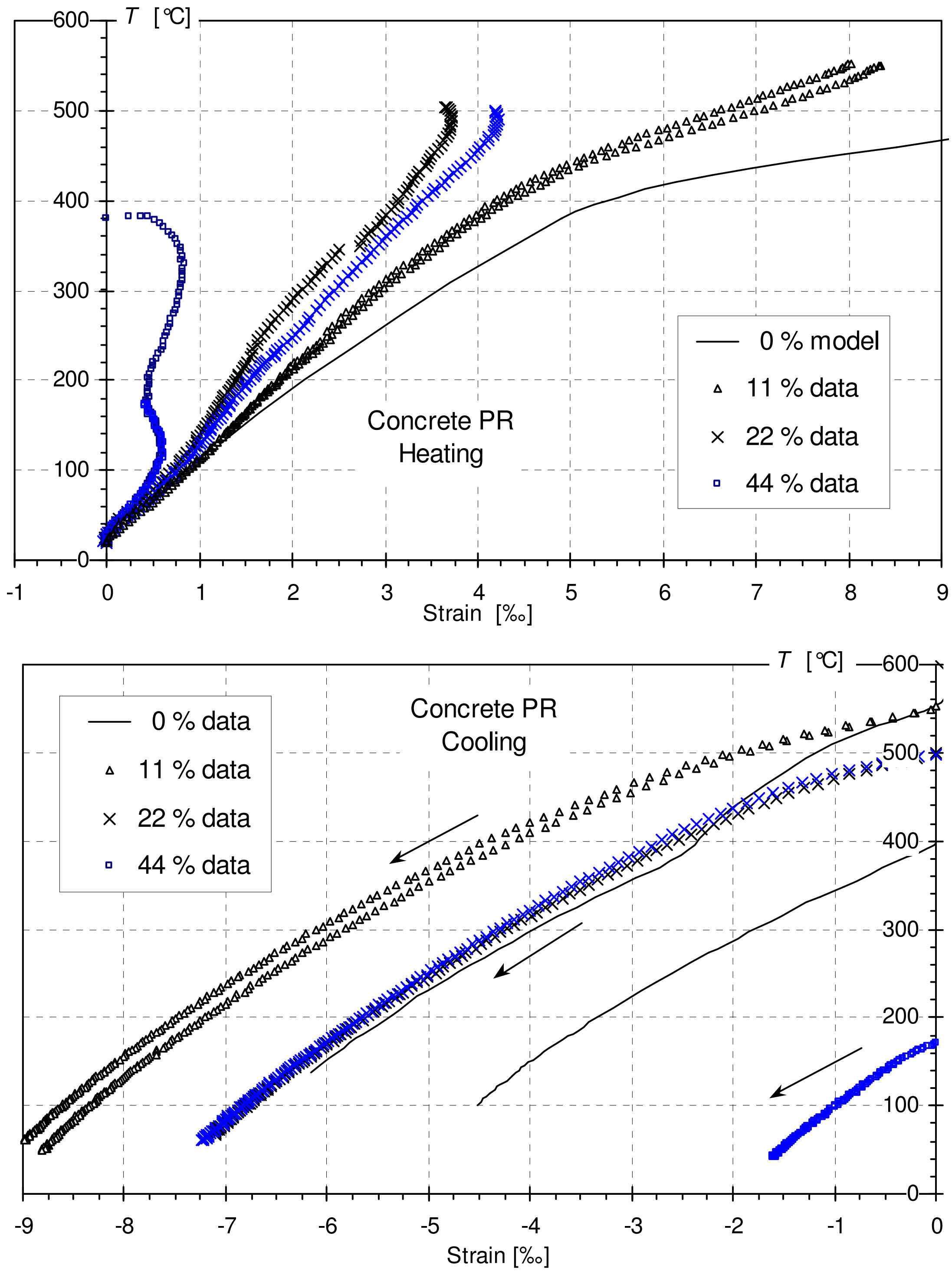


Figure 8 – Total strain data recorded during heating and cooling on PR. Positive strain is elongation. The legend numbers denote constant compressive stress in per cent of f_c^0.

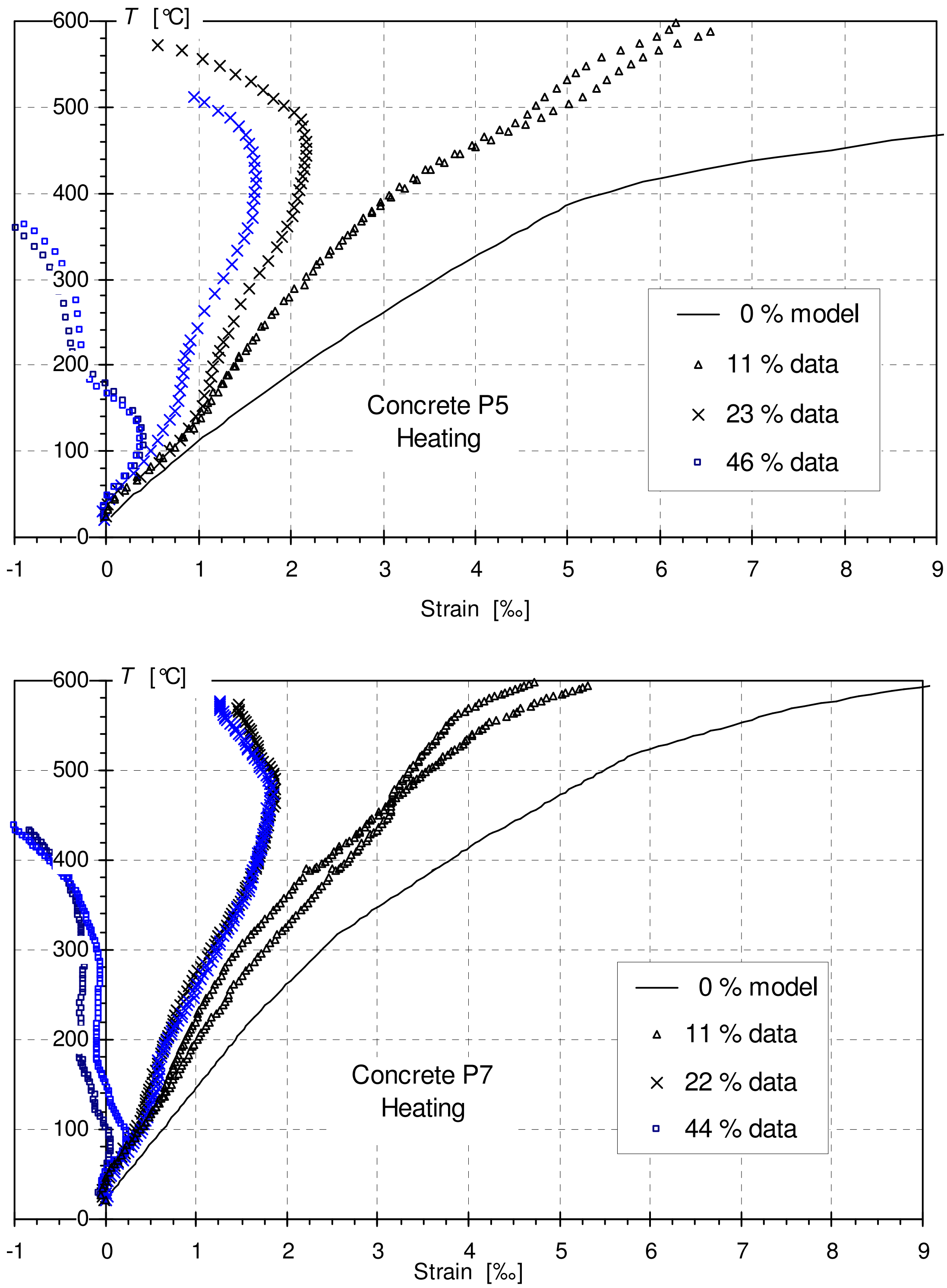


Figure 9 – Total strain data recorded during heating for P5 and P7. Positive strain is elongation. The legend numbers denote constant compressive stress in per cent of f_c^0.

4. THEORETICAL MODELING

In order to perform numerical calculations on the behaviour of concrete subject to varying high temperatures and stresses the constitutive relationship needs to be established. In the following this is done in the uniaxial case based on the test results described in the previous section.

The following formulation of transient strains under varying heating and loading is adopted:

$$\varepsilon_{tot}(T,\sigma) = \varepsilon_{el}(T,\sigma) + \varepsilon_{th}(T) + \varepsilon_{th}^{\sigma}(T,\sigma) \tag{1}$$

where the three components on the right-hand-side correspond to mechanical elastic strain, free thermal strain and combined thermo-mechanical strain, respectively. Temperature is T and stress is σ. Each of the three strain components is treated in the following sections. This formulation is in correspondence with the normal approach [2-4]. Note that (1) does not include conventional creep taking place under sustained loading and isothermal conditions. It is generally recognised that during the short-time duration of a high temperature test the conventional creep strain is negligible compared with the other strain components [3].

In the article stress is only taken as externally applied load. However, the presence of temperature gradients (Fig. 3) produces internal eigen-stresses, which are not considered here.

4.1 Model of mechanical strains

The mechanical strains are formulated by means of a temperature dependent elastic modulus

$$\varepsilon_{el}(T,\sigma) = \frac{\sigma}{E_c(T)} = \frac{\sigma}{E_c^0 \xi_E(T)} \tag{2}$$

where E_c = elastic modulus of concrete and E_c^0 = reference value at room temperature. This approach does not include plastic strains in order to keep the model simple. However, it is realised that in order to model high temperature response realistically the plastic strains need to be included. Several models are already suggested to include non-linear stress-strain behaviour at high temperatures [2-4].

It is proposed to apply a separate reduction factor on the E-modulus instead of connecting the reduction factors for strength and stiffness as it is done in DS 411 [5].

The test results from Fig. 4 are used to model the reduction factor on the E-modulus. The following parabolic expression is suggested to fit the test results:

$$\xi_E(T) = \left(1 - \frac{\Delta T}{T_0}\right)^2, \qquad \xi_E = \frac{E_c(T)}{E_c^0}, \qquad 0 \le \Delta T \le T_0 \tag{3}$$

where ΔT = heating temperature above room temperature (i.e. $\Delta T = T - 20$ °C) and T_0 is a model parameter determining the temperature rise where the reduction factor equals zero. In Fig. 4 a line with T_0 = 800 °C is depicted for illustration purposes. A more general model could easily be formulated if there is the need for it. For instance the power 2 could be treated as a model parameter along with T_0.

The model in (3) does not include any influence from compressive stresses during the heating. It is generally acknowledged that stresses during heating increases the stiffness as well as the strength of the concrete due to the fact that the stresses inhibit the development of cracking. In Fig. 10 the temperature influence on the elastic modulus is illustrated for specimens heated under constant compressive stress [4]. It is seen how the parabolic model in (3) is plausible to predict the reduction factor. The effect of stress level is seen to be most significant at high temperatures. Furthermore, a straight line from the starting point to the end point of (3) is seen to provide a reasonable model for this stress effect. However, the amount of test data is insufficient to give a more detailed model for the stiffening effect of sustained loading under heating.

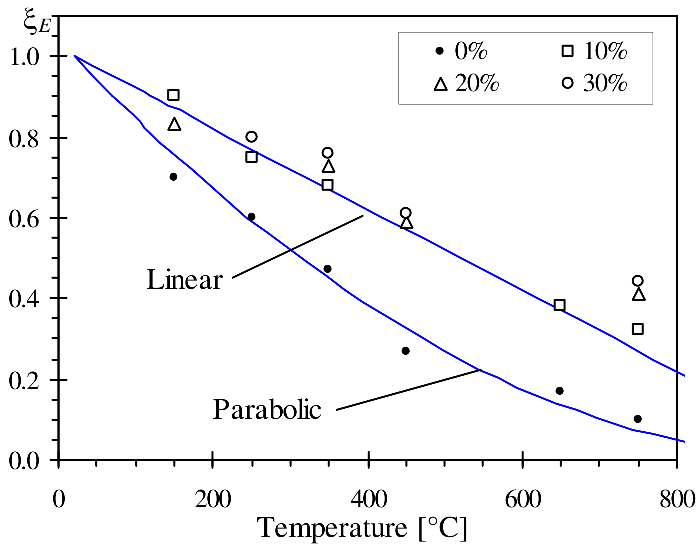


Figure 10 – Reduction factor for E-modulus for normal concrete. Data taken from [4]. The legend numbers denote compressive stress applied during heating in per cent of f_c^0. The parabolic curve is given by (3) with T_0 = 1000 °C and the linear curve connects the end points of the parabola.

4.2 Model of free thermal strains

The free thermal strains depend solely on the temperature:

$$\varepsilon_{th}(T) = \int \alpha(T)\, dT \tag{4}$$

where α = coefficient of thermal expansion and the integration is performed over the temperature range in question. Typically the coefficient of thermal expansion is approximately 10×10^{-6} per °C at room temperature for normal concrete. However, as the temperature increases the coefficient also increases (Fig. 6).

In order to model the abrupt change in behaviour observed during the experiments a bi-linear model of the coefficient of thermal expansion is applied. By integration this results in thermal strains following a parabolic function of temperature:

$$\varepsilon_{th}\big(\theta(T)\big) = \begin{cases} A\theta^2 + B\theta, & 0 \le \theta \le \theta^* \\ C(\theta-\theta^*)^2 + A(2\theta-\theta^*)\theta^* + B\theta, & \theta^* < \theta \end{cases} \tag{5}$$

The three parameters A, B and C together with the transition temperature θ* determine the thermal strain – temperature relationship. The temperature is normalised in order to simplify the notation:

$$\theta = \frac{\Delta T}{100^{\circ} C} \geq 0 \tag{6}$$

where it is recalled that ΔT is the temperature rise above room temperature.

In Fig. 5 the parameters of Table 2 are applied in (5). It is noted that the analytical expression should only be used within the temperature range where it is experimentally verified.

Table 2 – Parameters for free thermal strain function in (5). Transition temperature = 380 °C.

Parameters	A	B	C	θ*
Concrete	[‰]	[‰]	[‰]	[-]
PR and P5	0.1	1.0	3.5	3.6
P7	0.06	0.7	0.7	3.6

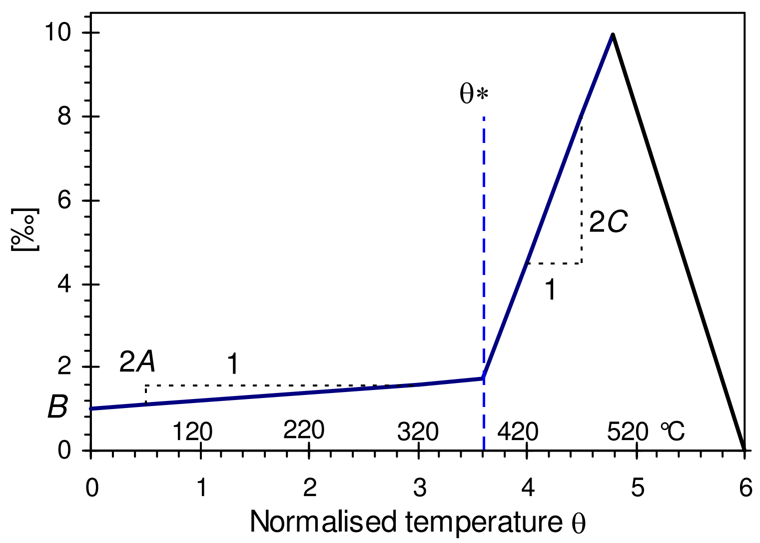


Figure 11 – Illustration of derivative $d\varepsilon_{th}/d\theta$ vs. normalised temperature. Based on parameters for PR and P5.

In Fig. 11 the influence of the parameters in (5) is illustrated. B governs the initial value of the coefficient of expansion at room temperature while A and C govern the slopes of the bi-linear curve. Furthermore, an upper limit corresponding to $\alpha=100\times10^{-6}$ per °C is applied in accordance with the experimental observations in Fig. 6, followed by a decreasing coefficient of thermal expansion. Note that the coefficient of thermal expansion is not determined experimentally beyond 600 °C. However tests shows that the coefficient of expansion reduces to zero after 600 – 700 °C [1-4,6].

4.3 Model of thermo-mechanical strains

The thermo-mechanical strain is assumed to be proportional with the stress level:

$$\varepsilon_{th}^{\sigma}(T,\sigma) = \frac{\sigma}{f_c^0}\tilde{\varepsilon}(T) \tag{7}$$

where it should be noted that $\tilde{\varepsilon}$ is zero during cooling and under isothermal conditions. Thus, the thermo-mechanical strains that develop during heating are considered irreversible. By inserting (2) and (7) in (1) the load dependent strains are formulated as

$$\varepsilon_{el} + \varepsilon_{th}^{\sigma} = \frac{\sigma}{f_c^0}\left(\tilde{\varepsilon} + \frac{f_c^0}{\xi_E E_c^0}\right) = \frac{\sigma}{f_c^0}\varepsilon^{\sigma} = \varepsilon_{tot} - \varepsilon_{th} \tag{8}$$

where $\tilde{\varepsilon}$ = normalised thermo-mechanical strain and ε^{σ} = normalised load dependent strain.

Since the total strain measurements obtained from a type B test (Figs. 8 and 9) are obtained under constant stress conditions they may be used to quantify ε^{σ} by means of (8). In Fig. 12 the test data are converted into ε^{σ} values with ε_{th} given analytically in (5).

The broken lines in Fig. 12 depict the mechanical part of the strains. Line a is based on a linearly decreasing E-modulus (Fig. 10) and line b is based on the parabolic decreasing E-modulus governed by (3). Thereby, line a includes the strengthening effect of sustained stress during heating while line b neglects this effect. The importance of including the correct temperature degrading effect on the E-modulus is clearly seen especially when the temperature exceeds 400 °C. The physically correct behaviour is probably lying somewhere in between line a and b.

The difference between the data symbols and the dotted line in Fig. 12 is equal to the normalised thermo-mechanical strain $\tilde{\varepsilon}$ in (7). However, instead of performing a subdivision of the load dependent strains into mechanical and thermo-mechanical strains they are kept together in the following description.

The shape of the curves in Fig. 12 generally follows 3 different slopes:

- Below 100 °C the load dependent strain is almost fully elastic and the thermo-mechanical strains are of minor importance.
- From 100 to 400 °C the thermo-mechanical strains start to increase significantly, which is associated with the escape of free and chemically bound moisture from the cement paste.
- Above 400 °C the thermo-mechanical strains increase even faster until the specimen experiences a compressive failure. This behaviour is connected with the formation of cracks within the cement paste. This behaviour is most clearly observed for PR and P5.

By comparison it is found that the test data in Fig. 12 for P7 is in agreement with the Swedish tests performed by Anderberg & Thelandersson [3,8]. Anderberg & Thelandersson suggested the thermo-mechanical strain to be proportional with the thermal strain, i.e. $\tilde{\varepsilon} = k\varepsilon_{th}$. This model has been used extensively in practical applications because of its simplicity; however, the proportionality factor is found to depend on the concrete and the aggregate in question. Normally $1.5 < k < 2.5$ seems plausible.

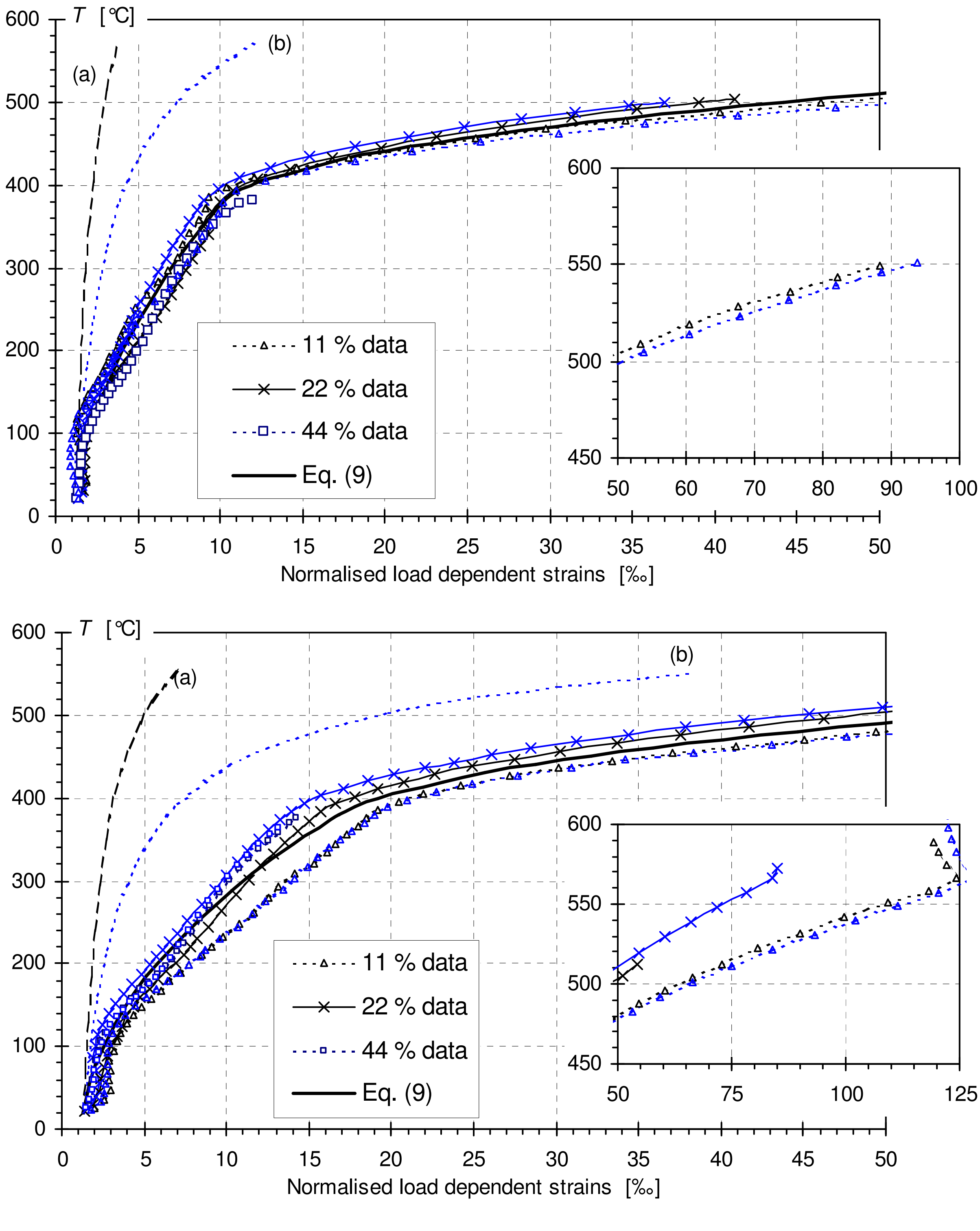


Figure 12 – Normalised load dependent strains ε^{σ} vs. temperature for PR (top) and P5 (bottom) under heating and constant stress (type B tests). The inserted diagrams depict the measurements beyond 50 ‰.

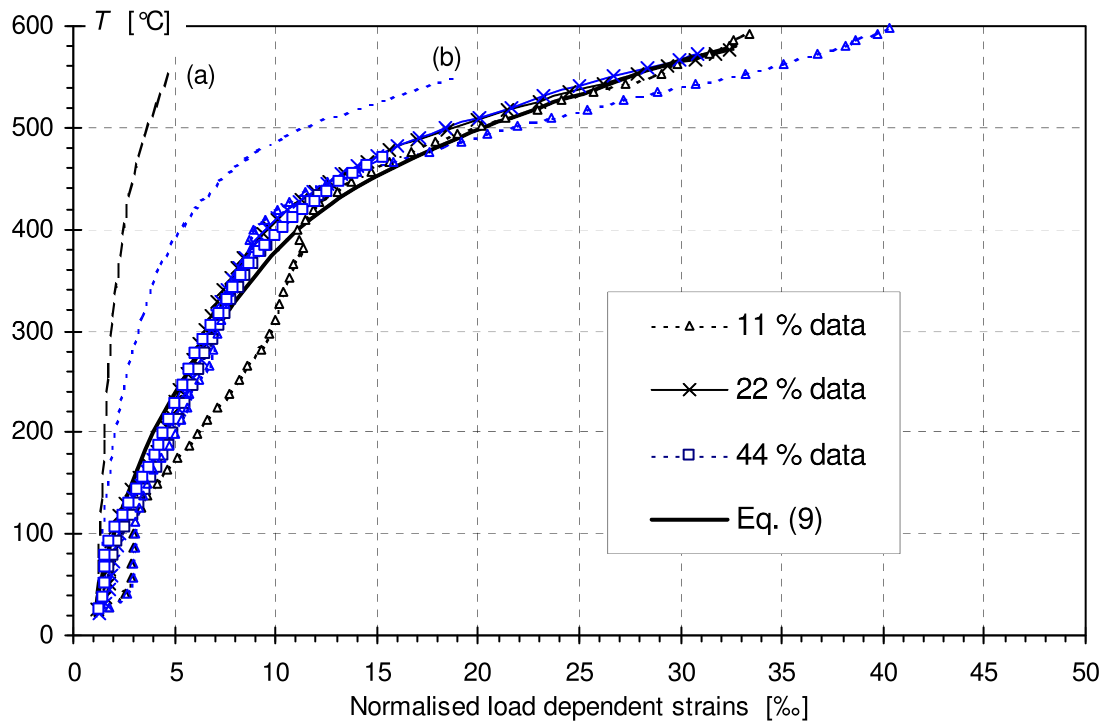


Figure 12 – …continued. Normalised load dependent strains for P7.

In the present article the normalised load dependent strains during heating are modelled by means of an expression similar to (5):

$$\varepsilon^{\sigma} = \tilde{\varepsilon} + \frac{f_c^0}{\xi_E E_c^0} = \frac{f_c^0}{E_c^0} + \begin{cases} A_1\theta^2 + B_1\theta, & 0 \le \theta \le \theta^* \\ C_1(\theta-\theta^*)^2 + A_1(2\theta-\theta^*)\theta^* + B_1\theta, & \theta^* < \theta \end{cases} \tag{9}$$

where subscript 1 have been added to the three parameters in order to separate them from the thermal strain parameters. In Table 3 the parameters are given corresponding to the lines depicted in Fig. 12. It is seen that P7 generally experiences the smallest thermo-mechanical strains followed by PR and P5. Furthermore, the test results show a rather large scatter especially at high temperatures. Therefore, the suggested model should be used with care and not taken as an exact description of the behaviour.

Table 3 – Parameters for normalized load dependent strains modeled in (9).

Parameters Concrete	A_1 [‰]	B_1 [‰]	C_1 [‰]	θ* [-]
PR	0.5	0.7	20.0	
P5	1.0	0.7	20.0	3.6
P7	0.5	0.7	3.5	

4.4 Verification of model through restrained deformation tests

In order to illustrate the effect of the various strain components under heating, tests of type C have been performed (Fig. 2). A test specimen is heated while its end planes are fully restrained. Thus, compressive stresses build up in the specimen under heating. These stresses may be calculated according to the following equation where $\varepsilon_{tot} = 0$ is applied in (8):

$$\frac{\sigma}{f_c^0} = -\frac{\varepsilon_{th}}{\varepsilon^{\sigma}} \tag{10}$$

The thermal strains modelled in (5) together with the normalised load dependent strains modelled in (9) are applied with parameters given in Table 2 and 3, respectively.

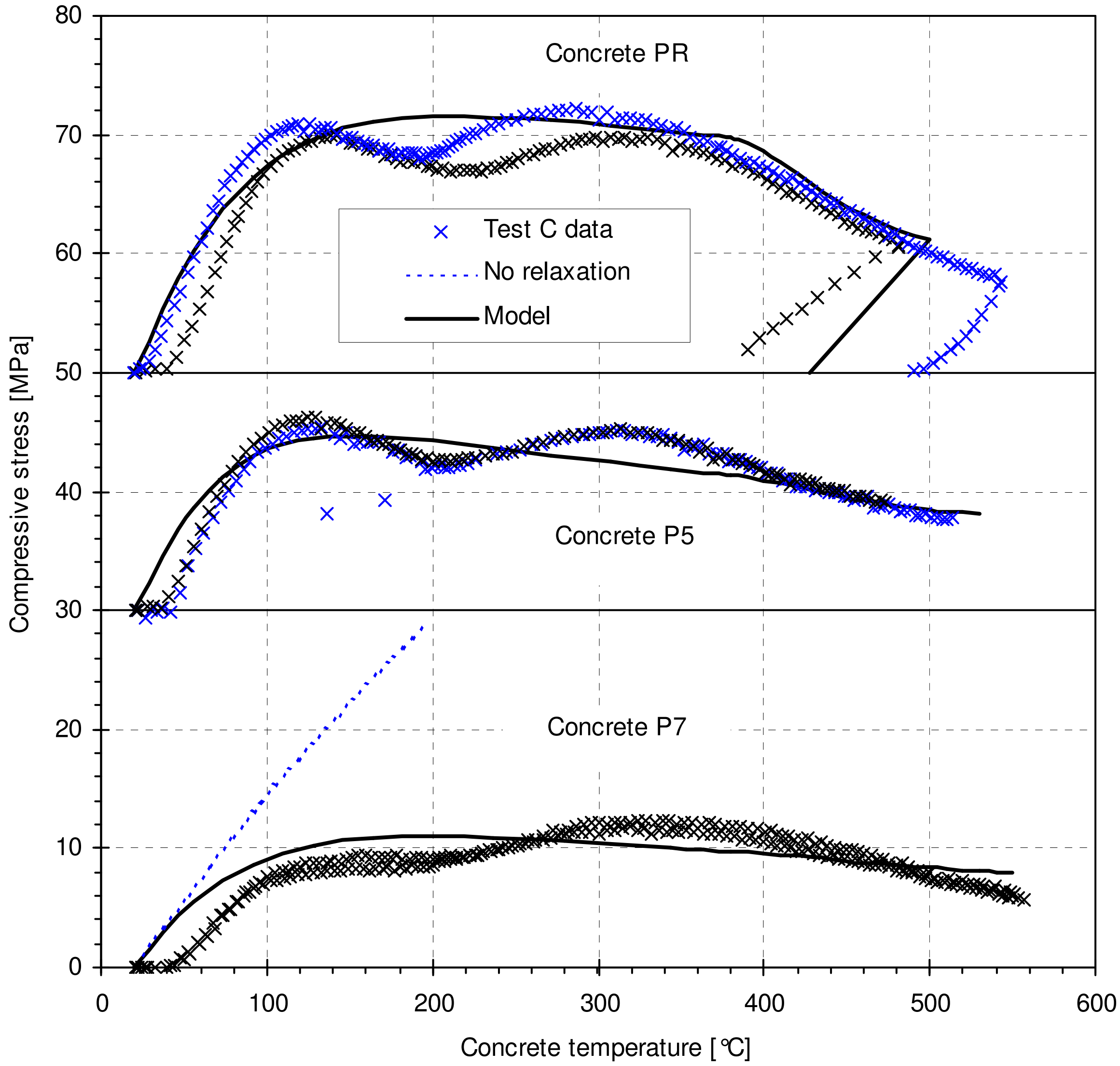


Figure 13 – Restraining stresses during type C test. Note that PR and P5 are shifted along stress axis in order to separate the curves. For PR the stresses are also registered during cooling. Two tests are performed for each concrete.

In Fig. 13 the measured restraining stresses are depicted together with model calculations based on (10). The model calculations are found to be in good agreement with the measurements, finding the correct order of magnitude for the stresses. However, the model cannot predict the local stress peaks that are due to small variations in the coefficient of thermal expansion and in the thermo-mechanical strain. These variations that are mainly caused by vapour transport are not included in the proposed analytical model.

The dotted line in Fig. 13 illustrates a situation neglecting the thermo-mechanical strains, i.e. $\tilde{\varepsilon} = 0$ in (10), i.e. the specimen feels no stress relaxation. It is clearly seen that neglecting the thermo-mechanical strains leads to erroneous results.

During cooling the specimen contracts without any recovery of the thermo mechanical strains and after almost 100 °C temperature drop the specimen is stress-free. In Fig. 13 the stress loss during cooling is calculated for PR based on the hot E-modulus at 500 °C (Fig. 4) and a constant coefficient of thermal strain equal to 15×10^{-6} per °C (Fig. 5). The correspondence with the test data is seen to be satisfactory.

If the specimen is restrained in tension as well as in compression the cooling process produces tensile stresses. This situation is often the case when concrete members are subject to fire and at the same time restrained against deformations due to adjoining structural members. See [9] for an example of this build up of tensile stresses during cooling.

5. CONCLUSIONS

The high temperature tests have resulted in the following main conclusions on the tested types of concrete:

1. The residual strengths are found to be in reasonable agreement with the normal behaviour reported in the literature and in the fire design codes.
2. The E-modulus determined at elevated temperatures is found to decrease almost identically as a function of temperature for the tested concretes. A new analytical proposal for the stiffness reduction factor as a function of temperature is given.
3. The stiffness reduction factor applied in the Danish concrete code DS 411 is found to overestimate the E-modulus significantly compared with the test results. It is recommended that the reduction factors for strength and stiffness should be separated.
4. For the Danish concretes the free thermal expansion under heating and cooling is found to be much higher than for the Swedish concrete.
5. This divergence between the Danish and the Swedish concretes may have several reasons: (i) it is possible that the behaviour is connected solely with differences in aggregate thermal behaviour; (ii) part of the difference may be associated with the Danish cement paste drying out and reversing its deformations from shrinkage to expansion when temperatures exceed 400 °C, or (iii) the difference is associated with internal damage/micro cracking.
6. In order to fully understand the origin of the above-mentioned differences in thermal deformations a more detailed test programme is needed, including separate investigations of aggregate, cement paste, microscopic testing, etc.
7. Tests with simultaneous loading and heating are used to quantify the magnitude of the thermo-mechanical strains (transient creep). It is experimentally confirmed that thermo-mechanical strains are irreversible during cooling.

8. The thermo-mechanical strains of the Swedish concrete are found to be in agreement with other test results reported in the literature. The Danish concretes show larger thermo-mechanical strains, which is partly attributed to the fact that the sustained load inhibits the internal crack formation during heating.
9. New analytical expressions are proposed for the free thermal expansion and the thermo-mechanical strain as a function of temperature. The validity of the proposed analytical models is found to be satisfactory by comparing with restrained tests under heating.

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Explorations of different means to achieve an industrial process for building bridges; part I – implications out of the current process

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ABSTRACT

In the ongoing multidisciplinary research at Chalmers University of Technology, different means of creating an industrial process for building bridges have been investigated, as a response to the urgent need of development in bridge construction. Out of the basis of structural design, the emphasis in this paper is to establish the foundations for achieving such a new process, basically through analysis of the current bridge construction process, as well as to determine the required improvements and provide a framework for a process model and for the future research. The main focus of the continued work is on investigating techniques and materials developments from a design viewpoint (e.g. concrete structures), in addition to include studies of construction characteristics such as production methods, etc. Hence both product and process development are emphasised.

Key words: concrete bridges, concrete concepts, industrial, process, design and construction, materials development.

1. GENERAL INTRODUCTION

The increasing interest in productivity development as a response to the lack of efficiency in the construction industry has highlighted the need of further research on how to make substantial improvements. Of particular interest for bridges are the benefits that could be extracted from an integrated industrial process of design and construction. In this respect, the great complexity and fragmentation of the construction process are one of the main issues that have to be addressed, not least for concrete bridges. The application of new or approved techniques, materials developments, methods of design and analysis, as well as construction methods are other important areas to be investigated. Recent development in the field of design and construction of structures has often aimed at utilising new or approved materials, for example the latest developments of concrete. The increasing use of information and communication technology (ICT) as well as more advanced computer-based analysis and simulation models are also strong ongoing trends. The major key to the construction concepts of tomorrow will be to combine these factors into an industrial process. Many suggestions on how to solve the problems in construction and improve efficiency have been presented, for example by Koskela [1], Warszawski [2], Sarja [3] and Gibb [4] and some interesting perspectives and opportunities for the next decades are given by Flanagan et al. [5].

However, the studies so far have usually not been carried out in the context of the construction process for bridges, although e.g. fib Bulletin No.9 [6] offer a study of conditions in bridge construction in an international perspective. Moreover, for bridge construction no one seems to have taken the overall approach of emphasising the development of an industrial process that takes account of all participants. Due to its multidisciplinary features, different parts of the process have been covered only separately, without essential linking. Thus, the question arises of how to achieve such a process and whether it really could solve the problems in bridge construction and encourage developments.

The purpose of the research presented in this paper is to lay the foundation for the continued research and to advance towards a new construction process by studying emergent applicable materials developments, techniques, design and construction methods to be adopted in an industrial building process for bridges, primarily concrete bridges. The work, involving many disciplines, focuses primarily on design viewpoints, but also concerns other crucial aspects such as production, economics and quality, while the results should be increased knowledge and suggestions applicable in industrial bridge construction. However, since the main objective is more efficient and rational construction of bridges, and thus includes linking of all its different parts, the point of departure for the research must be the process as a whole. The present paper is intended to fill this need by formulating the problem and defining the framework in which the research is to be carried out, as well as to evaluate directions with the highest potential for improvement.

This article has been divided into two parts, were this first part is devoted to comments and analysis of the current bridge construction process in comparison with developments in direction of a new industrial process. In the second part, the emphasis is turned towards problems and opportunities in an industrial context, were the underlying driving forces acting in favour of an industrial process are investigated and the necessary prerequisites are evaluated. Suggestions of possible solutions to problems are discussed. In addition, some priorities and conclusions about the continued work are summarised. The articles are organised to enable independently reading of each part.

2. A DIFFERENT APPROACH ON HOW TO CONSTRUCT BRIDGES

In this chapter, discussion of the underlying forces driving industrial developments for bridge construction is emphasised, also encapsulating the research problem in a condensed form. The background of this research project is more thoroughly described in Harryson & Gylltoft [7], but a short description in a picture and a few words can be found in Figure 1. The research has a focus on short- and medium-span road bridges, especially those where concrete is included.

2.1 On the agenda of industrial bridge construction

Industrial production in general is no longer equated with mass production; the emphasis has shifted towards a focus on creation of customer value. This should enable a widespread adoption of industrial concepts for bridge construction, since the ideas of mass production were never suited to bridges. Although such trends can be noticed for large bridges, few applications for minor and medium-size bridges have evolved, and this is particularly evident in Swedish bridge construction.

Of course the term ‘industrial construction’ is a bit awkward since it ought to imply in general that the construction industry – being an industry – should already be involved in industrial activities, and thus the need of industrialisation should not arise. This is the reason why it is better to use the term ‘industrial’ to imply contemporary solutions instead of ‘industrialised’, which seems more related to transforming the traditionally craft-based methods into something modern.

A major hindrance to development of new industrial bridge concepts is the prevalent market situation, with e.g. an extensive involvement of the society. The government rises funding for bridges, allowances of construction is complicated and handled by the authorities and commissioners of the same administrations act as public clients on behalf of the direct end-users, i.e. the drivers who use the bridge. Consequently the customer relationship is complex. This can cause confusion in respect of who is to benefit from enhanced customer value, resulting in a market that currently seems to act on lowest price basis solely. Thus no strong market force to pull demands has been noticed which also can be seen as one explanation to the lack of development, although a shift towards a larger commitment to development from the public clients has been noticed recently. The interaction between the industry, the public clients and administrators and the rest of the society regarding these matters has been insufficient so far. In absence of a clear ‘market-pull’, which probably would produce the best result in the short run, it seems like the strategy of ‘technology-push’ will have to be adopted. That is to provide new superior technical solutions and offer them to the clients, making them realise the increased value created. Hence, this will require a long-term commitment to research and development from e.g. the contractors, a commitment that sometimes has been noticed not to coincide with short-term demands from the market. This constitutes a large problem in introducing industrial construction processes. It seems obvious that to create a win-win situation in the long run, the best alternative would be a combination of strong forces of both ‘market-pull’ and ‘technology-push’. That is why the aim of an industrial bridge construction process must be enhanced value to both direct customers and public commissioners, in addition to be beneficial to the other participants as well. This can be interpreted as follows; the overall objective for any industrial construction process is to make the product either at a lower cost or with higher quality at the same cost, giving increased customer value. The main means of doing this is reduction of waste by practically eliminating the uncertainties and the peculiarities of construction (at the price of increased complexity though, as argued below). Other advantages that could be included are more efficient and rational construction, leading to shorter time of construction; improved employee performance, due to better working conditions; better use of resources from a public economic point of view. Further, a comprehensive process approach will provide better possibilities to predict and reduce the environmental effects of construction, to take sustainability aspects into account, and to foresee and reduce the need for maintenance.

Figure 1. One of the reasons why developments are needed – a common sight from a Swedish bridge construction site today.

The new technical solutions can origin from wide range of developments, e.g. in materials, design and construction methods etc. For example there is a large span of developments in concrete features that could be applicable. Both off-site and on-site construction can be emphasised in different concepts. It has to be kept in mind though, that technical improvements never can solve everything, hence they have to be governed by a comprehensive new industrial production process as well. However, the most important reason why contemporary concepts of industrial bridge construction have much better prerequisites and possibilities to succeed than their forerunners is, of course, the continuous rapid pace of development in information and communication technologies (ICT), along with an emergent understanding of how to use and benefit from this computerisation. The impact of further progress in ICT can hardly be overestimated, making it a prerequisite to successful implementation of an industrial process for bridge construction. ICT will provide the cement that ties the different pieces together and it will be increasingly employed to develop a more efficient and rational process. Moreover, it is essential to find solutions in which all participants can work in an integrated manner in order to avoid losing information on the way. One gradually more common solution to this problem is the support of a database throughout the whole process, also allowing the information to be reused in other projects. The current high technological level in general, and knowledge in construction management, also adds to the enhanced possibilities for success of contemporary industrial bridge construction concepts.

The drawback of industrial construction, as both Koskela [1] and Warszawski [2] conclude, is that it increases the complexity of the process. Koskela refers to the increased complexity in prefabrication (e.g. precast concrete concepts): a plurality of production locations (factory and site) causes longer flow and greater variability as well as need for coordinating the different stages; an increased amount of design must be done earlier due to prefabrication lead times, resulting in incomplete and changing orders; the error correction cycle is longer, and tolerances for dimensional accuracy are lower; thus the cost of increased non-value-adding activities often exceeds the benefits to be gained from prefabrication. It can be argued, however, that such processes are obviously far from being optimised to reap the full benefits from an industrial process, and must be regarded as yet another example of partially industrial concepts with unsatisfactory performance, e.g. industrialisation imposed on a traditional process. It is important, though, to address this deficiency in a deep and thorough manner, so that the advantages of the process strongly outweigh the drawbacks.

2.2 Perspectives on industrial bridge construction

Both researchers and practitioners have been tempted to commence work on realisation of the industrial ideas in construction ever since the massive success of other industries became a fact, although the early efforts as well as many other attempts in this area never became any competitive advantages. From the literature, the main features connected with industrial construction are identifiable (compare Löfgren & Gylltoft [8], especially for in-situ cast concrete) as standardisation, modularisation, prefabrication or off-site fabrication as well as on-site fabrication, pre-assembly, mechanisation, automation, and the use of different building systems; most of these features comply also for bridge construction. Many definitions connected with industrial construction can be found in the literature: for example, Gibb [4] defines off-site fabrication in the fullest sense as a change of the process emphasis for the project, from construction to manufacture and installation.

A brief look at these attributes from the public viewpoint immediately summons associations with large uniform, straight-in-line, dull prefabricated concrete structures. Moreover, from the modest use of prefabricated concrete elements in Swedish bridge construction, the experience of the past is that manufacturers often have neglected to provide an appropriate quality in their products. These are heavy burdens from the past to be rectified so that the client can see the actual customer value from current and future industrial concepts. Regarding the previously unsatisfactory performance of industrial construction, Gibb [4] concludes that industrialised building techniques have not been developed incrementally on a continual basis, but rather as a sporadic evolution, and have even been totally neglected at times; the construction industry has seldom focused on industrial methods for their own sake. Summing it up similarly, Sarja [3] states that the construction process and project management methods have undergone sparse industrialisation when compared to industrial production in other areas. According to Warszawski [2], the main reason for the relative lack of success for industrial construction is to be found in the absence of a system approach to construction and its efficient management. Koskela [1] refers to the previous unsatisfactory success of industrialisation as a result of deficient consideration of contemporary approaches (e.g. flow management and value management) in the theory of production.

Directing the discussion towards the specific area of industrial bridge construction, a rapid development in bridge construction commenced in Europe as a response to the vast reconstruction efforts following the war, see e.g. fib Bulletin No.9 [6] and Murillo [9]. Several bridge concepts such as the balanced cantilever, incremental launching, segmental bridges, span-by-span, progressive placement, and for larger spans cable-stayed bridges, were developed, and were accompanied by massive research efforts in industry, especially when imperfections were encountered. Also temporary bridges (similar in many ways to industrial concepts) developed very fast at this time. Since Sweden was spared from involvement in the war, much of this progress was by-passed, but most bridge techniques used today originate from these mid-century developments.

An interesting question is to what extent current bridge construction applies industrial ideas, and what developments have taken place in recent decades. In an international perspective industrial systems of prefabricated concrete elements are frequently used for minor and medium span bridges. Overviews of different concrete bridge concepts is frequently found in the literature, e.g. Muller [10], Rossner [11], Martinez y Cabrera [12] and Sundquist [13]. A significant development in prefabricated concrete bridges during the last fifteen years is inferred by Calavera [14]. The continually competitive concrete-steel composite bridge concepts have a high industrial degree (see e.g. Nakai [15]), especially for the steel girder parts, while the concrete deck slab is often traditionally constructed *in situ*. Much information on bridge design and construction can also be found in fib Bulletin No.9 [6]. Furthermore, an international outlook reveals several other currently used concepts of industrial bridge construction for large bridges.

The most evident trend is gigantism (especially when the medium for transportation is water), i.e. to manufacture as large and completed parts as possible – within transportation capacities – in a protected and highly industrial facility (sometimes far away from the construction site) and subsequently installing them on site. Some examples of this are the Store Baelt Western Bridge (e.g. Kjeldgaard & Fries [16]), the Öresund Bridge (e.g. Sorensson & Thorsen [17]), the Second Severn crossing (e.g. Gibb [4]) and the Öresund Tunnel (e.g. Spreng et al. [18]).

In summary, the disadvantages of the described concepts are that they can be economical only for large bridges, or they need to be repeated many times before they become profitable, or their degree of automation is too low. In many cases where the bridge market is rather limited or closed, these disadvantages become very evident. This is one of reasons why the development of new techniques and methods for bridge construction in Sweden has not been very progressive during recent decades, especially for concrete bridges. Most bridges are traditionally cast *in situ*, involving a massive use of manpower and craft-based techniques; e.g. pre-cast concrete elements are not commonly used. The overall impression is striking, when visiting a bridge construction site today, that no major changes have occurred, compare for example Figure 1. Essentially the same observations could have been made twenty or thirty years ago. Furthermore, most Swedish bridges are procured as design-and-build contracts. Despite the advantages expected from this form of procurement (compare e.g. Murillo [9]), the anticipated encouragement of developments and innovations has failed to appear, being counteracted by for example detailed prescriptions in design codes and traditional common practice. Beneficial from this form of procurement is that it mostly has provided a smooth construction stage with a minimum of claims. The conclusion is that the described situation creates substantial difficulties for new approaches to construction in gaining a foothold on the market. But reduced uncertainties due to an industrial process are likely to facilitate more appropriate methods of risk management, thus these difficulties can be accounted for. For example, competition on equal conditions could be achieved if an overall project insurance fee, including different concepts (traditional and industrial) as well as different bidders (i.e. different combinations of contractors and designers), were taken into account when evaluating the tendering phase, as argued in fib Bulletin No.9 [6].

2.3 Implications for the bridge-building process

It must be clearly stated, as been indicated above, that industrial bridge construction is not a single concept but rather a way of thinking and organising an industrial process with a very wide range of applications and technical solutions, ultimately resulting in many competing concepts of industrial construction of bridges. The advantage of concrete in aspects of industrial construction is obvious, both for the materials developments and out of design reasons as well as with regard to production matters. Concrete has an immense variability resulting in very flexible solutions, e.g. highly industrial concepts with a very diversified customisation of the products. However, to reach such benefits, it is incredibly important to adopt a comprehensive view to construction. For example, just moving the production indoors (e.g. prefabricated concrete) does not necessarily bring about improvements in efficiency; to be able to extract the excellence, all parts of the process must be designed for optimum performance. Hence the emphasis on the industrial bridge process must be to allow maximum efficiency through simplicity and repetition, with the customer as the focal point. As has been mentioned, the equation of industrial production with mass production, although still a common view and previously a fact, is no longer valid. Besides pure mass production, there is also mass customisation with a diversity of interchangeable features, as well as one-of-a-kind products being totally customised. The latter methods, suitable for bridge construction, can only be realised through a system approach to the entire process with a thorough interface management, thus physical, managerial and contractual interfaces are concerned (Gibb[4]).

The strategy to develop an industrial bridge construction process is basically to merge all appropriate contemporary means into a smoothly running entirety, since most necessary components or ideas on how to obtain them already exist. This implies interdisciplinary considerations in fields ranging from organisational and managerial issues to technological and production methodologies. It also calls for substantial benchmarking and transfer of technology, since there are many industries from which lessons can be learned. On the other hand, the development is counteracted by the short term thinking of the market. Hence, due to the prevailing state of conservatism in the construction industry, the strategy to implement the industrial process currently seems to lie in achieving a competitive concept that produces products solely at a lower cost, temporarily disregarding the potential of other added customer value created. However, a feature of the implementation strategy must be to interact with the customers. An interest from the public client about increased performance has recently been noticed and demands for enhanced value creation will thus release the full competitive benefits from the industrial bridge concepts.

2.4 Research issues

On the basis of the discussions of the driving forces, and given the description of the problems above (compare also part II of the article), some obvious questions arise. What can be done to straighten out the situation and to encourage development in bridge construction? Could an industrial process solve the problems and improve the conditions? These main questions lead to several sub-issues:

- Which are the advantages that the different participants want to gain from an improvement of the bridge-building process?
- How can these advantages be gained?
- What demands will have to be fulfilled in gaining them?
- What priorities can be established between different kinds of improvements, and how can one distinguish between ways of gaining these advantages?

The solutions to these sub-issues, which are the objective of this paper, are essential for directing the future work in order to answer the main questions. Will concrete be competitive in this perspective?

2.5 Research approach

An appropriate start would be analysis of the process, requiring some type of tool or model. In seeking such a tool, the theory of the TFV concept, visualised in Figure 2 (see Koskela [1]), was evaluated and thought to provide an interesting framework in this respect, especially if an overall approach could be taken. Thus, the emphasis was on applying the theories to the process as a whole, although the original intention of the TFV theory was rather to explain the production stages of the process. Hence, in this emergent application the TFV concept will serve as a theoretical foundation for the studies, linking their parts together. The general idea of this concept is the integration of the three persistently occurring views on manufacturing in general, into a comprehensive theory for production, since the three views complement each other. These views are:

- *Transformation* (T); i.e. the traditional view of manufacturing as transformation of input into output. Here the emphasis is to realise the transformation as efficiently as possible by decomposing the production into tasks and minimising the cost of each task.
- *Flow* (F); which means that the production is regarded as a flow through the process. The main principle is to reduce the share of non-value-adding activities (waste), leading to compressed lead time, reduced variability, simplicity, increased transparency and increased flexibility. Just-In-Time (JIT) and lean production are methods of the F type.
- *Value generation* (V); whose main principle is to improve customer value by ensuring that all requirements are captured, ensuring the flow-down of customer requirements, certifying the capability of the production system, and measuring value. This is the origin of the quality movement and other customer-oriented methods.

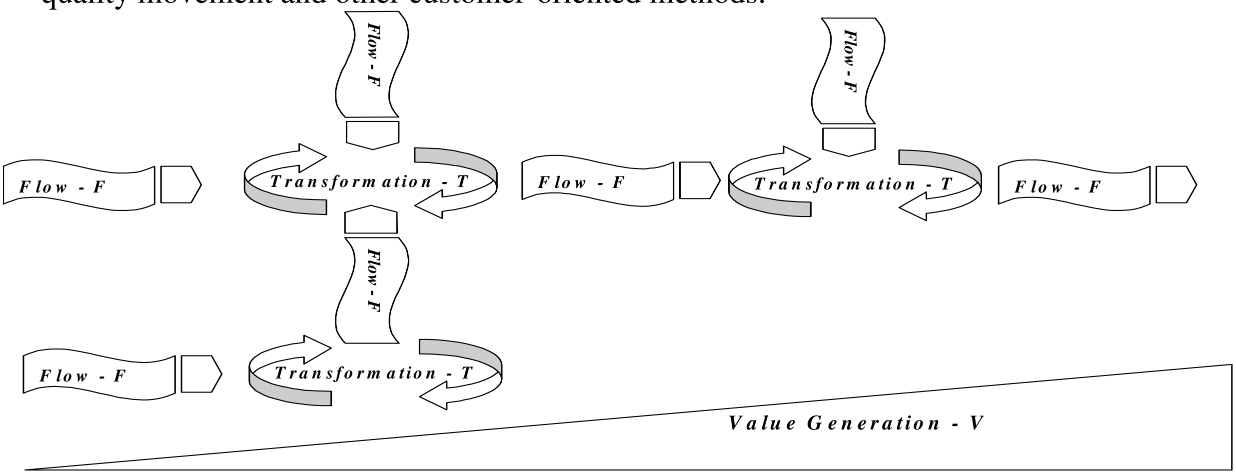

Figure 2. Generalised illustration of the TFV-scheme.

Basically, the production is looked upon as a flow with transformation parts, focusing on creating customer value through the whole process, compare Figure 2. The centre of attention in the analysis is to determine the impacts on behalf of the different views in each part of the bridge construction process. This will enable conclusions to be drawn about problems and potential needs for improvement in the process. The analysis of the bridge construction process is conducted in Chapter 3.

3. EXPLORING THE BRIDGE CONSTRUCTION PROCESS

3.1 Characteristics of the process

Compared to many other industries the general construction process shows a substantially higher degree of complexity, the large number of participants being one reason for its fragmentation. Another major problem is the common division between design and construction, which confuses customer relationships; moreover, the peculiarities of the construction business add to the complexity (compare also part II of this article). Products of the construction industry are stationary with an anticipated long service life, whereas the production facilities are movable, i.e. totally contradictory conditions compared to most other industries. When compared to the construction process in general (for buildings etc.), the infrastructure construction process is at once simpler and more complicated. It is simpler in the sense that fewer participants are involved (different trades, subcontractors etc.), but more complicated since infrastructure investments almost exclusively come from governmental funding, as been discussed. The latter leads to an even more complex customer relationship with a formal customer (e.g. the national rail or road administration) representing a somewhat diffuse real customer, e.g. the end-user, as can be seen in the process map of bridge construction in Figure 3. Furthermore, between the two are the political establishment and the parliament, transferring the needs from the real customer. This extensive involvement of society outweighs by far the increased simplicity due to fewer participants, thus adding more to the aforementioned complexity.

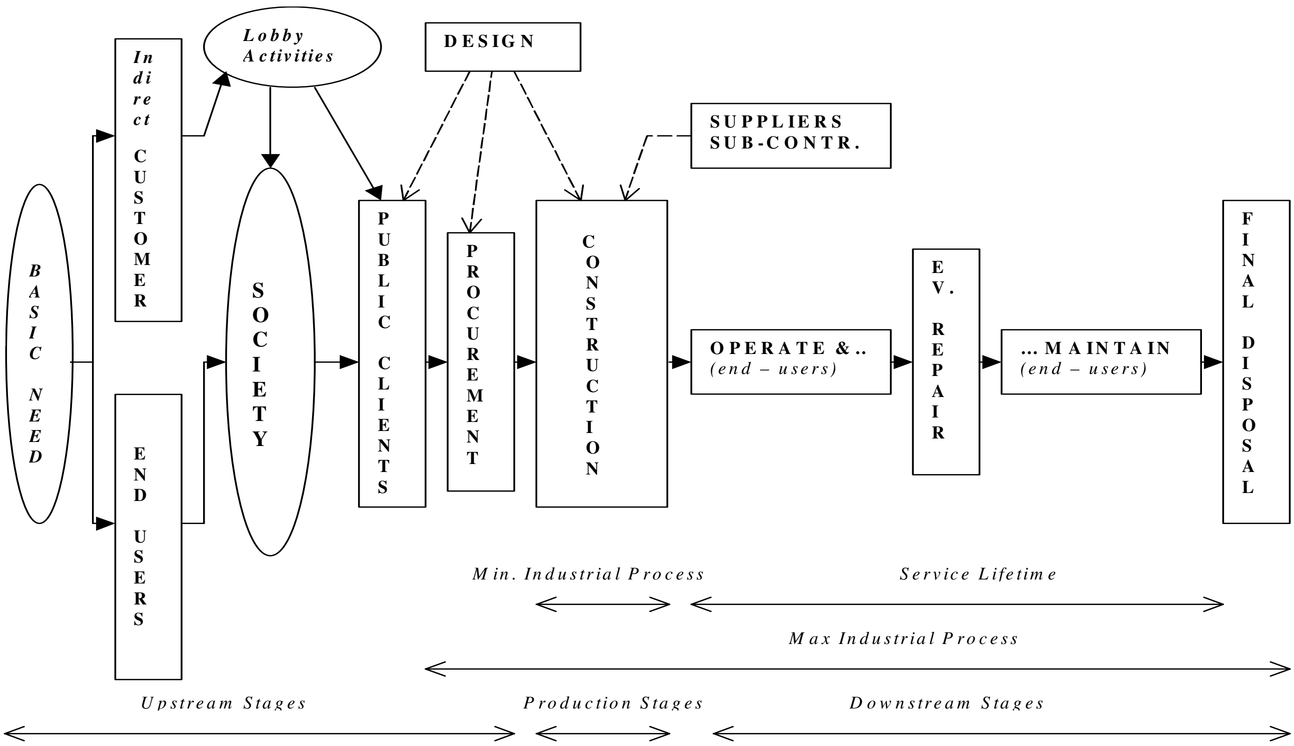

Figure 3. The conventional bridge construction process.

In addition, detailed prescriptions from the authorities put restrains on the process. In the following, 'process' will mean the whole current traditional construction process, while 'industrial process' or 'new process' stands for the suggested improved aspects of the overall process as described below.

3.2 Assessment of the entire bridge construction process on the basis of the TFV theory

As a basis for the analysis, a map of the entire bridge construction process is shown in Figure 3. This diagram was derived from empirical and individual experience, but guidance in the scope of international bridge construction processes can be found in fib Bulletin No. 9 [6]. In the analysis, each part of the process is dealt with separately and comments are presented occasionally. Details from the analysis are presented in the Appendix.

When analysing the process of bridge construction from a TFV viewpoint, it becomes very clear that the emphasis for the three different views changes as the process continues. To generalise, it seems that the changes in content for the views are correlated and occur at the same position in the process scheme, as indicated in Figure 3. This change of scope is most obvious for the view of value generation (V). In the first upstream part, the focus of this parameter is solely on gradual clarification of the needs and demands of the customer; no actual creation of value can be found in these stages. On the other hand, the next production parts, design and construction, are very closely connected with value creation. In the last downstream stages, the focus changes again, now towards checking on the fulfilment of previous demands. The same phases for the flow (F) view consist of flow of information, real flow (e.g. products, material, workforce, documents, etc.) and flow of services. A similar comparison for the transformation (T) view would imply no transformation in the upstream parts, significant transformation taking place in design and construction, and a very small degree of transformation in the downstream parts (e.g. to postpone the degradation of the structure). It is not surprising though, to find that the emphasis of the TFV theory is somewhat altered when applied to the process as a whole. Out of necessity, the following analysis must be kept at a rather superficial level; thus a further breakdown in several steps is possible, but out of place in this context. In the following some comments on the different stages are given and details of the analysis regarding the TFV theory can be found in the Appendix.

<u>Upstream stages:</u> *(also compare Appendix, Table A1)*

End-users/customers: The real customer (in contrast to the formal customer, i.e. the rail or road administration) can be divided into end-users or direct customers, i.e. the public traffic, the drivers, etc., and indirect customers, i.e. the business community, the social community (local governments, the union, etc.) and professional organisations or interest groups. The direct customer influences the process primarily through democratic elections, while the indirect customers mostly are active in considerable lobbying directed to the political establishment. Some organisations even undertake investigations to promote their own interests.

Society: By society in this context is meant the government, the parliament and other political instances as well as governmental departments and public authorities other than road and rail administrations (which are treated separately in their role as public commissioners).

Road or Rail Administration: The society grants funding to the road or rail administration. It is usually to some extent up to the administration to decide on how to divide the funding.

Procurement: Disregarding the possibility of accounting extra rewards for other parameters than the price, in reality the currently most common form of procurement is awarding the bidder with the lowest price the contract on a fixed-price basis, also for design-and-build contracts. These contractual conditions are routinely transferred from the contractor through the whole value chain. Although unusual, there has been some experimentation with other forms, e.g. functional contracts, and contracts with different forms of involvement in financing, operation, maintenance, etc. This is probably something that can be expected to increase significantly in the near future.

A development in terms of procurement is very important and beneficial for the overall process. Interesting experiences and opinions on different procurement forms in an international perspective can be found in fib Bulletin No. 9 [6]. As been mentioned, it is inferred that a system with a project-global insurance including both design and construction and paid for by the commissioner or included in the tendering offer, would be self-regulatory in terms of applying appropriate risks. Both at company and project level, with due regard to qualifications, experience etc. and the risk for each specific project, as well as between participants (e.g. avoid putting a high risk on designer whose fee only represent a few percent of the construction cost), since the insurance fee will be adjusted in accordance not only to the specific project risks, but also according to the performance of the different combinations of contractors-designers-commissioners. This system would provide great opportunities to implement new industrial bridge concepts.

Production stages: *(also compare Appendix, Table A2)*

Design: The design stage is usually divided into three phases, namely conceptual design, preliminary design and detailed design, each phase performed in different stages in the bridge building process. In turn, detailed design can be divided into general design (or basic design) and final design (or execution design), although this division is not often used in Sweden. A consultant contracted by the commissioner sometimes does the final design, but more often the design falls under the responsibility of the contractor, since most small and medium-size bridges are procured as design-and-build contracts, as been discussed. Hence, many conceptual considerations due to alternative solutions originate from the tendering phase. This fragmentation in design phases is unfortunate but currently necessary to keep pace with the overall process, especially in the first two phases where for example the need for coordination with the road design is sometimes overlooked. However, the isolation from other parts in the process is not beneficial and often leads to confusion in the customer perspective and to the absence of a comprehensive view. In fact, the design stage can often be compared to a black box. One possible solution is to use other forms of procurement where the contractor comes in at early stages, thus taking part directly in the conceptual discussions or to compete with different bridge concepts.

Construction: Since construction falls within the original application of the TFV theory it is apparently considered in depth by Koskela [1], but a brief summary and some complementary comments regarding bridge construction can be found in the Appendix.

Downstream stages: *(also compare Appendix, Table A3)*

Operation and maintenance: The longest stage in the process is closely related to compliance with previously stated requirements, and thus emphasises on minimising maintenance. Recently it has been highlighted as a very important part of the process (see e.g. Silfwerbrand & Sundquist [19] about the issue of operation and maintenance). In this stage the final user can see the results of the process and it is also the part that governs the overall cost of the project, e.g. lifetime cost, environmental effects etc. Mistakes made in earlier stages often come as surprises in the form of costly reparations.

Repair or strengthening: If this stage appears, it is usually the consequence of previous mistakes or increased live load due to heavier traffic. This part of the process is usually not foreseen, or precautions would have been taken to avoid it or prepare for it. This stage can more easily be accounted for in industrial concepts.

Decommissioning and final disposal: Flexibility in disposal is an important added customer value, i.e. different ways of reusing the remains, recycling or even possibilities to rebuild the structure or parts of it elsewhere.

3.3 The industrial bridge construction process

In this chapter some possible effects and viewpoints on an idealised industrial process in a TFV perspective are summarised.

Upstream parts: There are different degrees of industrial implementation which, in the broadest sense with other forms of procurement, could constitute an involvement in the domains of the public commissioners, as shown in Figure 3. In this case the most significant gain is a major reduction of waste (i.e. non-value-adding activities such as uncertainties due to lack of clear requirements, excessive tendering phase, etc.); therefore, the improvement is mostly in the F vein. It is of course also easier to influence the transformation that is taking place, resulting in increased efficiency in the T aspect as well. All this adds up to increased customer value in the V aspect.

Procurement: As noted earlier, it is very important to develop new contractual forms to attain a process that motivates all participants in reaching an overall optimisation of the project. It is also crucial that each participant has a clear and attainable goal, i.e. being able to produce a profit. The aim is to realise both factors simultaneously and thus reach goal congruence, which is quite essential for an industrial process. As mentioned, the degree of implementation is closely connected with the form of procurement. Procurement forms allowing an early entrance of the contractor (as the process leader) will evidently contribute to the success of industrial bridge concepts as would other procurement forms e.g. the method with project-global insurance fees, as mentioned. The obstacle of the current procurement could also be overcome, but at the price of less customer value creation and less competitive concepts. Moreover, including downstream stages in the contract increases the extent of the industrial process, although this should not affect the basic emphasis of quality for the final product, they should remain the same.

With regard to the TFV theory, the same conclusions as for the traditional process apply. For example, the possibilities of reducing waste due to multiple tendering work through other forms of procurement fall very well in line with an industrial process.

Design: An industrial process in the field of bridge construction will undoubtedly mean a different way of working in the design stage, leading to product development instead of the present unique project-based design. From this will follow a new role for the designer, upgrading him to become one of the most important persons for the success of each specific industrial concept in the long run. This is also likely to erase the borderlines between contractors and design consultants, integrating them into a process-oriented team. As mentioned, in an industrial process an integrated design is a necessity: the more of the three design phases that can be incorporated in the process, the better. It is compulsory that the detailed design phase is included, and the use of some contemporary design techniques, e.g. concurrent design, seems beneficial compared to the traditional sequential design; compare e.g.

Koskela & Ballard & Tanhuanpää [20]. Depending on the rate of industrial employment, the conceptual design and the preliminary design phases could also be integrated with other forms of procurement, thus resolving the unfortunate fragmentation of the design stage and enforcing the stress upon a comprehensive view. This would also fill the great need for construction versus design coordination, currently overlooked in many cases. Production parameters, such as workability etc., can then continuously be taken into account; hence structures from the drawing table ('desktop products'), which are simply not possible to construct, would become a thing of the past. Also the problems in coordinating the many different design disciplines could be solved, although this problem is more accentuated when design of buildings is concerned. In conclusion, design management would become easier.

Most bridge designers act as consultants, but usually the major contractors also have design departments of their own, even though they cannot be said to be an integrated part of the production currently but mostly act like in-house consultants. Thus the stakeholder viewpoint differs somewhat between the two more on the organisational level than otherwise, but this will become even more obvious since the contemporary tendency towards erased borders between contractors and designers seems to be accentuated, as mentioned. Hence, new organisational cooperation will emerge, e.g. teamwork. Some consultants may look upon these changes as threats, but they can just as well be a great opportunity, since nothing dictates who is to own the process – it could equally be a large consultant that is the driving force behind the alteration. Furthermore, other industries employing similar system have extensive amounts of consultants in their business.

From the standpoint of the V view, the enhancement in customer value would be tremendous, chiefly because of continuous product development but also due to significant improvements in the other veins. Regarding the concept of F, integration of design leads to considerable reducetion in waste; e.g. the majority of construction peculiarities can be eliminated or accounted for. This is mainly due to better possibilities to control the design and the whole process, so that the uncertainties are significantly reduced, especially if all design phases can be included. In terms of the T view, an integrated design would lead to enormous opportunities for creating more efficient transformation tasks. Again, the influence of further developments in ICT and computerised design cannot be overestimated in this aspect, with regard to both calculations/analysis and drawings (e.g. integrating the two into one transformation task and taking other parameters into account, model based design etc.) as well as communication with other participants, e.g. coordination with road design in early stages. ICT is also necessary to facilitate the fullest sense of computer integrated construction (CIC), thus possibilities to build the bridge on forehand in the computer will further enhance the waste reduction.

Construction*:* Naturally, the great bulk of the industrial process will fall under the construction stage, since the physical completion of the project takes place here. Again, the huge anticipated impact of further evolution in the information and communication techno-logies (ICT) will be substantial and beneficial. Generally, the same conclusions as for the traditional process apply with regard to the TFV theory. However, for an industrial process there must be a major shift towards a more thorough consideration of the V and F principles, although proper attention still has to be paid to the T view. The increased customer value reached by the combination of reducing waste and improving efficiency in the transformation tasks enables the overall emphasis of the TFV concept to fit extraordinarily well for an industrial process.

On the other hand, an industrial process inevitably leads to increased complexity, especially in the F view, as mentioned. This contradiction has to be dealt with, one solution possibly being further enhancement in the use of ICT throughout the process to reduce the effects of the enlarged complexity to an acceptable level. Even if the complexity is increased, the flows in the pro-cess are known with a minimum of uncertainties, which can thus be acted upon in a continuous process-improvement cycle. Hence the advantages of the process will by far overweigh the drawbacks.

Downstream stages: The downstream parts are not likely to be included in the industrial process initially, even though procurement forms stipulating this do exist. On the other hand, the industrial concept could include additional enhancement for these aspects (e.g. considering the life cycle cost, LCC, etc.); thus the procurement will have to take this into account. Otherwise, the same conclusions as for the traditional process apply with regard to the TFV theory. Further, efforts in after-sales activities (e.g. longer guaranties, inspections, maintenance, etc.) will presumably increase as the competition eventually shifts towards delivering highest customer value.

3.4 Demands and advantages extracted from the process in view of different stakeholders

It is obvious that a certain number of demands must be raised, but also that the participants can claim many additional benefits from the new process. However, a distinction must be made concerning the driving parties of change (i.e. primarily public commissioners, contractors or even consultants) who expect to reap some predetermined advantages, and those taking part indirectly in the changes (i.e. the rest of the participants) who will attain their advantages as a consequence of the new process. Furthermore, the difference between advantages and demands is not always clear-cut; hence fulfilment of the demands sometimes also entails an advantage, and therefore it must be stressed that the advantages inferred here are complementary to the specified demands. The demands and advantages, categorised according to the original stakeholders, are presented in the Appendix, Table A4 and Table A5. They were derived from the process analysis and from the literature.

3.5 Performance specification

The requests from the different stakeholders stated in the foregoing, can be condensed into a performance specification for an industrial bridge-building process as presented in Table 1.

Table 1. Performance specification for concepts of Industrial Bridge Construction.

Basic demands:
Demands that can be stated for any construction project.
- Functionality, safety, serviceability and comfort criteria.
- Economic and efficient construction, and delivery within time schedule.
- Quality and durability aspects, lifetime assessment, maintenance reduction and environmental concern.
Advantages causing added value:
Advantages (or bonus) on behalf of an industrial process.
- Reducing overall construction time and cost.
- Flexible process, both during construction and afterwards and no disturbance due to construction work.
- Optimisation of design and an enjoyable experience of the aesthetics.
- “Extras” creating goodwill, e.g. compression of time schedules, easier procurement and less
- administration.

- Improved quality aiming at zero mistakes and consequently no guarantee work.
- Continuous development and improvement of products and productivity.

Complementary demands for an industrial process:
Besides the first general demands, the other examples depend to a great extent on the features of each specific concept; thus there is no general applicability.
- Reduction or elimination of wastes, uncertainties and peculiarities.
- Simple and rational construction or manufacturing, resulting in lowered construction cost.
- Easy, fast and straightforward design, integrated in the process and simplicity in aesthetic adjustments.
- Improved working conditions for employees.
- Simple and minimised work on site and all work carried out in sequence, thus avoiding later rework.
- Simple and light elements to minimise transportation, heavy lifts etc. and to provide efficient assembly.
- Open systems with standardised interfaces of components.
- For prefabricated systems, a high degree of prefabrication is aimed at, preferably with no on-site work except erection and assembly.

3.6 Framework for an industrial process

To be able to implement a new industrial process, some underlying model must be applied. The process analysis performed calls for an outline of such a model for an industrial bridge-building process, and some suggestions in this direction are presented here. The conclusions are that a framework for a process model can be derived from the analysis, also serving as a framework for the research, and it is presented in Figure 4.

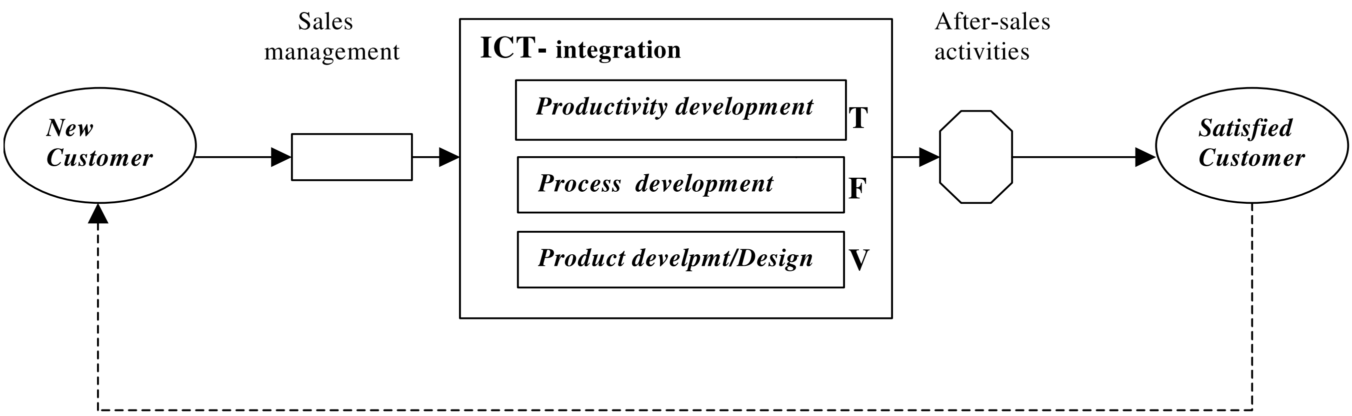

Figure 4. A model of an industrial process for bridge construction out of a managerial perspective.

From a managerial perspective the new process is looked upon as three parallel streams with regard to the TFV theory: productivity development with main emphasis on the T view, process development with main emphasis on the F view and product development/design with main emphasis on the V view, with integrated linkage between them, thus naturally, each stream will take all views into account. Sales management could for example include promotion of company specific bridge concept with brand image etc., and after-sales activities refers to creating enhanced customer value, for example through offering different additional services, such as operation and maintenance, longer guarantees or inspections, etc. It is essential to adopt a clear-cut customer/seller perspective in the new process, especially if a 'technology-push' approach seems to be necessary, as been mentioned. However, to attain a truly industrial process there is also need for another package integrating the parts and, as mentioned, this is the powerful support from ICT throughout the whole process.

It has to be kept in mind, though, that the implementation of a new process like the proposed one for industrial bridge construction is a managerial question. Without devoted support and strategic commitment from the management, no changes will occur.

3.7 Conclusions

As a summary of part one of this article, one can conclude that developments in bridge engineering is lacking behind and that there is an urgent need to promote research aiming at progress in bridge construction. In perspective of the current process for building bridges, it is evident that a development towards a new industrial process seems beneficial for all parties involved. However, in order to be able to achieve such an evolution there are many factors, research fields and research topics involved which have to be dealt with and combined in an interactive way. These questions are dealt with in the second part of this article, **'**Explorations of different means to achieve an industrial process for building bridges; part II – evaluation of contemporary strategies', were also some conclusion about the continued work are drawn.

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APPENDIX: ANALYSIS OF THE BRIDGE CONSTRUCTION PROCESS ON THE BASIS OF THE *TFV* THEORY

The Appendix contains five tables (table A1 – A5) with detailed results from the analysis conducted on the basis of the TFV theory. The tables are referred to in chapter 3.2 and chapter 3.4.

Table A1. Upstream parts of the current process in the three TFV views, compare chapter 3.2.

Basic needs of infrastructure:

V: The actual source of the V view.

F: The actual source of the F view.

T: No tangible transformations are done here.

End-users:

V: The needs and demands are further refined in this stage.

F: A large flow of information and argument takes place, mostly informal.

T: No tangible transformations are done here.

Society:

V: It falls upon society to make necessary priorities regarding the funding, thus deciding to what extent the communicated needs can be realised.

F: In the F vein there is still a large flow of information.

T: No tangible transformations are done here.

Road or Rail Administration:

V: There is still no value generation taking place, but the demands become even more detailed as they stream through this stage and funding for physical projects is crystallised.

F: There is a large internal and external flow in huge administrations, mainly information and communication with other authorities or within the organisation. Thus, great amounts of waste can be found, e.g. in pure administration such as handling of documents, checking, controlling, remittances, etc., but the main waste is lost time. Naturally, to some extent this cannot be avoided.

T: The first transformation can be recognised, the planning which is the first phase for a physical project. However, planning in early stages is closely connected with overall investigations, prognoses, financing, etc.; thus there is a lot of input to be dealt with. The output should be a conceptualisation of the project, also regarding some initial design considerations. Hence, the first connection to the early design stage, the conceptual design, can be found here, although bridges usually are very superficially treated at this stage.

Procurement:

V: Procurement is more a transfer of customer demands than an actual generation of customer value. However, the contractual condition under which the work is carried out has proved to be one of the most important factors, with a major impact on the final result and the motivation of each contracted part. Thus, it implicitly plays a significant role for the customer satisfaction as well. Nonetheless, it can also have explicit influence in special cases where an alternative solution is obtained in the tendering phase.

F: This stage consists of non-value-adding activities, the prime waste being in the large amount of contractors performing the same tendering work. Yet one obvious way of reducing this is another form of procurement where, for example, contractors can offer their products and concepts. The common case of design-build contracts often leads to sub-optimisations due to the design consultant being procured at a fixed price by the contractor with guarantees concerning the bill of masses from the tendering, thus often resulting in design with slim structures and low workability. Hence, procurement means a lot to the efficiency of the process.

T: No actual transformation takes place, unless regarding the above-mentioned case of a better alternative solution coming to the surface.

Table A2. Design and construction of current process in the three TFV views, see chapter 3.2.

Design:

V: The design stage is essential from the value generation viewpoint, V, since it is when streaming through this stage that the somewhat widespread customer needs become concretised into physical descriptions and documents of the final requirements. In other words, quantitative customer needs are turned into qualitative customer requirements. In some respects thought, detailed prescriptions from authorities counteract the value creation. It is a well-known fact that a large part of the mistakes encountered downstream in the process can be related to the design stage; thus the quality assignment must be very strong here.

F: The massive flow of information and related waste, such as uncertainties due to lack of information, wrong information etc., and control or verification has been thoroughly treated by other authors, e.g. Koskela [1]; especially the authority control of design documents is tedious in the present process. The lack of transparency is evident, particularly in the detailed design phase.

T: The transformation taking place is primarily of information into design documents. Thus, one obvious way of creating a more efficient design is further enhancement in the use of ICT throughout the design phases, for communications, documents, drawings, calculations, approval, etc. Linking of all parties can be achieved e.g. by using a project database or a product model. It is also very important to acquire and use the best knowledge as a resource in the transformation; hence knowledge management is a vital issue.

Construction:

V: This is where customer value is realised through fulfilling the wishes and requirements of the client. This is also the stage where value easily can be lost, e.g. if the importance of proper planning is not considered, disregarding the fact that systematic planning and quality go hand in hand with creation of customer value.

F: Numerous internal flows can be found, e.g. building material and products, workforce, logistics, resources (tools, machines, etc.), subcontractors and suppliers, etc. Consequently, there is also a lot of waste to be found, both physical and intangible, e.g. material waste, coordination problems, etc. As for the overall idea of continuous compression of lead time to reduce waste, it has to be borne in mind that this time can only be decreased to a certain limit without the input of a new and more efficient approach (e.g. the use of new designs or new construction methods); otherwise the quality of the final product will clearly be put at risk.

T: Multiple transformation tasks are taking place during this stage. In brief, input in the form of material, components and workforce are transformed into the finished product; thus, this stage has traditionally been very much a matter of reducing the costs in these transformation tasks. Among contractors who currently base their business primarily upon on-site concrete production, there is sometimes a tendency to exaggerate the use of *in-situ* cast concrete based on short term decisions, instead of the most efficient combination of material use, in order to keep as much as possible of the value chain within the firms.

Table A3. Downstream stages of current process in the three TFV views, compare chapter 3.2.

Operation and maintenance:

V: This stage is a matter of living up to what has been promised. There is no actual creation of value, but the significant stress is on fulfilling previous commitments, even though the formal guarantee of the contractor currently seldom reaches more than a few percent of the construction lifetime.

F: A certain amount of maintenance activities occurs, but these are likely to be predetermined and a consequence of earlier decisions.

T: An input in the form of maintenance results in achieving the predetermined lifetime.

Repair or strengthening:

TFV: Basically the same considerations as for construction above.

Decommissioning and final disposal:

TFV: Most of the considerations about the Operation and Maintenance stage, with some general modifications, are also applicable here.

*Table A4. Demands and advantages for the commissioner and the contractor, compare chapter 3.4. General demands (**D**) for any process, advantages (**A**) from an industrial process and demands specific to an industrial process (**DI**).*

Public authority commissioners:

D: The most important demands transferred from the users can be recognised as (compare Table 5):

- Functionality of the structure, safety against failure, and serviceability demands, e.g. deflection, settlements, cracking, etc., and comfort criteria such as avoidance of expansion joints and demands of continuity.
- Safety for users, i.e. traffic environment and traffic safety.
- Minimal disturbances to traffic during construction.
- Adaptability to local conditions and minimum impact on local environment including noise reduction,
aesthetic compatibility, etc., and environmental aspects, including final disposal or recycling, etc.

D: Demands with a bearing on the role of administrator and maintainer are:

- Low cost on a lifetime basis, low need of maintenance and high durability.
- Easy access for planned work, easy control assessments and easy operation.

A: The advantages that public authority commissioners could reap from the new process are:

- Better quality, less repair and maintenance, as well as lower lifetime cost.
- Better possibilities for eventual future strengthening or even reuse of the bridge.
- Easier and more cost-efficient control of design and construction.

Both the commissioners and the contractor:

D: Interrelated demands that cannot be separated from both the commissioners' and the contractor's viewpoint:

- Low production cost and an efficient construction stage and delivery within time schedule.
- Quality aspects and practical environmental considerations in construction work.

A: Advantages from the new process derived from both the commissioners' and the contractor's viewpoint:

- Improvements in construction efficiency resulting in lower building costs and shorter construction time.
- Higher quality with no need of guarantee repairs.
- Better working conditions leading to improved employee performance.
- Simpler forms of procurement, reduced administration, short and predictable design stage.
- Reasons to anticipate a continuous product and productivity development.
- Closer cooperation and a process with fewer uncertainties (e.g. design, cost, time, etc.)
- Flexibility during construction and during service life.

Contractors:

D: Demands specifically related to the contractor could be specified as:

- Competitive product with high return on investments (ROI).
- Low sensitivity to unpredictable circumstances (e.g. weather conditions).
- Possibilities to protect intellectual property , i.e. the concept.
- Possibilities to achieve reliable information about future demand for new bridges.

DI: Some additional demands with regard to an industrial process are:

- A product possible to produce in a serial process, and thus minimising the impact due to advantages of scale, i.e. reducing the dependency on large volumes to carry the necessary investments.
- Trustworthy and optimised design integrated in the industrial process, and easy, predictable work on site.
- Extension of the value chains into a value network, thus empowering all participants.

A: The main advantages from an industrial process for the contractor's scope of interest are:

- Great possibilities to direct and develop the new process, thus creating good opportunities to produce a
reasonable profit and a product that yields goodwill and hence creates a positive image.

- Improved employee performance, due to better working conditions.
- Better opportunities to take advantage of personal knowledge and ideas through empowerment.
- Competitive advantages, and increasing market shares on a larger and more stable market, e.g. the global market on the basis of patents.
- Possibilities to create niches with even higher ROI from ventures in R&D, especially in areas protected by patents.
- Possibilities for an efficient value network management.
- Possibilities to apply the new process in other construction areas, e.g. road construction and buildings.

*Table A5. Demands and advantages for the other participants, compare chapter 3.4; (for (**D**), (**A**) and (**DI**) compare Table A4).*

All participants:

A: The main overall advantage with regard to all users is the anticipated economic benefit that can be derived from a more efficient industrial process. This is an evident increase in customer value. Beneficial to all participants will also be better possibilities to foresee and reduce the environmental effects of the construction, enhanced sustainability in consequence of increased quality, and a significant increase in attractiveness to interest educated young people in joining the construction business.

End-users (direct customers):

D: Demands from the end-users' viewpoint are mainly concerned with functionality, safety and comfort, i.e. mostly design problems normally dealt with in design codes and standards. Another great concern, perhaps of more importance in day-to-day life, is the issue of minimising or avoiding disturbances to traffic, e.g. due to maintenance, rehabilitation and the like. Also the problem of small deficiencies in performance (such as a bump when passing the expansion joint), and the issue of aesthetics, attracts a lot of attention.

Professional organisations and interest groups (indirect customers):

D/A: Organisations generally pay attention to the functionality of the total system, thus applying the same demands on the bridges as a part of the system. The advantages that they can rely upon from the new process are more acceptance of their needs and use of the new process as an argument in their lobbying activities.

Society:

D: The demands of society basically conform to the political constraints. It is very difficult to generalise these demands since they are a product of the current opinion and political majority; thus they can shift very rapidly at times. Typically, the demands of society reflect a situation balancing the need of improved communications with environmental consciousness and economic limitations. An investment in infrastructure has to be profitable in relation to the society's dimensions.

A: The advantages that society can achieve from an industrial process are better and more efficient communications that give spin-off effects, since it is a dynamic source of activities leading to developments in society and benefits for the economy (e.g. increased competitiveness for industry, growth in enterprises, decreased unemployment and increased tax income); thus it will become easier to raise funding and to find alternative solutions for funding. Moreover, better communications are a solution to some of the problems caused by increased urbanisation. Other benefits could include greater possibilities of counteracting urbanisation and a better overall use of resources from a public economic point of view.

Designers:

D: Some general demands to be raised by the designer are:

- Visible and comprehensive methods for design management and a better coordination of the activities.
- Reducing uncertainty by achieving the right information in the right time.
- Contractual incentives enabling reasonable payment for efficient solutions that lead to lower construction costs, rather than acting on a fixed lump sum price; and to extract feedback from the production.

A: The advantages for designers from the new process are mainly a different and a more stimulating way of working in teams, thus more like other industries, which will enhance the competition for competent personnel between industries. On the other hand, the new process will attract many young people into construction. Other benefits from the new process are working as a product developer close to the production, good possibilities of continual personal life-long competence development, and possibilities of enlarging the design task to include the production process, erection, assembly, etc.

Suppliers and subcontractors:

DI: Suppliers and subcontractors represent a significant part of the value chains, although they sometimes have difficulties in influencing the process since they are considerably fragmented. Thus, their demands primarily stemming from the industrial process can be perceived as:

- Cooperation on a long-term basis, thus able to yield a reasonable profit.
- R&D commitments in joint ventures; and planning facilities that allows one to keep ahead of time.
- Participation in networks with effective value network management.

A: The anticipated advantages on behalf of the suppliers and subcontractors are better possibilities for future planning due to long-term cooperation possibly as a result of partnership, and thus possibilities of taking part in the development of products and processes. Moreover, new business opportunities are likely to occur due to outsourcing, mainly from contractors.

Explorations of different means to achieve an industrial process for building bridges; part II – evaluation of contemporary strategies

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ABSTRACT

In the ongoing multidisciplinary research at Chalmers University of Technology, different means of creating an industrial process for building bridges have been investigated, since there is an urgent need of development in bridge construction. Out of the basis of structural design, the emphasis in this paper is to establish the foundations for achieving such a new process, basically through analysis of the current bridge construction process, as well as to determine the required improvements and provide a framework for a process model and for the future research. The main focus of the continued work is on investigating techniques and materials developments from a design viewpoint, in addition to include studies of construction characteristics such as production methods, etc. Hence both product and process development are emphasised.

Key words: concrete bridges, concrete concepts, industrial, process, design and construction, materials development.

1. INTRODUCTION – PART II

This article has been divided into two parts were the first part, 'Explorations of different means to achieve an industrial process for building bridges; part I – implications out of the current process', was devoted to comments and analysis of the current bridge construction process in comparison with developments in direction of a new industrial process. A general introduction to the research topic can also be found in part I. In this second part, the emphasis is turned towards problems and opportunities in an industrial context, were the underlying driving forces acting in favour of an industrial process are investigated and the necessary prerequisites are evaluated. In addition, some priorities and conclusions about the continued work are summarised.

Some of the interacting components that can be connected to Industrial Bridge Construction are illustrated in Figure 1.

2. PERSPECTIVES ON OPPORTUNITIES AND PROBLEMS IN AN INDUSTRIAL CONTEXT

2.1 Underlying driving forces in favour of an industrial process

In light of the problems surveyed in the first part of this paper and below (see Chapter 2.2), it is important to apply a comprehensive view in discussing solutions. From a global perspective, it is a widespread opinion that the construction industry faces major changes, as is also true for the whole of society.

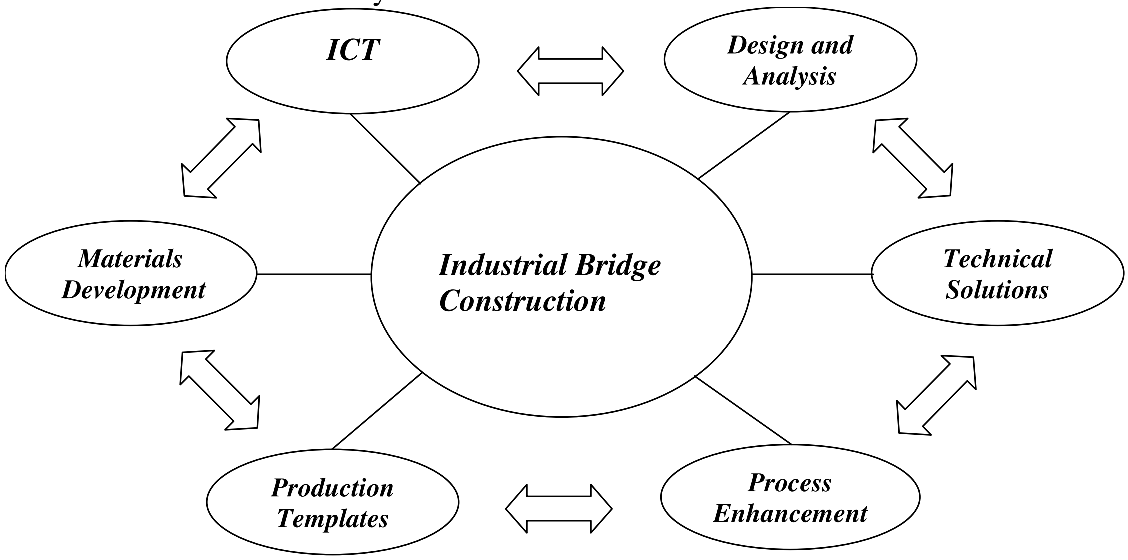

Figure 1. Some of the interacting components connected to Industrial Bridge Construction.

Flanagan et al. [1] describe the overall driver of change as social, technological, economic, environmental and political (STEEP). Demographic changes are one of the most powerful factors affecting worldwide development in a broad sense. The chief features of these changes are exceedingly high population growth in developing countries and declining growth and an elderly population in developed countries, as well as continued urbanisation. Hence, the applicable solutions will differ significantly between regions, since for example the educational level, the wage levels and the general degree of industrialisation strongly influences the development. This is somewhat illuminated by Sarja [2], who argues that it is small, mainly European countries that lead the industrialisation process of construction worldwide; in larger countries under the influence of strong building traditions, the level of industrialisation is lower than would be allowed by the general level of knowledge and industrialisation of the society

Effects of the global economy will impact significantly with its fast rate of change and many fresh opportunities of doing business (see e.g. Otley [3] for a summary of these impacts). Globalisation and the expansion of markets, causing companies to strive for world class to survive, will not spare the construction business, resulting in fewer and larger companies and many small service providers. Furthermore, a great increase in both cooperation and frenzied competition resulting in 'co-petition' will enhance the formation of strategic alliances (also with companies from outside the construction industry) and partnership with suppliers, as well as to increased specialisation. Initially, this progress will be enhanced by the need of companies to ultimately improve their revenues in order to increase their capitalisation and become attractive for investors on the market. But as the process continues, there will probably be a shift in emphasis of the driving force towards a stakeholder perspective, since other incentives than purely economic variables – such as lack of labour and attractiveness to employees (losing manual skills because workers cannot see a future in working under harsh and unsafe conditions, as well as for demographic reasons of ageing) or public confidence as a response to opinions in society – will also play a very important role, requiring companies to act accordingly.

The effects on the organisational level will also be extensive, and the managerial approach will have to comprehend a variety of issues throughout the whole organisation. There will be a specialisation on behalf of company undertakings as well as within organisations; thus integrated multidisciplinary teams, complemented by outsourced specialised services, will replace functional departments in companies. Managerial efforts must be devoted to logistics, supply chain and network management to facilitate partnership commitments. Since knowledge in the whole organisation as well as in individuals will be a major factor for competitiveness, knowledge management will be essential; hence, continuous learning on both individual and organisational levels will be vital. Otley [3] remarks that this is especially important for '*the control of knowledge-based workers where the key resource is time and the key outputs include innovation and responsiveness to customer demands*', e.g. service providers or consultants. Furthermore, reduced uncertainties due to new processes will facilitate more appropriate methods of risk management. Thus, the overall emphasis is likely to shift towards fulfilling customer needs and competing by providing added value rather than on lowest building cost solely; to seek demanding customers driving the development can become a competitive advantage. This indicates large benefits from a strong commitment to innovation throughout the organisation.

In this context a comprehensive theory for continuous strategic renewal is of vital importance for seizing the right opportunities of doing business in the global economy. The framework presented by Simons [4] is interesting in managerial respects. Simons stresses the importance of managerial support and argues in favour of maximising return-on-management (ROM) as a substitute or complement to return-on-investments (ROI) in order to enhance the outputs of value. He concludes that the process is a balancing act between the dynamics of creating value, the dynamics of strategy-making and the dynamics of human behaviour, each of them causing organisational tensions which must be coordinated to allow effective implementation and control of business strategy. Anthony & Govindarajan [5] also provide some interesting views on management control, especially stating that the central role of management control systems is to achieve goal congruence, defined thus: '*Goal congruence in a process means that actions it leads people to take in accordance with their perceived self-interest are also in the best interest of the organisation*'. Another aspect of the problem of goal congruence is notably present in many current forms of procurement in construction, namely a lack of goal congruence between the different participants. Hence, changing 'people' for 'participants' and 'organisation' for 'project' makes the definition still valid. Otley [3] puts the development of organisations into perspective: '*But although survival has evolutionary connotations, and the idea of the 'survival of the fittest' is much mooted in the rhetoric of market-based economies, we have to move beyond biological ideas of evolution. The problem with the evolutionary model of biology lies in the prolific inefficiency of nature and the relatively slow pace of random evolutionary adaptation. Thus we need to move towards the idea of evolution by design, by designing organisations that are capable of redesigning themselves.*' Moreover, Otley points to a crucial condition concerning all innovations and developments, which is particularly present in the construction industry: '*A further area lies in the encouragement of innovation and appropriate risk-taking. Most performance measurement systems encourage conservatism and 'playing it safe'. Managers need to be encouraged to identify defined areas* *within which a degree of experimentation and risk-taking might be beneficial. Too often we stifle creativity and learning by insisting upon good performance from all activities.*' And in a similar vein: '*In essence we are having to move away from hierarchical, top-down approaches to control to one where self-control, innovation and empowerment are of at least equal importance.*'

More directly addressing the construction sector, Koskela [6] in his thorough research refers to three contemporary views on production (the TFV theory, as been mentioned in part I) that must be extensively linked together and balanced also in regard to construction management resulting in task management, flow management and value management. Koskela's conclusion is that construction now faces the same situation, as did manufacturing in the early 1980s when new production developments (Just In Time, JIT, and the Quality movement) caused reevaluation of most prevalent approaches to production management. As for innovation in construction and its inherent risks, Beeby [7] suggests that the risks be reduced by innovating in relatively small steps with a consolidation after each step, and that innovations in the process of design and construction may be less risky than changes in the technology so that larger changes can be attempted. However, it must be argued firstly that it is very difficult to govern improvements and to predict which way to go; secondly, if it were possible to choose, customers probably would not allow adoption of a slower incremental by-pass, with due regard to the ever more rapid pace of changes on a global scale; and thirdly, innovations and development are urgently needed in both process and technological perspectives.

2.2 Problem statement – the basis of opportunities

Problems in construction will have to be addressed in the near future if the industry is going to fulfil its commitments to society. Paradoxically, these problems also create great opportunities for innovations. On the other hand, the insufficient acquaintance with developments and new building systems among professionals is a serious problem in itself and provides the greatest impediment to successful application in practice, as e.g. Warszawski [8] argues. There are quite extensive documentations of the problems related to the construction industry to be found in the literature. While numerous investigations have been focusing on the problems, the suggested solutions have been implemented seldom or adopted only partially. Several governmental surveys (e.g. Egan [9], SOU 2002:115 [10] and others) can be summarised into the conclusion that the main problems show great similarity regardless of country of origin. Löfgren & Gylltoft [11] offer a comprehensive review of the problems concerning construction; furthermore e.g. Koskela [6] provides some analysis of the background and the roots of the problems. In general, there are some significant sources of problems that seem to be intrusively connected and interrelated, for example the complexity and the fragmentation of the industry, the peculiarities of construction (see further Koskela [6]; especially important for bridge construction is the extensive governmental involvement as noticed in the process analysis in part I), the common division between design and construction, deficiencies in managerial coordination, and the large amount of waste and value loss that is encompassed in construction. All of these add up to the primary problem of low efficiency and profitability in the construction industry. The entrance resistance is very low, among other things leading to a large number of participants in the process (one reason of complexity in construction) and extensive competition based on lowest price. Investment in both knowledge and capital can thus be kept at a low level, and the emphasis on research and development (R&D) is negligible compared to many other industries.

However, there is really not much to be found in the literature regarding the problems in the specific sector of bridge construction (e.g. some problems in procurement etc. can be found in fib Bulletin No.9 [12] and Murillo [13] describes the problems overseas), thus some of the problems are described in the following. Although primarily referring to Swedish conditions, the description could to some extent be general applicable. To begin with, the different participants in the building process each have their own reasons and motives for acting in a certain way (as seen from the process analysis in part I of the article). Consequently, many problems are related to a lack of communication and cooperation between the parties. This also increases the fragmentation of the process, leading to each participant trying to optimise the efforts within his own limited field. Hence, no one seems to take on the responsibility of an overall approach aiming at optimising the entire process to benefit all of the participants. Here the need of strong clients being able to lead the development is obvious. Furthermore, the linkage of problems in construction seems to resemble a downward spiral loop or a vicious circle. First of all, there is a high level of competition in the matured industry of bridge construction, mainly because of the low entrance resistance. Although somewhat higher than for construction in general, still both the capital employment and the technical level needed for entrance are relatively low. Secondly, this leads to lower margins in business revenue, limiting the scope for investments in R&D. In addition, there is a large risk in venturing into R&D since there are great difficulties in assuring that advantages can be kept to benefit the investor alone. Thirdly, what follows seems to be the stagnation and rigidity of the current situation, with continued lack of improvement in efficiency and productivity aggravating the conditions. Thus, another round downward is completed. Koskela & Vrijhoef [14] also argue that the prevalent theory of construction (i.e. the transformation theory) is the main hindrance to innovations and developments in the construction sector, a statement with relevance in some respects.

In addition, some facts that promote uncertainty in the bridge construction industry are:

- A great variation in volumes from year to year. The main reason is that most infrastructure investment is subject to political decisions, although sensitivity to economic fluctuations also has some influence.
- The production series, apart from one-of-a-kind projects, are very small. Moreover, the extensive variability in geometry, aesthetics, etc., makes each bridge unique. This does not leave much economic space for heavy investments in equipment etc. for a given system; in fact, it works in favour of preserving the entrance resistance at a low level.
- The large infrastructure commissioner, mostly public authorities, seems mainly satisfied with the situation over the last decades. Thus, they have not used their huge influence as the main direct clients to change the process of bridge construction to any observable extent. Only recently, a change in this aspect can be noticed.
- The significant conservatism in the construction industry leads to problems of introducing new ideas, due to lack of confidence among the participants in the process.

On the other hand, one can argue that the great increase in uncertainty that all businesses have to cope with caused for example by the globalisation, and that project based production has become more common in general, consequently leads to an enhancement in similarity of the conditions for other industries with those for the construction sector. As the problems and uncertainties in construction by necessity seem to drive the development in a direction closer to other industries, this will also add to the increased similarity mentioned. In conclusion, this should also provide enhanced opportunities for the construction industry to apply the same approach to methods, strategies, production, etc., as in other industries. These circumstances are illustrated in Figure 2.

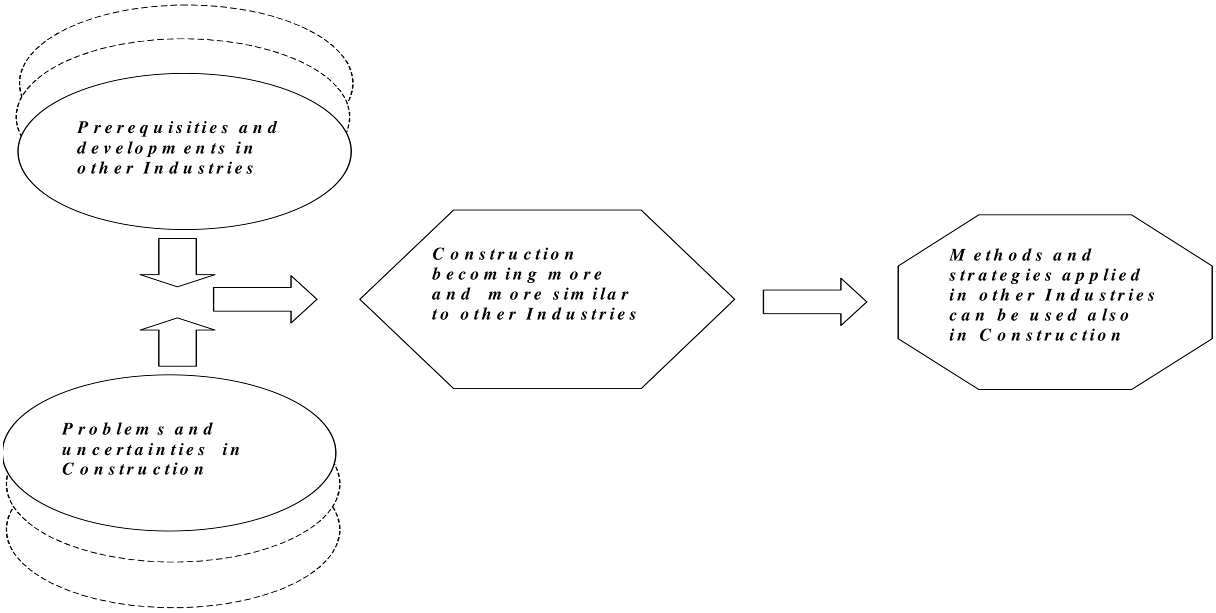

Figure 2. Problems – the base of opportunity.

3. EMERGENT STRATEGIES ON HOW TO ATTAIN AN INDUSTRIAL PROCESS

3.1 The basis for foundation of industrial bridge construction

As we have seen, there are many different ways leading towards an industrial process for bridge construction. In this brief summary, emphasis is laid chiefly on overall factors influencing the creation of a new process. In order to reach sustainable industrial development in bridge construction, some fundamental demands will have to be fulfilled. There must be suggestions on how to organise the industrial construction process, since the current process is not particularly suited for industrial purposes. Some kind of consistent theory of production will have to support the industrial ideas. There also have to be some basic ideas of industrial concepts, utilising appropriate developments in materials, design and production methods, etc. In addition, there is need for a comprehensive interconnection of all the parts, namely by means of information and communication technology (ICT), as have been discussed (compare part I of this paper).

Moreover, effects of globalisation and expanding markets is likely to result in considerations about demands for rapid development of new products and continual improvement in cost efficiency becoming very important, and the only way to secure the competitiveness of a concept over a period of time is to ensure continuous development and improvement. This is one of the principles from the Quality movement (see for example Frid [15] for an introducetion). In terms of the process, this will mean constant process development. Consequently, a continual flow of ideas is needed. It has to be clearly stressed that the industrial process is merely an infrastructure for an efficient production of bridges, linking the different parts involved. Thus one vital task is to offer guidance and hosting for good ideas and innovations.

Industrial concepts of bridge construction are possible only through intensive correlation and co-operation between different disciplines, as has been pointed out. Accordingly, there will be aspects bearing on the organisational or managerial domain as well as aspects stressing the technical domain, but there will be no aspect with emphasis in just one of the domains. The cornerstones of industrial concepts of bridge construction can be identified as the three P's –

process development, productivity development and product development – forming a progressive, continuous circle integrated by ICT, as visualised in Figure 3.

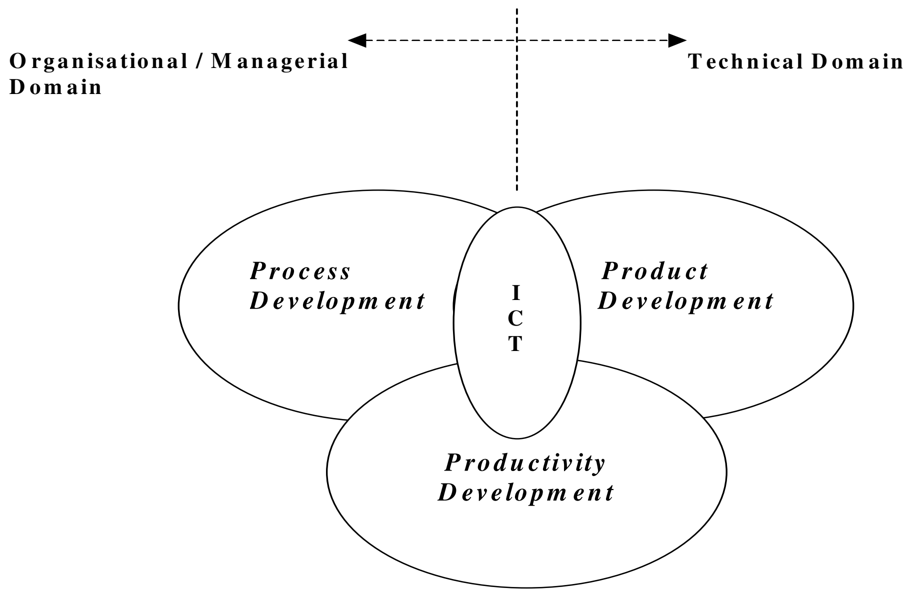

Figure 3. The cornerstones of Industrial Bridge Construction, the three P's.

To begin implementing industrial ideas, it seems necessary to suggest an appropriate sketch of a suitable process to channel the new ideas. For a start, the process is likely to contain the core of construction – the production and the design stages. In a way the two other P´s, productivity and product development are embraced with the process, since the process will have take all activities and all participants into account as well as constituting the infrastructure to support and govern the actual production. The main emphasis in the process development is flow management (the F view in the TFV philosophy, compare Koskela [6] and part I of this article), aiming at eliminating waste and no-value-adding activities. The implementation of an industrial process will provide one part of the immediate competitiveness of the bridge concept, while to ensure continuous process development will enhance the performance and govern the competetiveness in the long run. As the industry and the customers become more familiar with industrial concepts, the borderlines of the process are likely to be extended. For example, it is possible that sales management will be somewhat integrated into the domain of the commissioner, allowing early contributions in the planning phase of infrastructure projects. At the other end of the process, it is possible that after-sales management will develop to provide different services in the 'operation and maintenance' stage. An outline of the process from a managerial perspective has been presented in part I of this article (part I, Figure 4).

Product development can be seen as a continuous loop of optimisation and refinement of the features of the product, responding to feedback of deficiencies from the whole process, as well as the initial design of new products. The aim of the improvements is to enhance the performance of the product, which covers a wide range, from aspects such as easier production, simpler assembly or jointing on-site, emphasising more efficient construction, to features such as a more appealing layout of the design or improvements in durability, more directly relevant to the customer relationship. This is probably one of few opportunities to ensure competitiveness over a period of time. Hence, emphasis is mainly on value creation and value management (the V view in the TFV philosophy) in product development.

By productivity development, as a cornerstone in this context, is meant merely the production issue of how to attain more efficient production, for example by rationalising prefabricated production in factories or the execution on-site. Thus, the main emphasis of productivity development is in the transformation vein, the T view of the TFV concept – to execute the decomposed transformation tasks in the most efficient way – although, of course, the two other P's contribute in rationalising these tasks. It can be said that productivity development is the ultimate goal, i.e. more efficient construction of bridges, while the two other P's tend to be the means of fulfilling this goal, by developing the products, smoothing the way and providing the necessary tools for governing progress in the right direction.

Thus, all three cornerstones are closely linked and appropriate attention must be paid to each of the P's in order to reap the full benefits of industrial concepts. For example, a new product with excellent performance has small chances of becoming a success if it is not produced efficiently and if it is not guided by a rational process all the way to the market. Similarly, an excellent production technique is hardly relevant if the product is undeveloped and unwanted on the market, or if the process fails in providing a market connection. Thus, it can be said that the three P's have their 'centres of gravity' in somewhat different domains, as been noted above (compare Figure 3). While process development seems to rely more on the organisational/managerial domain, the emphasis of product development lies in the technical domain, and productivity development tends to stress both domains about equally. It has to be remembered, though, that as in the case of balancing the three P's, efforts in the two domains must be thoroughly balanced as well.

The subsequent work builds upon demands raised by the different participants in the process with the overall goal of satisfying the customers, i.e. real customers and society represented by public clients. As a part of this, one aim is to suggest possible solutions to the problems the industry is facing (see above and e.g. SOU 2002:115 [10]). It is evident that the driving participant in terms of developments, changes and implementation of the new process must be the industry providing and utilising the products, i.e. primarily public commissioners, contractors or even consultants. Hence, other participants in the process will have to be convinced and realise the benefits.

3.2 Emerging technical solutions

Current research and development are yielding a large amount of promising results, and the great challenge is to combine the right features that can materialise into emerging technical solutions. In this context, a few very interesting areas with a bearing on bridge construction will be discussed.

In the field of materials science, there are many applications that would fit into an industrial concept. Such applications in the field of concrete are high-performance concrete or ultra-high-strength concrete, self-compacting concrete, fiber reinforced concrete using different fiber materials (e.g. see Ay [16] and Guerrini & Rosati [17], about research on steel fiber reinforced high-performance concrete for bridge structures), lightweight aggregate concrete and combinations of these. In this project, studies on an innovative solution for structural joints between prefabricated concrete elements have been conducted; see Harryson [18]. This result from materials development in combination with research in structural engineering can be seen in

Figure 4 (the joint before casting). The concrete in the joint is a steel fiber reinforced high strength concrete called Compact Reinforced Composite (CRC).

Other materials developments to be combined with concrete (since a bridge are likely to always contain concrete in some way) are high-strength steel and the exciting progress in fiber reinforced polymer composites. Also new developments in wood technologies could be of interest (especially in composite action with e.g. concrete), but they will have to be excluded since wood will not withstand the excessive lifetime assessments under severe environmental conditions that are normally used for bridges. Moreover, Flanagan et al. [1] explore some other emergent materials developments, such as ceramics, shape memory alloys, biomimetics ('the abstraction of good design from nature') and nanotechnology (ultimately to construct material 'bottom-up', i.e. from the level of atoms or molecules). It is quite possible that some of these advances will be applicable also in bridge construction. Further, recovered techniques such as concrete-filled steel pipes or externally prestressed concrete, and new developments in adhesives, new joints between components, and many more that could be mentioned are, of course, highly applicable in the context of industrial bridge concepts. Another exciting tendency is development of smart structures, i.e. structures with embedded systems (e.g. microchips, optical fibre) that can communicate to simplify construction or to facilitate continuous monitoring, etc.

Other developments, such as in manufacturing (e.g. automation, robotics), transportation (e.g. road-rail solutions, by air etc.) as well as assembly and erection techniques (e.g. heavy lifting, jointing techniques) will also play a significant role in specific concepts; for example, in concepts with a high degree of prefabrication the transportation and erection of the components will be crucial issues. In this respect active benchmarking of other industries is extremely valuable. Developments in on-site activities such as participating formwork and efficient scaffolding could also be of interest for *in-situ* cast alternatives.

Figure 4. An example of result from combination of materials development and research in structural engineering; an innovative new joint concept for structural joints between concrete members.

A variety of different prefabricated concrete concepts, with different combinations of con-crete technologies and variation in degrees of prefabrication, will be highly competitive. Pro-vided that a proper and rational detailing is developed, especially of such essential and sensitive parts (both for constructability and durability reasons) as the construction joints, it is most likely that precast concrete will somehow be included in a successful concept of industrial construction. It is obvious that combinations with industrial *in-situ* cast parts could also produce strong alternatives.

Further development of composite concepts, such as the traditional one with the combination of steel and concrete, will continue to be competitive. But more interesting are the new innovative composite concepts that will emerge, utilising the essence of the latest materials developments. The problem in this context is not to find applications with potential, but rather to get things going at the start since new materials developments tend to be very expensive before they are commonly used in large volumes. It is necessary that the concepts that are likely to surface adopt a system approach, so that the system will include not only all parts of design and construction, but all parts of the industrial process as well

3.3 The important design issues

As mentioned, the latest materials developments are likely to lay the foundation for competitive industrial bridge concepts. However, the counterpart of these developments in structural design research seems to be lagging behind, thus becoming the bottleneck and a major hindrance in implementing the research results. There is a great need for this research in order to obtain the necessary authority approvals, and the insufficient cooperation between structural-design and materials scientists is very unfortunate. Also in the light of the problems faced with two new post-tensioned box-girder bridges in the Stockholm area in Sweden during 2002 see Hallbjörn [19], which seems to be the result of lacking knowledge in structural engineering, the importance of research into design issues cannot be stressed enough. The major question to be answered by researchers in structural design is how to design with these materials, resulting in practical design rules, especially when more than one material is concerned and composite action is accounted for. Thus, the conventional aspects of design (e.g. safety against failure, serviceability demands, fatigue) will have to be investigated. Other aspects that have been focused on recently are durability and lifetime assessments, sometimes summarised with regard to economic and environmental effects in life cycle analysis or life cycle cost.

Another very important design issue with regard to new materials developments is to find the optimal structural forms for the new material. Hence, copying the old conventional forms and cross sections used for our ordinary materials will not do; e.g. compare Keller [20]. One certain feature of a competitive future concept is to use the right material with the right structural form in the right place combined with a high structural utilisation of the materials. Moreover, research in loadings can be beneficial, especially since live load and load from constraints have such a major impact on bridges; the actual load must be correct before it can become worthwhile to optimise the structure. However, this liability falls upon the public authorities responsible for the design codes.

As far as building systems are concerned, an extremely vital issue is to develop fast, smooth and simple detailing of joints between components (inferred to be a large problem by Warszawski [8]) for example standard interfaces. As been mentioned, studies on an innovative solution for joints between prefabricated concrete elements have been conducted in this project; see Harryson [18].

Other design issues more specifically addressed to each concept are those connected with the constraints of the production, transportation, erection and assembly; solutions allowing a rapid and smooth transition through these phases (e.g. production-friendly solutions) are valuable. In addition, design of the temporary works needed, i.e. the on-site preparations, will have to be addressed.

3.4 Towards new templates for production

As been noted by Koskela & Vrijhoef [14], the construction industry has neglected or not managed to implement at least two major production templates that deeply restructured other industries – namely the mass production and lean construction (e.g. JIT). The arguments from the industry was (to some extent rightly) that the specific problems connected with the peculiarities of construction could not be overcome. However, time is now proceeding towards the inevitable point where new templates for production will be an inescapable reality. New computerised features such as e-Tags (or i-Tags) and e-Commerce will most likely entail a small revolution in the whole chain of businesses, not least in the supply chain (or supply network) management. Encouraged by the continuing enhancement in benefits from ICT, the tendencies of this industrial evolution could be similar to the suggestions in the following.

A strong improvement factor, probably becoming a prerequisite some years from now, is to perform construction in a protected environment. This means weather-protected production (indoors in industrial facilities or temporarily covered on-site work), but also provision of efficient facilities and safety precautions. It must be kept in mind that weather protection is only the means, and that it is supply of the appropriate methods that leads to increased efficiency, which is sometimes forgotten; compare Moström & Asplund & Samuelsson [21].

As for most heavy work in developed countries, the development in the construction business will not differ – machines will replace heavy manual work. This is for economic reasons (cost reduction) but also due to problems in finding employees willing to undertake these heavy tasks and for labour safety. These developments have already started with demands from authorities on labour environments as the driving force. Hence, there will be a significant increase in mechanisation of construction sites, both on-site and off-site. Further developments in machines, tools and other equipment will enhance the trend of man being replaced by machines. With the aid of powerful computer support and ICT, the automation will gain speed and industrial manufacturing in construction will use robotics extensively, especially in off-site production. The conclusion is that the production will become more high-tech and the facilities will be more like any ordinary industrial production for both on-site and off-site production.

As for the completion of structures on site, the evolving methods of erection and assembly as well as other on-site preparations will strive towards simplicity with a minimum of manual work. There will be a high degree of mechanisation, and developments in heavy lifting, mounting, jointing etc., will encompass many special machines and equipment to allow rapid installation, thus increased parts of structures (components as well as systems) is likely to be manufactured elsewhere and assembled on site. An objective will also be to complete the structure at once, thus avoiding later on-site reestablishment for complementary works. In Figure 5, an example of a common semi-industrial bridge concept is shown, a bridge with prefabricated pretensioned concrete beams and prefabricated concrete slab as lost formwork for the traffic slab. This must be regarded as an example of an 'industrialised' bridge concept, as argued before (compare Chapter 2.1 in part I).

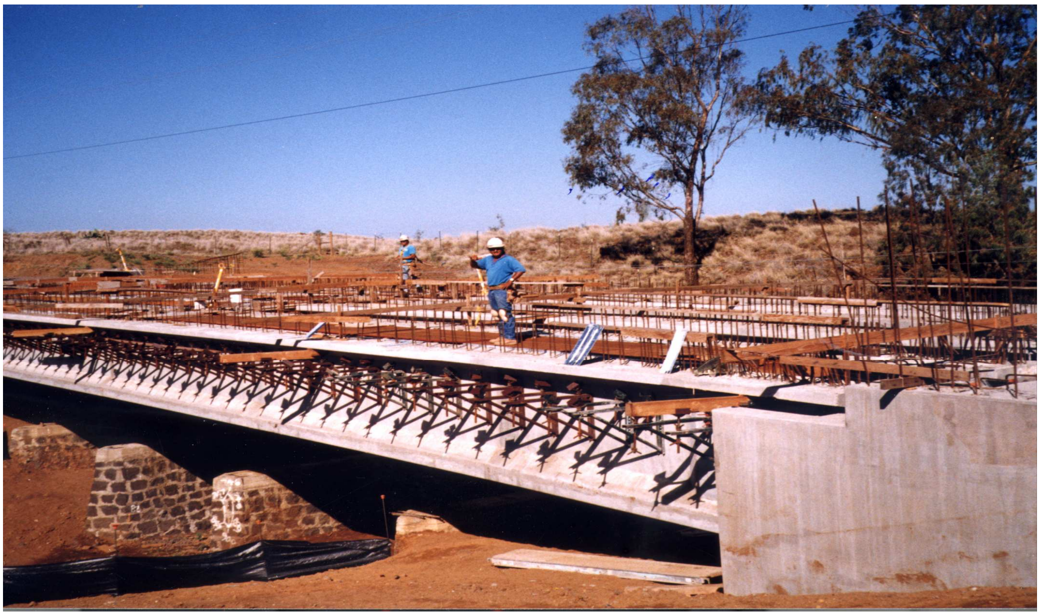

Figure 5. A common semi-industrial bridge concept, here an example from the island of Hawaii.

4. SETTING PRIORITIES

4.1 Developments with potential

All developments presented above have large potential for improving efficiency in construction. The following main features are therefore discussed in the light of the key issues of potentiality, time horizon and difficulties in implementation.

ICT and computer science:

Developments in ICT will provide an infrastructure for the system; thus there is an urgent need for this development and it also has a very high potential. The current development is extreme-ely rapid, so the time issue should not cause problems. Such developments are also exceptional-ly easy to implement if they are user-friendly, since there are no authorities or similar agencies to give approval and the general computer consciousness is increasing irreversibly. On the ot-her hand, this calls for caution and appropriate controls to avoid drawbacks, e.g. bugs in the software products.

Structural design and materials science:

Developments in structural design and materials science are extremely important in order to proceed with new concepts of potentially high interest, not least the developments in concrete. The state of research differs depending on the material, but generally there are several materials developments ready to implement, e.g. fiber reinforced polymer composites. Usually, however, the research in design matters is lacking behind, as argued earlier, and will possibly act as a bottleneck when implementing these results. The design matters are also very important in order to achieve the necessary authority approvals, which probably is one of the difficult stages to overcome in the implementation of different developments. In addition, the general conservatism pervading the industry will increase the time for implementation.

Process developments:
Compared to the current situation, there is a very high potential in introducing process improvements. A model of the new process seems as a prerequisite for implementing a new concept and as an example a framework for a process model has been presented in part I of this paper (part I, Figure 4). However, implementation is likely to be gradual depending on the generality of the model as well as the trust generated among the participants; hence a good start would be implements parts of the process or to find ways of verifying a process model at small scale.

Basic production template:
On-site versus off-site production is a continuously interesting issue. As far as bridges are concerned, the big achievement is likely to be in off-site fabrication, if the increased complexity due to the industrial process is firmly counteracted as been argued. There are of course opportunities to industrialise on-site fabrication to a certain extent as well (e.g. prefabricated reinforcement cages, temporary weather protection, the use of self-compacting concrete, integrated formwork, etc.), but in the context of traditional on-site construction it is possible to sense the limitations already at the start. However, there will most likely develop competitive on-site concepts (or in combination with prefabricated components), but it is argued that work on site should be minimised (even though it is unavoidable to some extent), and the potentially most interesting of the basic conceptual principles undoubtedly seems to be off-site fabrication.

Production methods and techniques:
Production methods and techniques, despite their high potential, can be established in detail only in conjunction with the basic concept or after it is decided upon. Many developments are available on the market, i.e. many applications are already implemented, so the vital work will be to combine the most competitive methods and techniques.

4.2 Priorities between developments

The conclusion from the above is that all development areas are needed for implementation of the final concepts and setting priorities are difficult. However, to accomplish the gradual improvements that are likely to take place, development in ICT seems most important to provide the infrastructure for the new process, but since research in ICT is beyond the scope of this project the developments in this area must instead be followed closely although no actual work will be done in it. It is also a high priority to commence the implementation of the new industrial process by finding ways to study or verify a process model. Of equally high priority is the issue of continuing the recent materials developments into the design area, so as to be able to introduce them on the construction site. In this respect, it is crucial to investigate those applications that are in line with a suggested concept of industrial bridge construction, for example if prefabricated concrete elements are included in the concept, the detailing will be essential to investigate – especially new solutions of structural joints between the elements, as has been mentioned. When deciding upon a concept, investigations regarding production methods etc. become interesting. For the continuation of this research, some conclusions about special project priorities have been drawn in Chapter 5.

5. CONCLUSIONS

Apparently, we are close to a major shift in the foundations of the construction industry, and this is particularly obvious in bridge construction. The results of the present and other research clearly indicate that there is an immediate need for a substantial alteration in the current construction process. Furthermore, the need spans across the whole industry: there are demands for new form of procurement, new ways of financing, new production templates, a new integrated design process, new forms of long-term cooperation, new ways of communicating, increased development of even more powerful ICT applications, attraction of more competent young people into the construction business, and also most urgently the need of developments in new techniques and methods, as well as an adoption of developments in recent materials science. The conclusion is that implementation of an appropriate industrial bridge-building process among all participants would definitely lead to changes and provide possibilities to solve many problems. If it could be made interesting and prove profitable to enhance developments, the transformation would be initiated and gradually the principles of the new paradigm will spread among all participants. Actually, the limitations of a new process are only in the minds of the participants.

The real physical limitations, though, consist of a risk of not being able to present feasible solutions, especially with regard to the previously stated problem of increased complexity or on purely technological matters. It must be emphasised that, with the conservatism that currently permeates the construction business, new concepts to be adopted into an industrial bridge-building process will have to be competitive and show profitability from the outset if they are likely to be given a reasonable chance of survival on the market. This is probably the most challenging and most difficult task to succeed with. Obviously, there is much work to be done before we arrive at new concepts and a new process ready for implementation. Given the multidisciplinary features, it is not feasible for a single researcher to accomplish this; more probably, massive efforts will result in multiple contributions from different disciplines. It would be a real progressive step to initialise a major multidisciplinary research project in the field of industrial construction, linking many researchers from different areas together to strive for a mutual goal. For the continuation of this research at Chalmers University of Technology, some conclusions about project priorities must be drawn in order to provide suggestions for the future work. The importance of the client perspective, both the direct customer (e.g. drivers etc.) and the formal customer (e.g. the national road or rail administrations), must be stressed in this aspect, that is to focus on features beneficial to the client. Therefore, the first overall emphasis will be to identify a potentially successful concept for industrial bridge construction and to apply the basic ideas outlined above to this concept. With this concept as the point of departure, the main focus will be to investigate new or approved techniques as a continuation of recent materials developments from a design viewpoint, but also to perform studies of construction characteristics such as production methods, assembly, etc. for the specific concept. The aim is also if possible, to try to find ways of studying or verifying a suitable model of the process from a client perspective, if feasible within the same bridge concept.

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Determination of Dynamic Uniaxial Tensile Strength of Concrete using Modified Hopkinson Pressure Bar

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ABSTRACT

The tensile strength of concrete is highly dependent on the strain rate. For applications where the structure is subjected to impact loading, which results in tensile stresses, the so-called dynamic tensile strength is of importance. A simplified method to determine the dynamic tensile strength by using a Modified Hopkinson Pressure Bar is presented. In the performed tests a compressive stress wave was introduced in an incident bar and the attached test specimen by a projectile. Tension stresses, which arose in the test specimen due to reflections of the compressive wave, caused fracturing of the brittle material. Strains at two points in the incident bar were measured and the tensile strength was determined by using elementary uniaxial stress wave theory.

Key words: hopkinson bar, modified hopkinson pressure bar, impact loading, concrete, tensile strength

1 INTRODUCTION

Determination of tensile strength of concrete has been a subject of discussion since the use of concrete began. At an early stage it was observed that the loading rate has a significant influence on the so-called dynamic tensile strength. To mobilise the loading rate at a considerably high level, the Split Hopkinson Bar (SHB) is used in several applications. A uniaxial (tension) stress wave is generated and the tensile strength of a (brittle) material can be determined by studying the transmitted stress wave. However, producing a pure tensile stress wave requires a rather comprehensive testing arrangement. Producing a stress wave in compression is much more straightforward.

In this paper a method to determine dynamic tensile strength of concrete in a so-called Modified Hopkinson Pressure Bar (MHPB) will be outlined. A compressive wave was mobilised in a simple manner and the tensile stresses were caused by reflections. The tensile strength is proposed to be determined by using measured strains and elementary application of uniaxial stress wave theory.

2 THE MODIFIED HOPKINSON PRESSURE BAR

2.1. Background

Material testing for impact loading was introduced by Hopkinson [1]. He generated a compressive pulse in a bar with an explosive charge or an impacting bullet. The compressive pulse was reflected in the opposite end of the bar as a tensile pulse and caused fracture in brittle material. This technique of testing material subjected to impact loading has since been a common method, and has been further developed by several researchers. In [2] Kolsky used the idea of wave propagation in a bar was used and the method was made operational for numerous appli-cations. He introduced what today is called the Split Hopkinson Bar (SHB). The general idea is shown in Figure 1 for a compressive wave. A compressive wave is introduced to the incident bar by the projectile. When the wave reaches the boundary between the incident bar and the test specimen, the wave is partly reflected, due to different impedance in the bar and test speci-men. For a small specimen the variation of the strain over the volume is limited and the strain is therefore assumed to be uniform at each time. However, for very fast loading this assump-tion is not valid and thereby limits the maximum strain rate of the method.

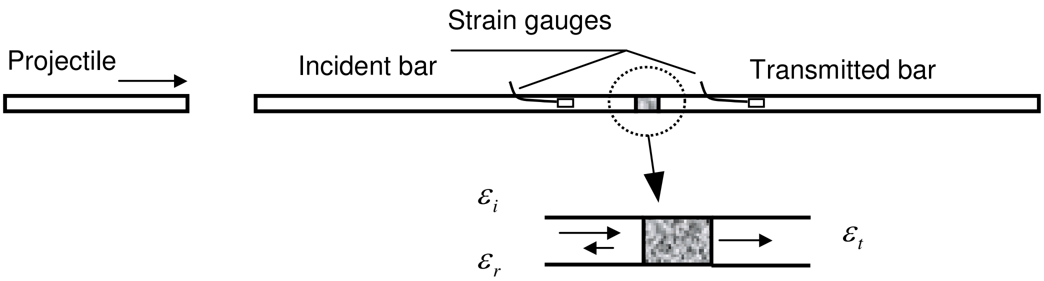

Figure 1. Split Hopkinson bar (SHB)

In concrete research the SHB has been given attention during the last three decades. Comprehensive tests for impact strength of concrete in compression was performed in [3]. A review of the development of the SHB for impact testing of concrete in tension, developed at Delft University of Technology, was presented in [4]. The method continuously improves and in [5] an extension of the classical SHB method that considers non-uniform stress fields in the axial direction for specimens subjected to compressive stresses was presented. In [6] the limitation of strain rates by using the SHB was discussed and an experimental method for tensile testing of concrete under high strain rates by spalling was presented. A specimen of concrete was attached to the incident bar and tensile stresses were achieved when an incident compressive wave was reflected in the free end of the specimen.

At Chalmers University of Technology, Bohloli [7] proposed a similar, but simplified, method to deter-mine tensile strength of rocks with a Modified Hopkinson Pressure Bar (MHPB). A principal set-up of the system is shown in Figure 2. The specimens of granite are glued to the incident bar via a threaded adapter. When the projectile hits the incident bar, a compressive wave prop-agates towards the specimen. Due to the difference in impedances in the incident bar and the test specimen, reflections arise which cause tensile stresses in the specimen and may lead to fracture. The outer part of the specimen then separates from the incident bar and the remaining specimen. By measuring the velocity of the released part it is possible to determine the tensile strength.

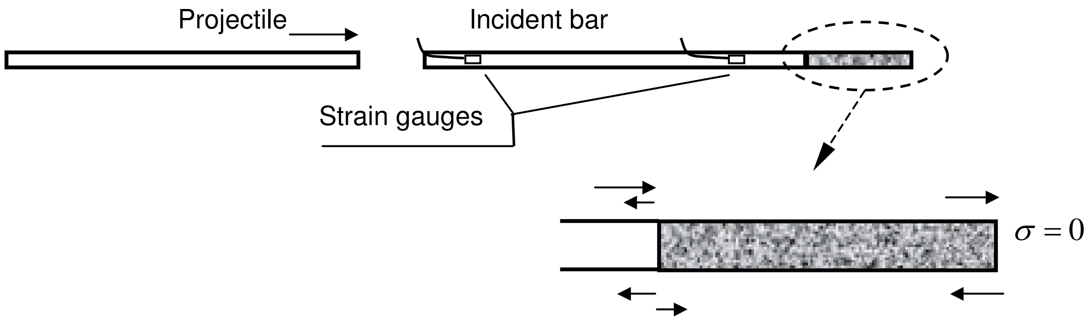

Figure 2. Modified Hopkinson Pressure Bar (MHPB)

This method is a simplified way to determine tensile strength of brittle materials. The main advantage is that it is simpler to introduce a compressive stress wave compared to a tension wave. The drawback of the method is that the strain rate is non-uniform and the stress level has to be calculated in a more circumstantial way. Furthermore, to determine the tensile strength the velocity of the projectile has to be successively increased until fracture arises.

The simplicity of the method makes it, however, attractive for determining tensile strength of plain concrete. By developing the method to determine the stress of ultimate strength, one of the drawbacks was eliminated.

2.2. Aims of measurement

The main aim of the test arrangement was to mobilise a stress wave in compression in the incident bar in a simple manner. The tensile stresses arise from reflections in the test specimen. In Figure 3 the strains in the incident bar and test specimen are shown for an idealised stress wave at four different stages of the wave propagation. To simplify understanding, an idealised stress wave with an initially infinite strain rate and subsequently constant strain level is addressed. At left of Figure 3, the different components of the strain wave, caused by reflections, are separated. At right the resulting strains are shown.

When the compressive wave reaches the test specimen, reflections occur (A). The specimen has lower impedance compared to the incident bar, and a tension wave is reflected in the backward direction of the incident bar (B). The remaining part is transmitted into the specimen as a compressive wave. The strain magnitude in the specimen increases, due to the lower impedance of the specimen. When the compressive strain wave in the specimen reaches the end, it will be completely reflected as a tension wave (C). Finally when this tension wave reaches the boundary of the incident bar, which has high impedance, it will partly be reflected as a tension wave (D). Resulting tensile stresses arise in the specimen. If the resulting tensile stresses reach the tensile strength of the (brittle) specimen, failure will appear.

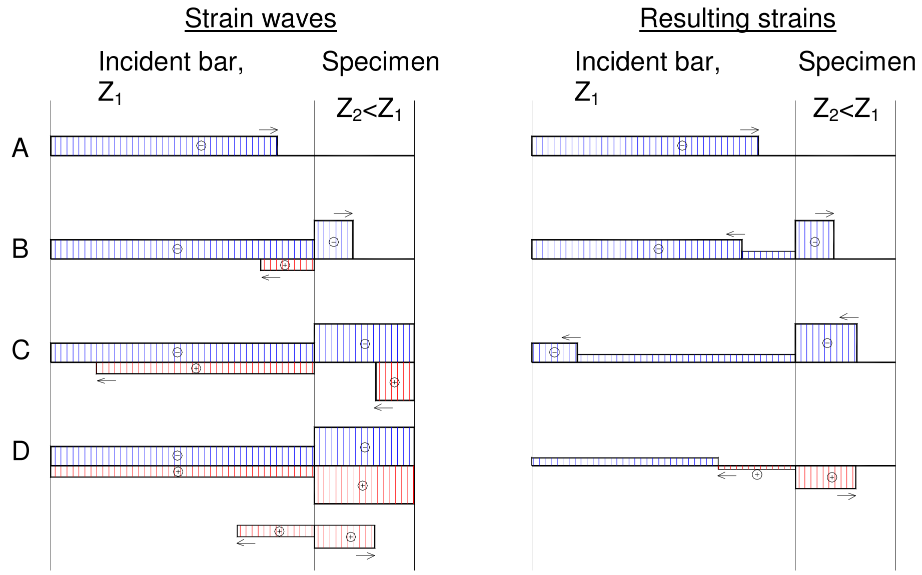

Figure 3. Reflections in test specimen mounted at end of incident bar.

2.3. Test arrangement

The test arrangement for testing of dynamic tensile strength is shown in Figure 4. The main measures of the equipment are presented in Figure 5.

The projectile was a steel bar $\phi25$, which was chromium-plated, and supported by two axial roller bearings. In the end of the projectile, in the boundary against the incident bar, a hardened flat top was attached. Before testing, the helical spring was prestressed, and the projectile was accelerated when a locking mechanism was released. The incident bar is also a steel bar $\phi25$, which was supported at two points with pendulums of chains. In the end of the incident bar was an adapter was attached via a threaded joint. The test specimen was glued to the adapter. To reduce the strain rate in some of the tests a cushion of a soft plastic sheet was placed between the projectile and the incident bar.

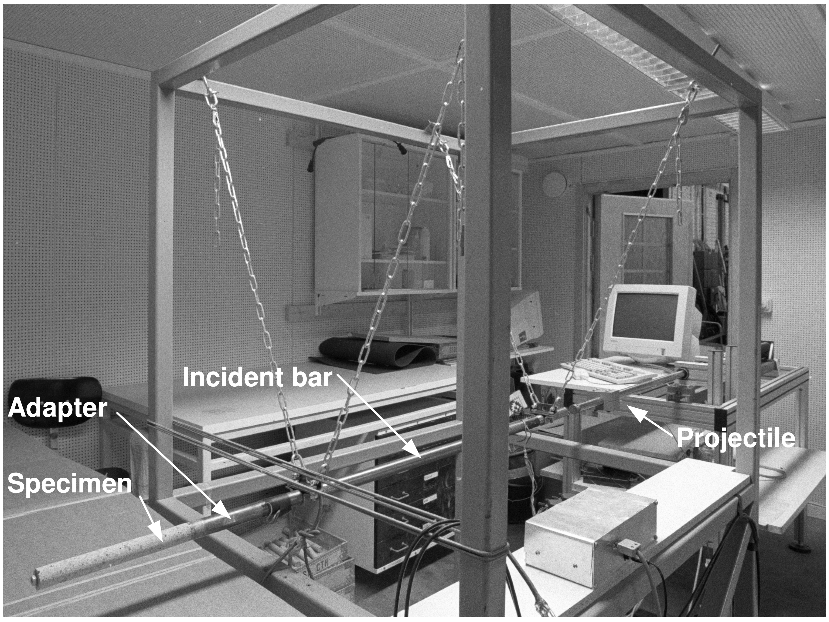

Figure 4. Photo of the test arrangement for tensile strength of concrete

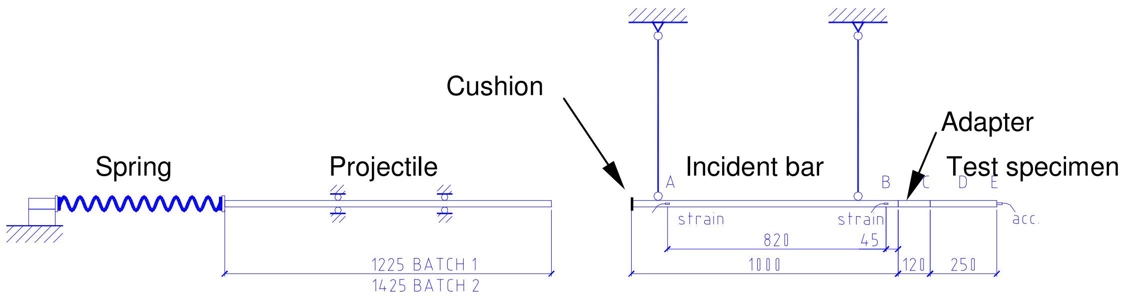

Figure 5. Main measures of the test arrangement

The test specimens were manufactured by casting a volume of concrete (~250x300x500 mm^3), which was wet-stored. The testing were performed on wet-stored ϕ150x300 cylinders after 28 days (f_{cm}~62 MPa). The specimens were bored out and glued to a threaded adapter in steel. Two batches were performed. The first includes 15 specimens. However, due to problems of evaluation of the test, which will be described subsequently, the projectile length was increased. The second batch includes 33 specimens. Only results from the second batch will be presented.

The measurement was performed by high-speed simultaneously sampling DAQ (National), and particular software for the measurement was developed in labVIEW. Strains were measured with strain gauges in either full or half Wheatstone bridge. The signals from these gauges were amplified to a suitable signal range (±5 V). Due to the dynamic measurement, the strain signal amplifier was designed in such a way that the strains were set to zero at the beginning of each measurement. Accelerations were measured with piezoelectric accelerometers of type Piezo-electric ICP® Sensor, with a sensitivity of 0.85 mV/g. Data were collected parallel on maximum five channels with a sampling frequency of 100 kHz during 0.20 s.

Before the testing a calibration of the test set-up was carried out. The repeatability of the test set-up was good, c.f. Figure 6. Using uniaxial stress wave theory it was possible to compare the measurements of strain and acceleration. The conclusion was that the measured magnitudes of acceleration have reasonably good agreement (± 5%) with corresponding calculated values based on the strain measurements. However, in the same evaluation it was concluded that there occurs a significant time delay between measured acceleration and strain signal, which probably derives from phase error in the amplification of the strain signal. In the evaluation of the tests, strain signals will be used, and if the phase error was constant between the channels it will have no impact on the result.

The test arrangement was evaluated and the measurement system was calibrated before testing; compare [8]. In Figure 6 measured strains at points A, B and D are shown. To determine the repeatability of the test arrangement, three equal tests were performed with a specimen of aluminium.

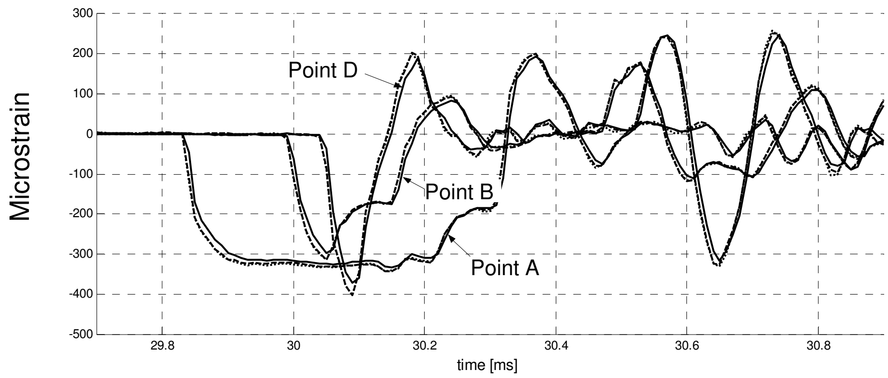

Figure 6. Strain measurement. Three equal impacts.

2.4. Theoretical background

In the present work it was assumed that the wave propagation could be considered uniaxial. This is valid if the wavelength is long compared to the radius of the bar, where the wave propagates, and if the loading rate is not too high in the initial part of the loading. Fagerlund and Larsson [3] discussed these circumstances and concluded that if the ratio between radius and wavelength is $r/\lambda < 0.1$ the wave propagation in the transverse direction is of minor importance and the wave speed is close to the acoustic speed. Regarding the loading rate there exists no certain limit value, but this effect is of minor importance.

The elementary governing equation of motion for an elastic bar, with Young modulus E and density ρ, for one-dimensional wave propagation can be expressed as

$$c^2 \frac{\partial^2 u}{\partial x^2} - \frac{\partial^2 u}{\partial t^2} = 0 \qquad c = \sqrt{\frac{E}{\rho}} \tag{1}$$

where u is the displacement and c is the acoustic speed or the wave speed. The general solution for the wave equation can be written according to d´Alembert's principle as

$$\begin{aligned} u(x,t) &= f^+(g^+) + f^-(g^-) = u^+ + u^- \\ g^+ &= x - ct, \quad g^- = x + ct \end{aligned} \tag{2}$$

The physical interpretation of this solution is two waves with the shapes f^+ and f^- respectively. The first term represents a wave that propagates in the positive x-direction and the second in the negative x-direction. Both waves propagate with speed c, i.e. the acoustic speed. The strain in the bar can be determined as

$$\varepsilon(x,t) = \frac{\partial}{\partial x} u(x,t) = \frac{\partial f^+}{\partial g^+}\frac{\partial g^+}{\partial x} + \frac{\partial f^-}{\partial g^-}\frac{\partial g^-}{\partial x} = \frac{\partial f^+}{\partial g^+} + \frac{\partial f^-}{\partial g^-} = \varepsilon^+ + \varepsilon^- \tag{3}$$

Hence the strain can be derived as the sum of strain waves in the positive and negative x-directions. In Figure 7 a bar consisting of two different materials is shown. In the boundary between the two materials the requirements of continuity and equilibrium have to be fulfilled.

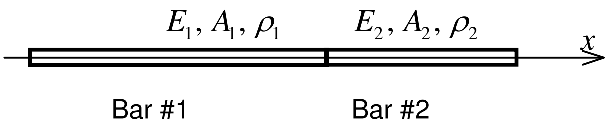

Figure 7. Stress wave propagation in a bar of two different materials

By continuity and equilibrium conditions the strains in the two parts can be determined. Assuming that the incident stress wave initially propagates in material 1 in the positive x-direction (ε_1^+), the transmitted wave into material 2 can be determined as

$$\varepsilon_2^{\ +} = \frac{c_1}{c_2} T_{12}\ \varepsilon_1^{\ +} \tag{4}$$

and the reflected wave in the negative direction can be expressed as

$$\varepsilon_1^{\ -} = R_{12}\ \varepsilon_1^{\ +} \tag{5}$$

where the transmission and reflection coefficients from material 1 to 2 are introduced:

$$T_{12} = \frac{2\,Z_1}{Z_1 + Z_2} \qquad\qquad R_{12} = \frac{-Z_1 + Z_2}{Z_1 + Z_2} \tag{6}$$

where the impedances for the respective materials are defined as

$$Z_1 = \frac{E_1\ A_1}{c_1} = \sqrt{E_1\ \rho_1}\ A_1 \qquad\qquad Z_2 = \frac{E_2\ A_2}{c_2} = \sqrt{E_2\ \rho_2}\ A_2 \tag{7}$$

Equations (4) - (7) are the elementary basis for the evaluation of strains and stresses in the test specimens.

2.5. Determination of stresses in the test specimen

The theory of stress waves outlined above is based on the fundamental assumption of elastic material. During the fracturing process of the concrete subjected to tensile stresses, this assumption obviously is not valid. However, by assuming that the failure is localised in one fracture plane, the classical stress wave theory still can be applied. Next, the wave propagation in an elastic specimen will initially be discussed. Thereafter wave propagation during fracturing of a brittle specimen will be considered.

The wave propagation can be illustrated graphically in a time-space graph. In Figure 8a the wave propagation in the incident bar and the test specimen, both of which are assumed to be elastic, is shown. The strains in the test specimen can be determined from just the measured strains at point A, as long as no reflected waves reach A.

To calibrate the system, the test specimen was replaced with an aluminium bar. Strains were measured at A, B and D. In Figure 9 are measured strains at point D are presented together with calculated strains at the same point based on measured strains at point A for three different impact levels. The agreements between measured and calculated strains are reasonably good for the initial compression and tension waves.

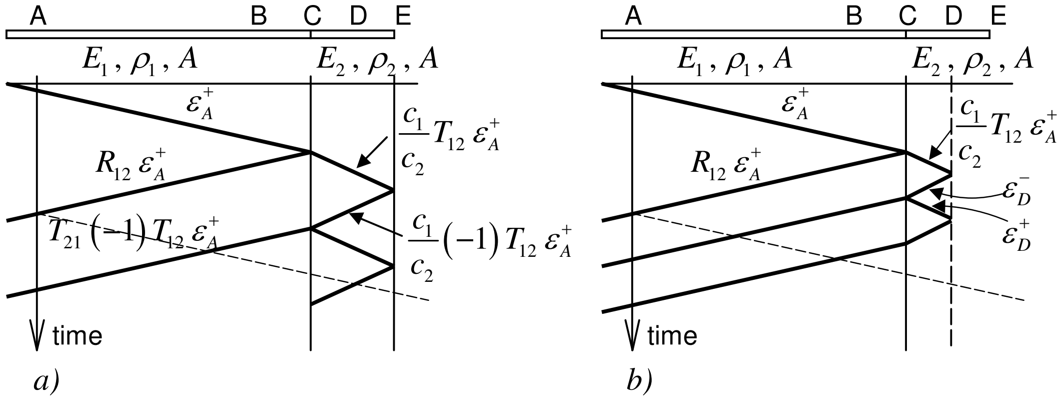

Figure 8. *Graphic illustration of wave propagation.*
(a) Elastic incident bar and test specimen (A – E).
(b) Elastic system from A to D.

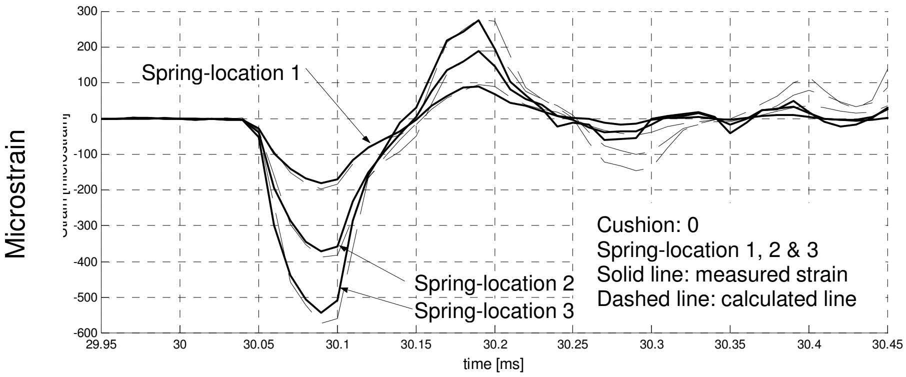

Figure 9. *Strain measurement compared with calculated values at point D for a test specimen made of aluminium for three different impact levels, i.e. spring-locations.*

However, when determining the tensile strength of concrete the test specimen is obviously not elastic. Determining stresses in the specimen based on measured strains in point A is valid only as long as the response in the specimen is elastic; but by also using measured strains in point B, the evaluation of stresses in the specimen can be further improved.

To simplify the evaluation of the maximum tensile stresses, two assumptions are introduced. A tensile failure of brittle material, such as concrete, is localised in discrete cracks. Furthermore, the non-linear response of the material is limited before the maximum tensile stress is reached. In Figure 8b the wave propagation of a system with elastic response between A and D is shown. The location of point D is assumed to be known, but the stresses in the same point are assumed to be unknown. The strains in points A and B are measured continuously. Initially the strain in point A consists only of a wave that propagates in the positive *x*-direction. This is valid until the first part of the stress wave has travelled forward and back in the incident bar.

The strain in point B consists of two waves according to (3). The wave in the positive direction can be determined from point A as

$$\varepsilon_B^+(t) = \varepsilon_A(t - t_{A-B}) \tag{8}$$

where $t_{A-B} = L_{AB} / c$ denotes the time for the stress wave travelling from point A to B. The strain wave in the negative direction at point B consists of two parts; compare Figure 8b. The first part is the primary reflection in the boundary between the incident bar and the test specimen. The second is a strain wave in the negative direction from the test specimen, which can be derived from the strain in point D. This can be expressed as

$$\varepsilon_B^-(t) = \varepsilon_A(t - t_{A-B} - 2\ \ t_{B-C})\ \ R_{12} + \varepsilon_D^-(t - t_{B-C} - t_{C-D})\ \ T_{21}\frac{c_2}{c_1} \tag{9}$$

where the reflection and transmission coefficients are defined according to (6). Note the order of the subindices. The total strain in point B is obtained by summation of (8) and (9). The total strain in point B is measured, however, and ε_D^- can therefore be expressed as

$$\begin{aligned}\varepsilon_D^-(t) = {} & \frac{1}{T_{21}}\frac{c_1}{c_2}[\varepsilon_B(t + t_{B-C} + t_{C-D}) - \\ & -\varepsilon_A(t - t_{A-B} + t_{B-C} + t_{C-D}) - \varepsilon_A(t - t_{A-B} -\ \ t_{B-C} +\ \ t_{C-D})\ \ R_{12}]\end{aligned} \tag{10}$$

where a shift of $t_{B-C} + t_{C-D}$ is performed in the time scale.

Hence the strain wave in the negative direction at point D can directly be derived from measured strains in points A and B. The strain wave in the positive direction at point D consists of two parts as well; compare Figure 8b. The first part is the primary transmission from the incident bar into the test specimen. The second is a reflection of the strain wave in the test specimen that reflects in the boundary between test specimen and Hopkinson bar, which can be expressed as

$$\varepsilon_D^+(t) = \varepsilon_A(t - t_{A-D})\ \ T_{12}\frac{c_h}{c_s} + \varepsilon_D^-(t - 2\ \ t_{C-D})\ \ R_{21} \tag{11}$$

By insertion of (10) we finally achieve

$$\begin{aligned}\varepsilon_D^+(t) = {} & \varepsilon_A(t - t_{A-D})T_{12}\frac{c_h}{c_s} + \frac{R_{21}}{T_{21}}\frac{c_1}{c_2}[\varepsilon_B(t + t_{B-C} - t_{C-D}) - \\ & -\varepsilon_A(t - t_{A-B} + t_{B-C} - t_{C-D}) - \varepsilon_A(t - t_{A-D})R_{12}]\end{aligned} \tag{12}$$

Thereby the strain in point D is expressed in (measured) strains in points A and B. If the material is elastic just close to the failure location, the stress can be determined as

$$\sigma_D(t) = E_2\varepsilon_D(t) = E_2\left(\varepsilon_D^+(t) + \varepsilon_D^-(t)\right) \tag{13}$$

where the Young modulus of the test specimen can be determined by evaluation of the acoustic speed in the specimen. Figure 10 shows measured and calculated strains in point D for a specimen of aluminium. The agreement is reasonably good for the initial part of the compression and tension waves.

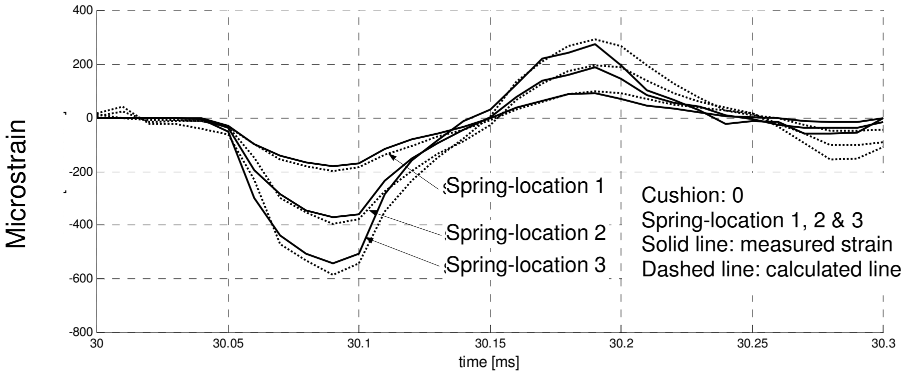

Figure 10. *Strain in point D, measured and calculated from measured strain at points A and B. Three different impact levels, i.e. spring-locations 1, 2 and 3 with no cushions.*

For specimens of elastic material, the two methods outlined above theoretically give equivalent results. Disturbance in the measurement and variations in assumed parameters such as wave speed imply deviations, but in summary the agreement was good. However, for testing of brittle material at a stress level above tensile strength, the methods of course deviate. In Figure 11, measured strains in points A and B are shown together with calculated strains in the specimen at a location where failure was localised. The two methods give very similar results for the initial compression wave. For the following tension wave, the agreement is good up to a certain strain level (~300 microstrain). Thereafter the two methods separate. Only the second method that includes measurements in both points A and B has the possibility to determine a failure, with adherent declined stress level.

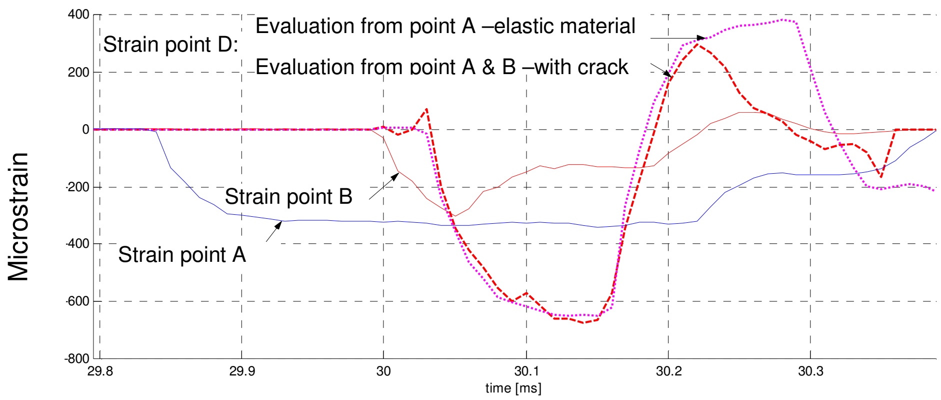

Figure 11. *Measured strain in points A and B together with calculated strain in point D for specimen 13B, determined according to the two methods outlined above.*

To evaluate the tensile strength, it is normally related to the present strain rate. Hence the strain rate was determined by differentiating the calculated strain. The calculated strains together with strain rates are shown in Figure 12. The maximum tensile stress and strain rate do not occur simultaneously, and it is therefore not obvious how to combine these results in an evaluation. In the present work, the maximum tensile stress is defined as the maximum calculated tensile stress, and the adjacent strain rate is defined as the maximum strain rate of the test.

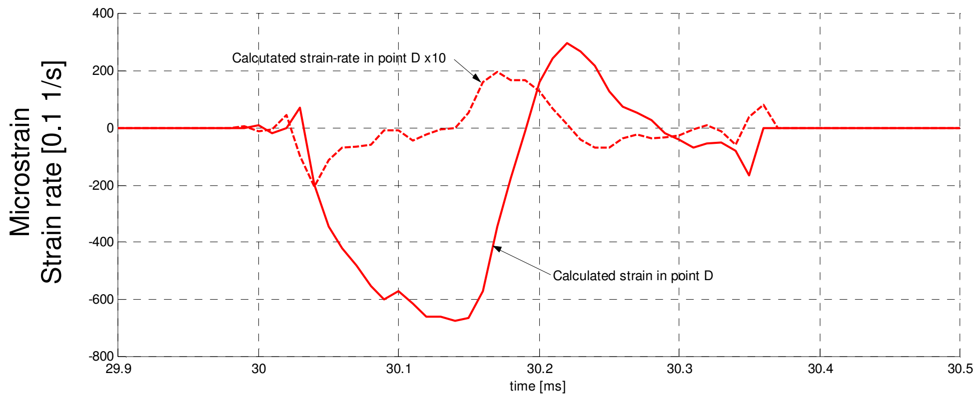

Figure 12. *Calculated strain and corresponding strain rate in point D for specimen 13B. Max. strain of 296 microstrains gives, with E=40 GPa, tensile strength 11.8 MPa. Max. strain rate 19.7 s^{-1}.*

2.6. Failure modes during testing

The proposed method to determine dynamic tensile strength is based on the assumption that an initial compression wave is reflected to a tension wave. To determine the ultimate stress a fundamental assumption was introduced that the *failure is caused by the initial tension wave*. To avoid tension waves of high amplitudes the projectile has to be of a certain length. For the first batch of specimens the length of the projectile according to Figure 5 was shorter than the length of the incident bar together with the adapter and specimen. During the evaluation of the test failures caused by secondary tensions, waves of high magnitudes could be identified. However, by increasing the length of the projectile the magnitude of the secondary stress wave was reduced significantly, and during evaluation of the second batch of specimens the primary tension wave caused the failure.

During some tests it was observed that failure occurred at a late stage and obviously neither the primary nor following tension waves cause the failures. In these cases it was likely that the specimen and incident bar received a vertical motion due to the pendulum support, and that failures arose due to stresses caused by bending. The behaviour of this failure was completely different from the pure tension failure and was easy to detect during the testing.

3 EXPERIMENTAL RESULTS

Of 33 specimens tested, 24 were evaluated according to the method outlined above. For the remaining specimens no results were achieved due to secondary bending failure or a lack of measured data.

The proposed method to evaluate stresses should be independent of the applied stress level. Hence it was of interest to change the stress level of the incident compression stress wave and compare the results. The velocity of the projectile determines the stresses in the incident bar, and by changing the prestressing level of the spring, which accelerates the projectile, two different stress levels were achieved.

The tensile strength is dependent on the present strain rate. Different strain rates were achieved by changing the loading rate of the incident bar when the projectile hit it. Two loading rates were applied. High loading rate was achieved when a hardened tip of the projectile directly hit the incident bar. By use of a soft ~2 mm thick plastic pad, which was applied as a cushion between the projectile and the incident bar, the loading rate was decreased.

In Figure 13, normalised tensile strength is shown as a function of strain rate. The equivalent static tensile strength is determined according to [9] based on measured compressive strength. In Figure 13 the normalised tensile strengths as a function of strain rate according to [10] and [9] are shown for reference.

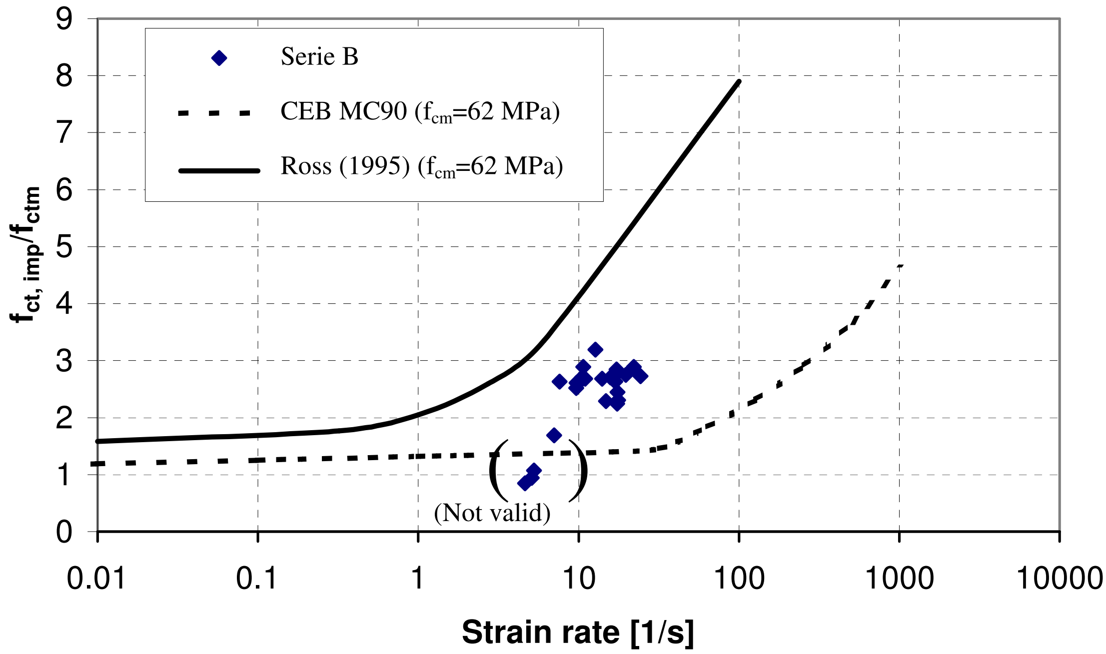

Figure 13. Summary of tensile strength as function of strain rate. For reference, tensile strength according to CEB-FIP MC 90 and Ross (1995) is shown.

The majority of the results are concentrated around a factor of 2.5–3.0 for the increase of the static strength. The corresponding strain rate lies between 10 and 25. However, some results show tensile strengths that are on the same level as static strength. In the results, different loading rates and stress levels are included. In Figure 14 the average of each group is shown. The group with low stress level (low velocity) and low loading rate (1 cushion) deviates from the remaining groups. Studying the measured and calculated strains for this group shows that the maximum stress level arises at a late stage, and it cannot be excluded that the measured strain signal in point A even includes a strain wave that propagates in the negative direction. To avoid this problem it is necessary to increase the length of the incident bar.

The results show that it was not possible to vary the strain rate over a wide range for the present test arrangement. By extending the length of the incident bar, tensile strength for lower strain rates can be obtained. However, the present strain rate level is pertinent for several applications, such as earthquakes, pile driving, etc.

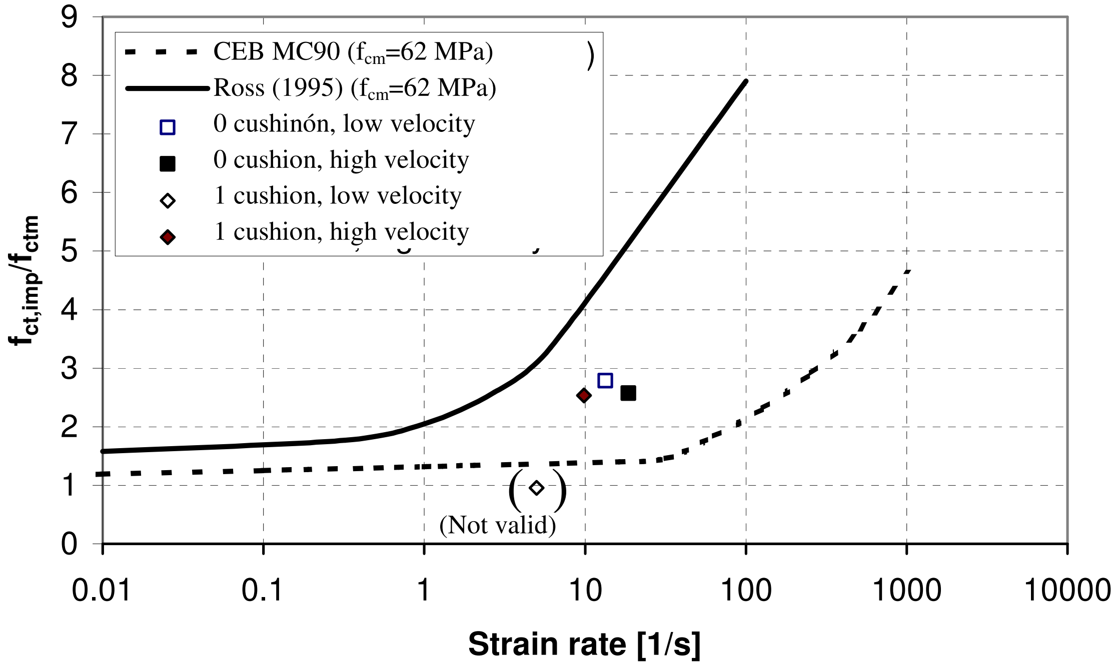

Figure 14. Summary of tensile strength as function of strain rate. Mean value for four different combinations of the testing parameters: numbers of cushions and velocity of the projectile.

The failure modes of the test specimen are shown in Figure 15. The specimens that are not included in the evaluation are marked with a solid line. The failure is normally localised close to the adapter. According to Figure 3, the resulting tensile stresses will arise due to reflection between incident bar and specimen (C), which the test results confirm.

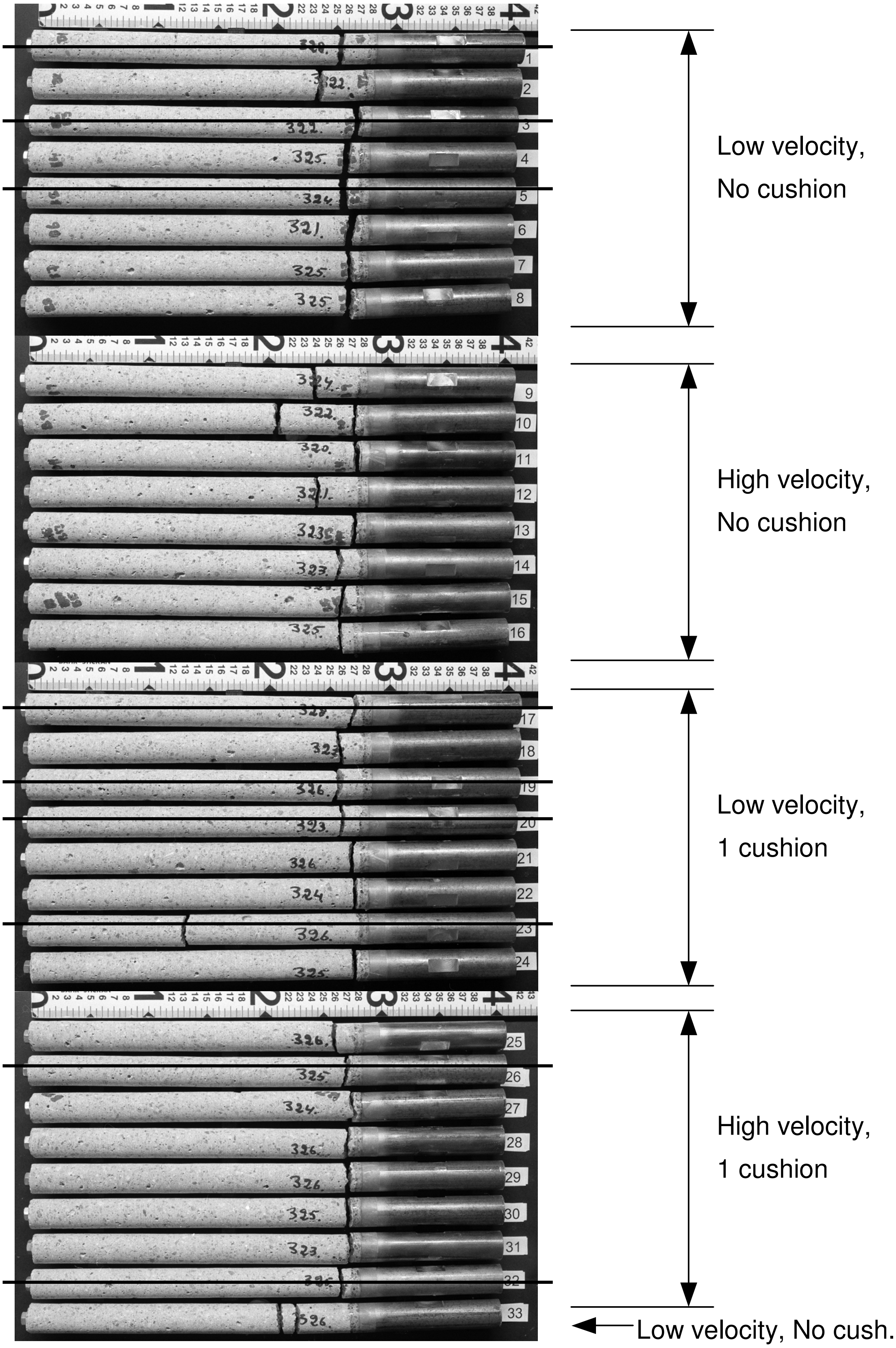

Figure 15. Specimens after testing (number on specimen is weight of specimen)

4 CONCLUSIONS

The Modified Hopkinson Pressure Bar (MHPB) has been used for determination of dynamic tensile strength of concrete. The MHPB is a straightforward method of direct determination of the tensile strength in concrete. Introducing a compressive stress wave requires only a simple test arrangement, and the resulting tensile stresses arise by reflections.

The tensile strength can be determined by analysing measured strains in two points of the incident bar. By using simple one-dimensional stress wave theory the ultimate stress and corresponding strain rate during fracturing of the brittle specimen can be determined. The validity of the proposed method of evaluation limits the lower boundary of the strain rates. For the present test arrangement, the applicable strain rates lie in an area that is of interest for many applications, such as pile driving and earthquakes. However, by changing the geometry of the test arrangement, the range of applicable strain rates can be increased.

ACKNOWLEDGEMENT

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Numerical Simulation of Projectile Penetration in Concrete

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ABSTRACT

This paper presents numerical simulations of concrete penetration by steel projectiles. To predict the penetration depth of the projectile and the crater size in the concrete, material models are required in which the strain rate effect, large deformations and triaxial stress states are taken into account. The analyses are made with AUTODYN, and the results of the analyses are compared with experimental data from the literature for the depth of penetration and the crater diameter. Two experimental series have been compared, with varying projectile weights and impact velocities; for both series, the depth of penetration was simulated well.

Key words: concrete, numerical simulation, projectile, impact, penetration.

1. INTRODUCTION

For protective structures, reinforced concrete has been the most widely used material. Protective structures of concrete have been built since the beginning of the 20th century. During and after World War II, there were large research projects for studying penetration effects on concrete.

For design of protective structures, the penetration by fragments and projectiles is a major concern; traditional empirical equations are used to predict the depth of penetration. In the literature there are empirical equations, such as, Bergman [1], Hughes [2], Forrestal *et al.* [3], and Chen and Li [4], to predict the depth of penetration for projectiles striking a concrete target. Although empirical equations give a good prediction of the depth of penetration, they do not describe the structural behaviour of the concrete structure. To improve the understanding of concrete subjected to severe loading, a combination of experiments and numerical methods is a powerful tool; it can be used for detailed analysis of the structural behaviour. This paper deals with examples of using numerical methods for projectile penetration.

The work reported here is a part of research project at Chalmers University of Technology, the long-term aim of which is to increase knowledge of concrete structures subjected to blast and fragment impacts. Chalmers has collaborated with the Swedish Rescue Services Agency, on earlier projects including non-linear finite element analyses of the blast loads and falling debris by Johansson [5], and studies of projectiles that penetrate concrete by Leppänen [6].

2. BEHAVIOUR OF CONCRETE UNDER DYNAMIC LOADING

2.1 Introduction

When a projectile or fragments hit a concrete target, the concrete crushes and cracks, and the structure shakes and vibrates. The pressure at the nose of the projectile is several times higher than the static uniaxial strength of concrete. This is due to strain rate and confining effects. In front of the nose of the projectile, the impact causes crushing. In addition, a stress wave propagates from the tip of the nose of the projectile. Since concrete is weak in tension, the tensile wave obtained when the compressive wave reaches the reverse side of a wall can cause scabbing there, and cracking in the lateral direction; when 50 % penetration is achieved, scabbing becomes a problem, according to Krauthammer [7]. Both the compressive strength and the tensile strength of concrete are important parameters for the depth of penetration and crater size.

2.2 Strain rate in concrete for uniaxial loading

The behaviour of concrete depends on the loading rate, known as the strain rate effect. The strain rate in the material is determined by the type of loading, as shown in Figure 1, for five kinds of loading, such as creep, static, earthquake, hard impact and blast loads.

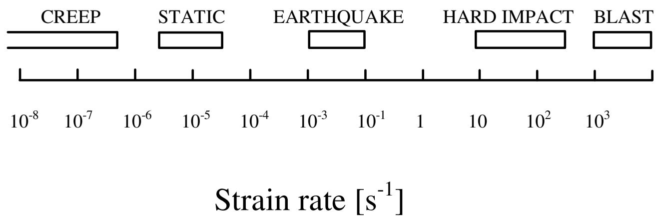


Figure 1 – Strain rates for five load types; based on Bischoff and Perry [8].

The strength, deformation capacity, and fracture energy are important parameters for characterizing and describing the response of concrete. For dynamic loading, these parameters are not the same as for static loading. When concrete is subjected to impact loading, the material strength increases. The dynamic increase factor (*DIF*) is the ratio between the dynamic ultimate strength and the static ultimate strength. In dynamic loading, the ultimate compressive strength can become more than double, see Bischoff and Perry [8]. Moreover, according to Ross *et al.* [9] the concrete ultimate uniaxial strength in tension rises by multiples of 5 to 7 at very high strain rates.

The greater strength is explained by a change in the fracture plane. As the loading rate becomes higher, concrete suffers multiple fractures and the amount of aggregate fracture also increases, see Zielinski [10]. Other explanations of the increased strength are the viscous effects and the forces of inertia. The viscous effects are explained by the following; when concrete is subjected to compressive loading, the pores tend to close. The pore water causes viscous effects, and develops an inner pressure in the pores that are filled with water, which augments the strength of the material. For concrete in tension, the resistance force arises when the pores are opening, see Rossi and Toutlemonde [11]. The *DIF* curve, as shown in Figure 2, calculated according to

CEB-FIB Model Code 1990 [12], has a flat part and a steep part: for the flat part, the viscous effects dominate, while for the steep part, the forces of inertia dominate. For concrete in tension, when the strain rate is less than approximately 1 s^{-1} the viscous effects dominate, and when the strain rate exceeds approximately 10 s^{-1} the forces of inertia dominate. For concrete in compression, the forces of inertia dominate at strain rates of approximately 60–80 s^{-1} according to Ross *et al.* [9].

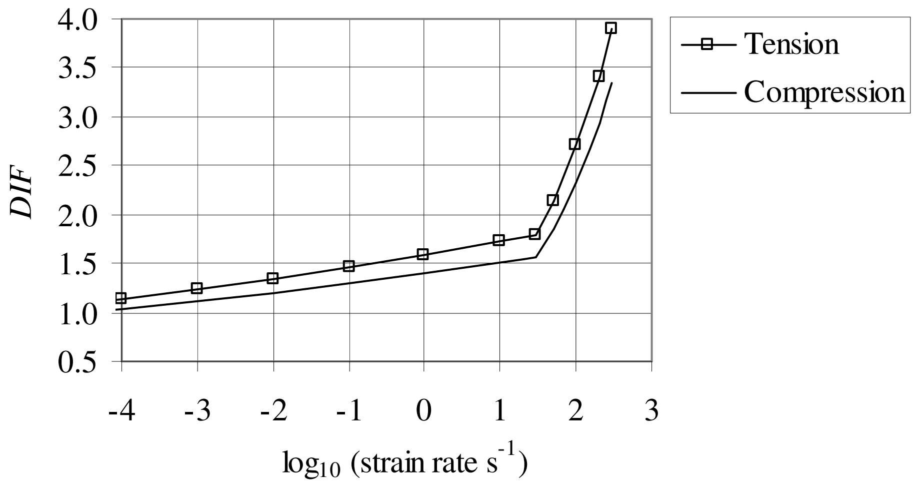


Figure 2 – DIF according to CEB-FIB Model Code 1990 [12].

2.3 Strain rate in concrete for confined concrete

The research on strain rate effects has been devoted mainly to uniaxial loading conditions. For multiaxial loading conditions, the relevant research has been done by Zielinski [10], Takeda *et al.* [13] and Weerheijm [14]. It was demonstrated in Takeda *et al.* [13] that the rate effects for confined concrete in compression resulted in same order of increase in strength at low compression levels. However, the strain rates that were used in those experiments were relatively low, i.e. up to 1 s^{-1}.

Zielinski [14] made a series of tests for which the loading condition consisted of uni-axial static or impact loading and a lateral confining pressure. In the static tests, the axial tensile force was gradually raised to failure; the rate of loading was approximately 0.1 $N/mm^2/s$. In the impact tests, a drop-weight was used and the rate of loading was about 10^4 $N/mm^2/s$. The results show that, at all levels of lateral compression tested, the impact tensile strength of concrete was higher than for the static load. However, the ultimate tensile strength of concrete was hardly affected by lateral compression less than 0.7 of the concrete cylinder strength. Furthermore, for high static lateral compression, the strains become greater for both static and impact tensile loading. For low static lateral compression, the strains are barely affected, see further Zielinski [15].

Since concrete members are in a multiaxial stress state during the penetration, it is important to describe the material behaviour in these conditions. However, experiments with multiaxial loading are limited to relatively low strain rates. To learn more about the dynamic behaviour, there is a need for experiments in multiaxial loading at higher strain rates.

3. NUMERICAL PROGRAM AUTODYN

3.1 General

The development of computers in recent decades has made it possible to use numerical methods for severe dynamic loading, such as blast waves, or for penetration analyses of concrete. Hydrocode is a code for solving variety of problems with large deformations, and transient problems that occur on a short time, see further Benson [16]. The code combines finite difference, finite volume, and finite element techniques, see further AUTODYN [17]. In hydrocodes there are two main descriptions for the material movement, i.e. the Lagrangian and Eulerian descriptions, as shown in Figure 3. There are other descriptions for the material movement, such as ALE (Arbitrary Lagrange Euler) and SPH technique, which are not discussed in this paper, see further AUTODYN [17]. In the Lagrangian description, the numerical mesh distorts with the material movement. In the Eulerian description, the numerical mesh is fixed in space, and the material moves in the elements. To allow the material movement, the fixed numerical mesh is larger than the structure analysed.

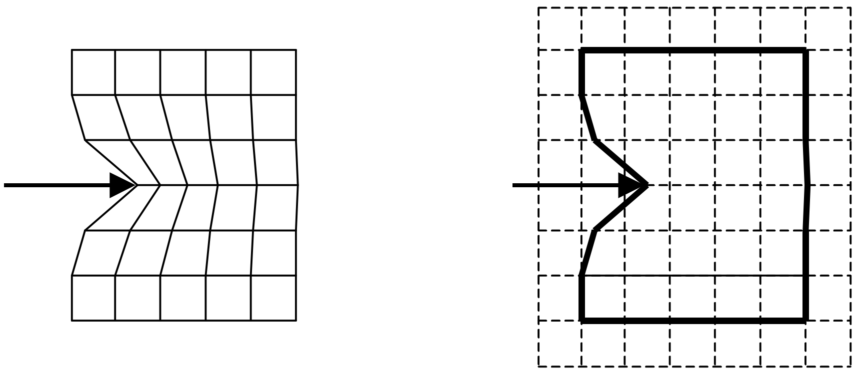


Figure 3 – The Lagrangian description (left) and the Eulerian description (right) for material movement.

With large displacements, when using the Lagrangian description of the material movement, numerical problems arise from distortion and grid tangling of the mesh. This leads to loss of accuracy and the time steps become smaller or even terminate the calculation. To overcome the numerical problems, a rezoning or erosion algorithm can be used. Rezoning transforms the numerical mesh being used into a new one. With great distortion or grid tangling, an erosion algorithm must be used to continue the calculation. Erosion is defined as the removal of elements from the analysis when a predefined criterion is reached; normally this criterion is taken to be the plastic strains. With the erosion algorithm, a non-physical solution is obtained because of mass reduction, which means that internal strain energy is removed from the system.

The advantage with Eulerian method is that no erosion algorithm is needed, since the material moves in the elements; and physical solutions can be obtained. However, Eulerian method is more computationally expensive.

The governing equations in AUTODYN are: conservation of mass, momentum and energy. To complete the description of the continuum, two additional relations describing the material behaviour are required (besides the load and boundary conditions): first the equation of state (*EOS*), and second a constitutive model.

3.2 The equation of state, *EOS*

The *EOS* relates the pressure to the local density (or specific volume) and the local specific internal energy of the material, according to the general form

$$p = p(\rho, e) \tag{1}$$

where ρ is density and e is specific internal energy.

In finite element programs used for static analysis, a constitutive model without any explicit description of the *EOS* normally describes the material behaviour. For these programs at high hydrostatic pressures (all principal stress components are equal), the material behaviour is linear (if the model has no cap combined with the original yield surface). For severe loading, e.g. explosion or penetration into concrete, the hydrostatic pressure levels are so high that the non-linearity of the material behaviour must be taken into account.

When hydrostatic pressure is applied to concrete, the relationship between hydrostatic pressure and density becomes non-linear at a given pressure level as shown in Figure 4. The pressure–density relationship can be divided in three regions, see Holmquist and Johnson [18]. Initially, for low-pressure levels, the relationship between pressure and density is linear (elastic loading). With further loading, microcracking occurs in concrete. Since concrete is porous, the pores collapse and the material is compacted; this is termed the plastic compaction phase. At very high pressure levels, when the concrete is fully compacted (all pores are collapsed), the relationship between pressure and density becomes linear again.

The *EOS* used in the analyses (Section 4) is a combined P-Alpha and a polynomial *EOS*. The P-Alpha *EOS* (in P-Alpha the plastic compaction phase is ten-point piecewise linear; P stands for pressure, and Alpha is defined as the current porosity) defines the starting point for plastic compaction, and the polynomial *EOS* defines the compaction phase. In Figure 4 the initial density, ρ_0, is the undisturbed concrete density, and the solid density, ρ_s, is defined as the density at zero pressure of the fully compacted solid. The material behaves elastically until the initial compaction pressure, p_{crush}, is reached; thereafter the plastic compaction phase takes place.

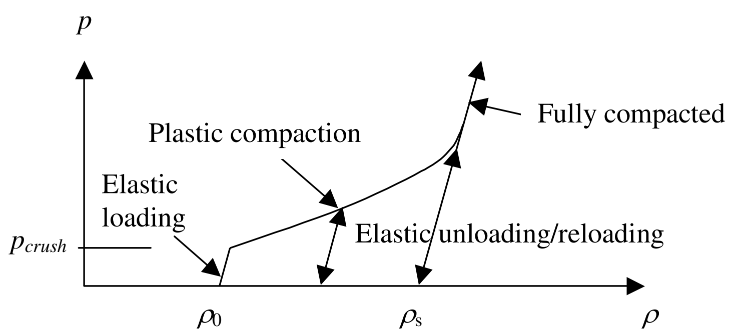
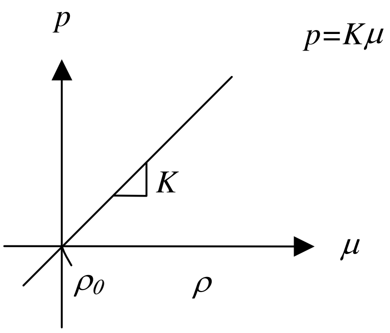


Figure 4 – Equation of state (EOS), for concrete, combined P-Alpha and polynomial; based on AUTODYN [17].

For hydrostatic pressure, steel compression is approximately proportional to the pressure level. Thus, a linear *EOS* for steel (the projectile) is used. The pressure level is dependent on the bulk modulus, *K*, and the compression, μ (ρ = density), as shown in Figure 5. Furhtermore, von Mises material model has been used in the analyses for the projectile in Section 4.



$$\mu = \frac{\rho}{\rho_0} - 1$$

Figure 5 – Equation of state for steel; based on AUTODYN [17].

3.3 The RHT model for Concrete

The constitutive model used in the analyses with AUTODYN here is the RHT model (Riedel, Hiermaier and Thoma), developed by Riedel [19], as shown in Figure 6. Here, a short summary of the model is given. For detailed description of the material model, see Riedel [19] or AUTODYN [17]. The model includes pressure hardening, strain hardening, strain rate hardening, third-invariant dependence for compressive and tensile meridians, and a damage model for strain softening. It consists of three pressure-dependent surfaces: an elastic limit surface, a failure surface, and a surface for residual strength. The elastic limit surface limits the elastic stresses and the hardening is linear up to peak load.

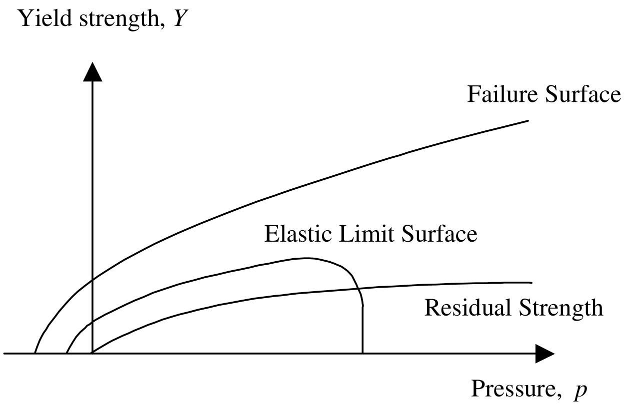


Figure 6 – The RHT model used for concrete; based on Riedel [19].

The failure surface is defined as

$$f(p, \sigma_{eq}, \theta, \dot{\varepsilon}) = \sigma_{eq} - Y_{TXC(p)} F_{CAP(p)} R_{3(\theta)} F_{RATE(\dot{\varepsilon})}. \quad (2)$$

The pressure dependency is defined as

$$Y_{TXC} = f_c \left[A(p^* - p^*_{spall} F_{RATE})^N \right] \tag{3}$$

where A and N define the form of the failure surface as a function of pressure, p^* is the pressure normalized by f_c, and p^*_{spall} is defined as $p^*(f_t/f_c)$. The failure surface is a function of the pressure and the strain rate. The third-invariant dependence is included in the failure surface with a function $R_{3(\theta)}$, which defines the transfusion from the compressive meridian to tensile meridian and stress states between these. Furthermore the model has a function, $F_{CAP(p)}$, which limits the elastic deviatoric stresses under hydrostatic compression. The rate dependency in the yield surface is defined as

$$F_{RATE} = \begin{Bmatrix} \left(\dfrac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right)^{\alpha} & for\ p > 1/3\ f_c\ (compression), \dot{\varepsilon}_0 = 30 \cdot 10^{-6}\ s^{-1} \\ \left(\dfrac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right)^{\delta} & for\ p < 1/3\ f_t\ (tension), \dot{\varepsilon}_0 = 3 \cdot 10^{-6}\ s^{-1} \end{Bmatrix} \tag{4}$$

where α is the strain rate factor for compression and δ is the strain rate factor for tension.

When the failure surface is reached, the softening phase starts, and continues until the residual strength surface is reached. The residual strength surface is defined by parameters B and M, and is a function of the pressure level as by

$$Y^*_{residual} = B \times p^{*M} . \tag{5}$$

4. NUMERICAL MODELLING OF CONCRETE PENETRATION

4.1 The experimental series

To ensure that a numerical model can predict the depth of penetration and crater size, results from more than one experiment must be reproduced. In this work two experimental series with a total of 6 shots were compared. Analyses with AUTODYN by using the RHT model for the concrete target were made. Two experimental series were compared with numerical analyses: first, a 6.28 kg projectile striking a concrete cylinder at a velocity of 485 m/s, experiments by Hansson [20]; and second experiments with a 0.906 kg projectile striking a concrete cylinder, the results of four striking velocities from 277 m/s to 800 m/s are compared with experiments by Forrestal *et al.* [3]. For the first experimental series, both the Lagrangian and Eulerian methods were used, while for the second experimental series, only the Lagrangian method was used for the numerical analyses.

The heavy steel projectile
In the experimental series reported by Hansson [20], the 6.28 kg ogive-nose steel projectile used had a length of 225 mm, diameter of 75 mm, density of 7 830 kg/m^3, bulk modulus of 159 GPa, shear modulus of 81.8 GPa, and yield stress of 792 MPa.

The target was a concrete cylinder, cast in a steel culvert, with a diameter of 1.6 m and a length of 2 m. The concrete cube strength was approximately 40 MPa (tested on a 150 mm cube). Two shots were fired at the same impact velocity, the first with support and the second without support at the opposite end of the target; the results are shown in Table 1.

Table 1 – Data summary for the experiments with 6.28 kg projectile striking a concrete cylinder. After Hansson [20].

Striking velocity (m/s)	Projectile mass (kg)	$f_{c,cube}$ (MPa)	Depth of penetration (m)
485	6.28	40	0.655 - 0.660 [a]

a. Two shots were fired, first with support and second without support at the reverse side of the target.

The light steel projectile
In the series reported by Forrestal *et al*. [3], ogive-nose projectiles comprising from 4 340 steel rods and heat-treated to a hardness of R_c 43 - 45 were used. Moreover, filler material was used in the projectiles, with a density of 1 580 kg/m^3. The projectile length, *l*, was 242.4 mm, the diameter, *d*, was 26.9 mm, and the ogival radius, *s*, was 53.8 mm.

The concrete targets were cast, in galvanized corrugated steel cylinders, with a diameter of 1.37 m and target length of 0.76 m. The shots had striking velocities of 277 m/s and 499 m/s. For two other experiments with impact velocities of 642 m/s and 800 m/s, the target diameter was 1.22 m and the length 1.83 m. The concrete had a density of 2 370 kg/m^3, and the unconfined uniaxial compressive cylinder strength varied between 32.4 MPa and 35.2 MPa. The four experiments with the 0.906 kg projectile are compared in this paper, and their results are summarized in Table 2.

Table 2 – Data summary for the experiments with 0.906 kg projectile striking a concrete cylinder. After Forrestal et al. [3].

Striking velocity (m/s)	Projectile mass (kg)	f_c (MPa)	Depth of penetration (m)
277	0.906	35.2	0.173
499	0.912	33.5	0.480
642	0.905	34.7	0.620
800	0.904	32.4	0.958

4.2 Mesh descriptions

It is well known that the size of a numerical mesh affects the results, and that a refined mesh extends the computational time dramatically. For dynamic loading, the mesh dependency is even more important, since more terms are added to the constitutive models (the strain rate effect). Johansson [5] studied the mesh dependency by comparing static and dynamic loading. He concluded that, when the strain rate effect was included in a constitutive model, the general behaviour changed considerably. Since, the strain rate depends on the numerical mesh, the increase in dynamic strength is also mesh-dependent.

To assess the mesh dependency, the common method is to halve the mesh and compare the first coarse mesh with the halved finer mesh; if the results differ only negligibly, the analyst is satisfied. In numerical analyses with dynamic loading, it is necessary to use several meshes to ensure the accuracy of the results. Moreover, changing a mesh size in the structure must be done with great care. When Zukas and Scheffler [21] made a study on the effects of meshing, they concluded that, for accuracy, there should be at least three elements across the radius of the projectile.

The mesh dependency was chosen, by starting, as a rule of thumb, with three elements across the radius of the projectile, after which the mesh was further refined. In this paper, only the final mesh is presented; for further details, see Leppänen [6]. The numerical mesh 1 is shown in Figure 7, which is used to analyse the experiments by Hansson [20] in Section 4.4. The target is of concrete, cast in a steel cylinder. The model is axisymmetric, formed by quadratic elements with an element length of 6.25 mm, totaling 128 x 320 elements (for the target).

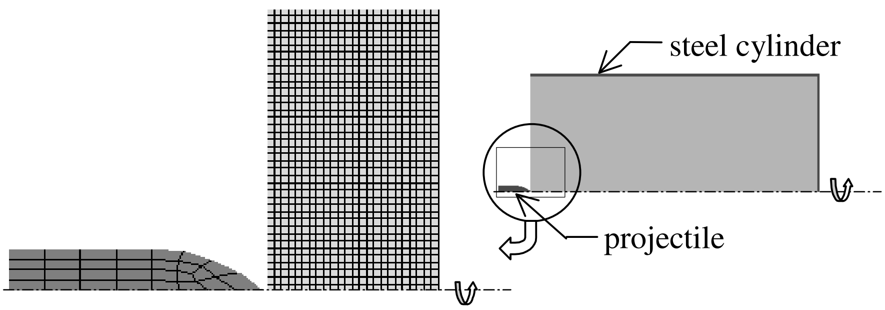


Figure 7 – Numerical mesh 1, 6.28 kg projectile.

Numerical mesh 2 is shown in Figure 8, which is used to analyse the experiments by Forrestal *et al*. [3] in Section 4.4. The target is of concrete, cast in a galvanized steel cylinder. The model is axisymmetric, generated by rectangular elements with an element length of approximately 4 mm. For a target length of 0.76 m, the mesh size is 190 x 172 elements as seen in the figure, while for a target length of 1.83 m, the mesh size is 153 x 458 elements.

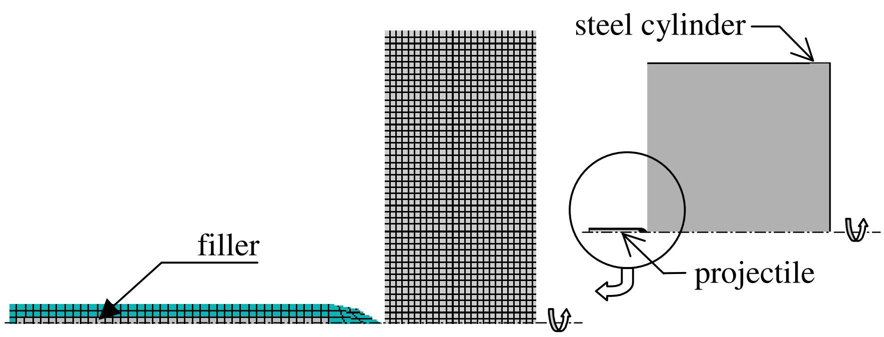


Figure 8 – Numerical mesh 2, 0.906 kg projectile.

4.3 Analyses with AUTODYN

A lack of data of dynamic material properties makes selection of parameters difficult and crucial. In this section, description of, and discussion on choosing the proper material parameters is given.

The *EOS* used in the numerical model combines a P-Alpha *EOS* with a polynomial one, see Figure 4. The material parameters are given in Table 3. Detailing of the material parameters is described in AUTODYN [17]; here, the compaction phase (polynomial *EOS*) is chosen to have the default values from the material library in AUTODYN. In the experimental series with 6.28 kg projectile, the density of the concrete is assumed to be 2 400 kg/m^3.

Table 3 – Input data for modelling concrete: RHT model, equation of state (EOS).

Parameter	Value
Porous density (g/cm^3)	ρ_0 [a]
Porous sound speed (m/s)	2920
Initial compaction pressure (kPa)	$2.33 \cdot 10^4$
Solid compaction pressure (kPa)	$6 \cdot 10^6$
Compaction exponent n	3
Solid *EOS*:	Polynomial
Compaction curve:	Standard
A1 (kPa)	$3.527 \cdot 10^7$
A2 (kPa)	$3.958 \cdot 10^7$
A3 (kPa)	$9.04 \cdot 10^6$
B0	1.22
B1	1.22
T1 (kPa)	$3.527 \cdot 10^7$
T2 (kPa)	0

a. 2 400 g/cm^3 for the experiments with 6.28 kg projectile, and 2 370 g/cm^3 for the one with 0.906 kg projectile.

The constitutive model used in the study is the RHT one shown in Figure 6 and described in Section 3.3. The material parameters of the concrete are shown in Table 4. Parameters A and N describe the failure surface (compressive meridian), see equation (3). From knowledge of the concrete behaviour in tri-axial stress states, the parameters can be determined. In the work reported here, the parameters used are calculated according to the model proposed by Attard and Setunge [22], as shown in Figure 9, for low confining pressures. However, parameters A and N used in the work here fit the experimental data presented by Bažant *et al.* [23], with f_c = 46 MPa, see Figure 9. In the experiments compared here, the ultimate uniaxial strength is approximately 34 MPa. Since, the pressure in the model is normalized by f_c, it is assumed that the behaviour is similar for the lower strength concrete.

Table 4 – Input data for modelling concrete: RHT model, constitutive model.

Parameter	Value	Comments
Shear Modulus (kPa)	$1.433 \cdot 10^7$	b
Compressive Strength f_c (MPa)	f_c [a]	b
Tensile Strength f_t/f_c	0.078	b
Shear Strength f_s/f_c	0.18	(default)
Failure Surface Parameter A	2	c
Failure Surface Parameter N	0.7	c
Tens./Compr. Meridian Ration	0.6805	(default)
Brittle to Ductile Transit.	0.0105	(default)
G(elas.)/G(elas-plas.)	2	(default)
Elastic Strength/f_t	0.7	(default)
Elastic Strength/f_c	0.53	(default)
Use Cap on Elastic Surface	Yes	(default)
Residual Strength Const. B	1.5	c
Residual Strength Exp. M	0.7	c
Comp. Strain Rate Exp. α	0.032	(default)
Tens. Strain Rate Exp. δ	0.025	d
Max. Fracture Strength Ratio	$1 \cdot 10^{20}$	(default)
Damage constant D1	0.04	(default)
Min. Strain to Failure	0.01	(default)
Residual Shear Modulus Frac.	0.13	(default)
Tensile Failure model	Hydro Tens.	(default)
Erosion Strain/instantaneous geometric strain (Lagrange)	1.25 and 1.4	d

a. 33.8 MPa for the experiments with 6.28 kg projectile, and see Table 2 for the one with 0.906 kg projectile.
b. Calculated according to CEB-FIB Model Code 1990 [12].
c. Calculated with model proposed by Attard and Setunge [22].
d. Calibrated by parameter studies, see Leppänen [6].

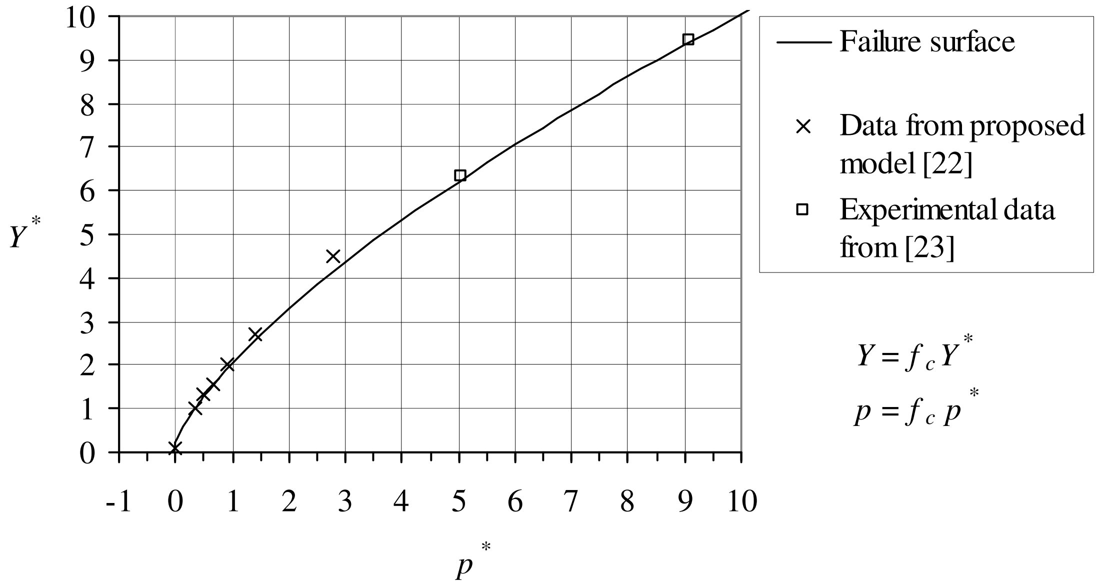


Figure 9 – Failure surface (static, and compressive meridian). Y^* and p^* are normalized by f_c.

The residual strength of the concrete, as shown in Figure 10, is calculated on the basis of the model proposed by Attard and Setunge [22]. The experiments and model which they proposed are for static loading with confinement pressure varying between 1 and 20 MPa. In the severe loading example analysed here, the confining pressure exceeds the range of those given by Attard and Setunge. Furthermore, it is not obvious that the residual strength is equal for both dynamic and static loading. There are no experimental results (according to the author's knowledge) on the residual strength for dynamic loading. However, the Attard and Setunge model indicates the level of the residual strength, which was used here in the numerical analyses.

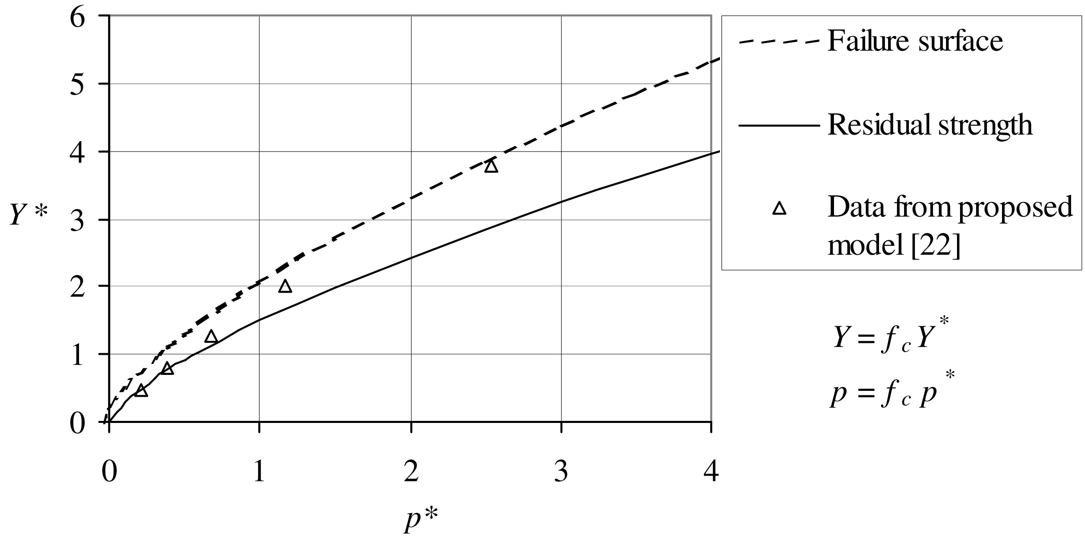
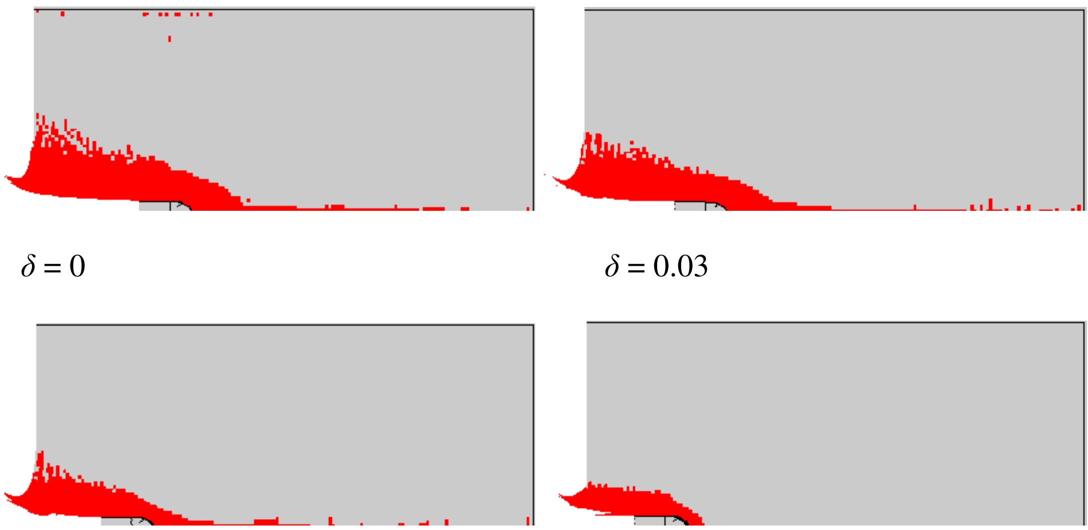


Figure 10 – Residual strength of concrete. Dotted line is the failure surface.

In the experimental series by Hansson [20], the tested concrete cube strength was 40 MPa. However, the cylinder strength is used as input to the material model chosen here, which is calculated from the cube strength according to the CEB-FIB Model Code 1990 [12]. In addition, the CEB-FIB Model Code 1990 is used for calculating the material parameters, for example the shear modulus or tensile strength.

The RHT model, which captures the rise in strength in compression, caused by increasing strain rate, was adapted to results published by Bischoff and Perry [8]. During the penetration of the projectile, the concrete is compressed in both the longitudinal and radial directions. The compression in the radial direction causes a tensile ring to be formed around the projectile which holds the concrete together; this is why the increase in tensile strength is important. Since the strain rate dependency for tension is uncertain, a phenomenological study has been performed by Leppänen [6], see Figure 11. It was found that the strain rate dependency was underestimated or overestimated, when the strain rate factor was varied from $\delta = 0$ up to $\delta = 0.11$, see equation (4). The results of the phenomenological study show that the strain rate dependency for tension has a huge effect on the maximum crater diameter. Moreover, if the strain rate dependency is underestimated or overestimated, the depth of penetration will be erroneous as well.



$\delta = 0$ $\delta = 0.03$

$\delta = 0.07$ $\delta = 0.11$

Figure 11 – Phenomenological study of the effect of the strain rate factor in tension on the crater size.

The tensile softening is limited to linear softening in the RHT model; this means that the concrete absorbs too much energy. To compensate for the energy absorbed, the strain rate dependency for tension was reduced, i.e. it was lower than in the experimental results reported in Ross *et al*. [9]. Hence, due to the uncertainty of the strain rate factor, δ, see equation (4), it was calibrated for the first experimental series, so that the crater diameter would agreed with the experimental result. Furthermore, the same strain rate dependency (factor δ) was assumed for the second experimental series (the concrete cylinder strength was of the same order).

4.4 Results

For the experimental results from Hansson [20], the crater size and depth of penetration were compared with both the Lagrangian and Eulerian methods. For the experimental results from Forrestal *et al*. [3], the depth of penetration was compared with the Lagrangian method; the numerical results for the crater sizes are shown. When using Lagrangian technique, erosion criterion must be used as described in Section 3.1. Here the erosion criterion is taken to be the instantaneous geometric strain.

Experiments with a 6.28 kg projectile striking a concrete cylinder

In the experiments two shots were fired, the first with, and the second without, support on the other end of the concrete cylinder; the depths of penetration were 655 mm and 660 mm, respectively. The crater diameter was approximately 0.8 m for both shots. For the numerical comparison without support on the far end of the cylinder, the depth of penetration was 636 mm with the Lagrangian technique (erosion criterion of 125 %), and 649 mm with the Eulerian technique, as shown in Figure 12. With the Eulerian technique, both the crater size and the depth of penetration agree very well with the experimental results. When the Lagrangian technique was used, with an erosion criterion of 125 %, the damage in front of the projectile was too deep. By increasing this criterion to 140 %, the crater size agrees with experiments, but the depth of penetration becomes 584 mm. However, the depth of the damage in the analysis corresponds to the experimental results. For numerical results with support at the far end of the cylinder, the depth of penetration was 627 mm with the Lagrangian technique and an erosion criterion of 125 %. With an erosion criterion of 140 %, the depth of penetration was 575 mm.

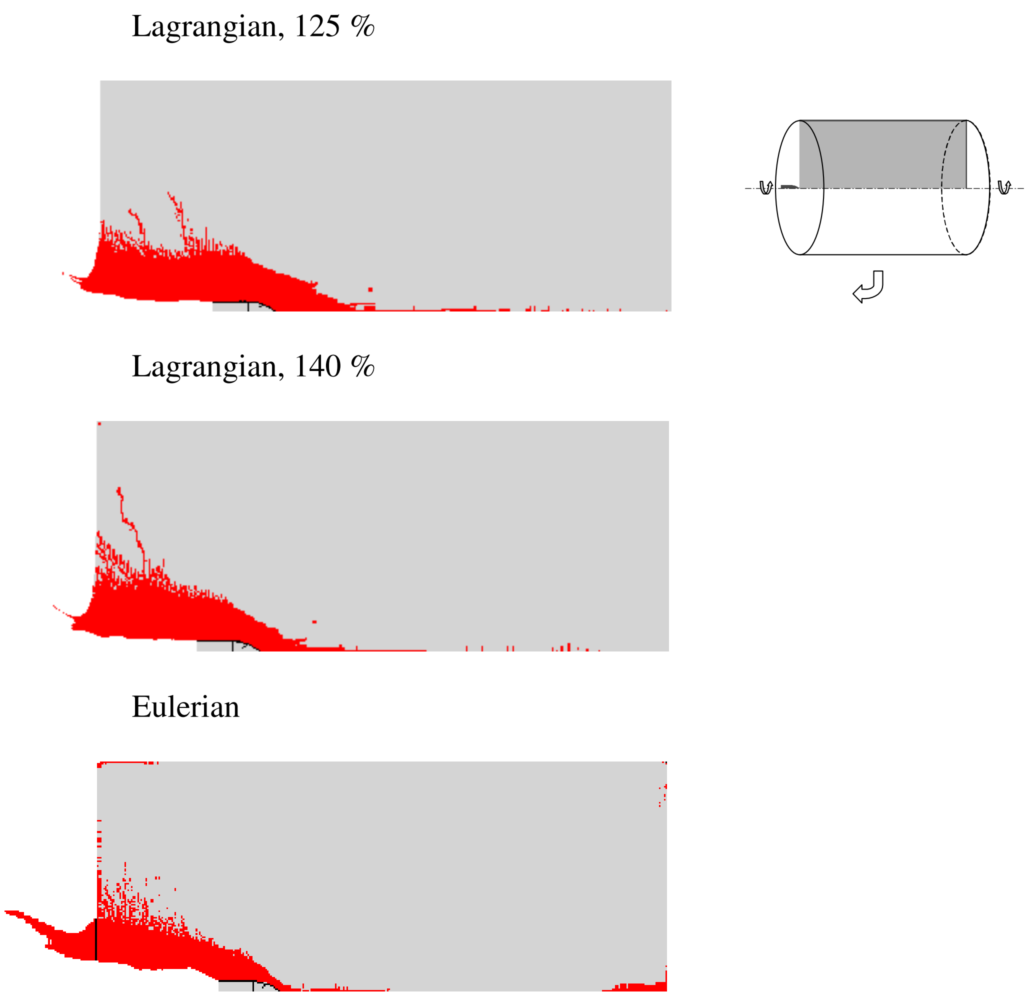


Figure 12 – Computed cratering and the depth of projectile penetration into concrete. Above: Lagrangian mesh: erosion criteria = 125 % and 140 %. Below: Eulerian mesh.

Experiments with a 0.906 kg projectile striking a concrete cylinder
A total of four experimental results were compared with experiments by Forrestal et al. [3], all for a projectile diameter of 26.9 mm, and with differing impact velocities. The results from the analysis are shown in Figure 13, where the depth of penetration is analysed with the RHT model for the impact velocities.

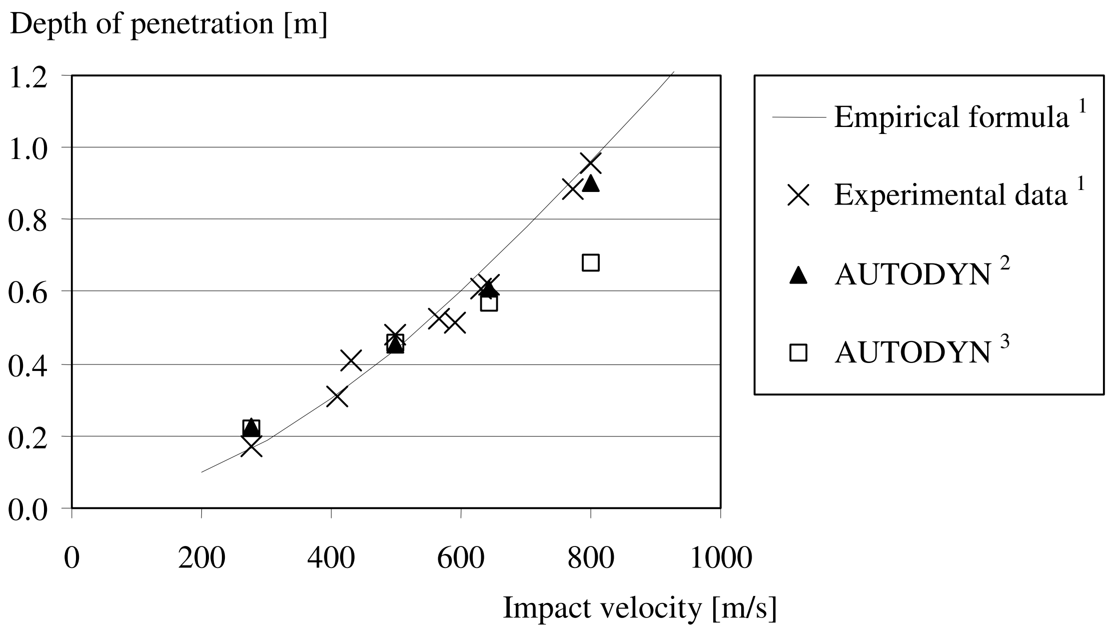
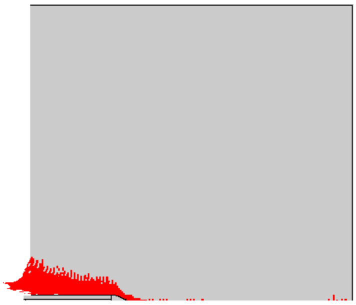


Figure 13 – Comparison of numerical results with experimental results [3].

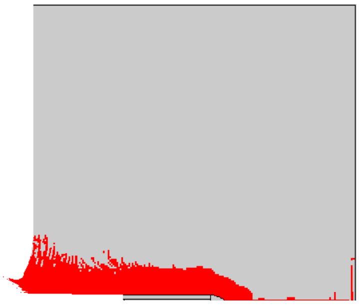
1 Forrestal et al. [3].
2 Yield strength of steel is 1 448 MPa (true ultimate strength).
3 Yield strength of steel is 972 MPa (true yield strength).

The projectile was made of steel with a hardness of R_c 43 – 45; the yield strength of this R_c 43 - 45 steel is 972 MPa, and the ultimate strength is 1 448 MPa. In the numerical model, a von Mises material model is used for the steel. Since the von Mises material model has no hardening in AUTODYN, the yield strengths of 972 MPa and 1 448 MPa were used in the analyses; this gives upper and lower limits of the strength of the steel (if the strain rate effect is neglected).

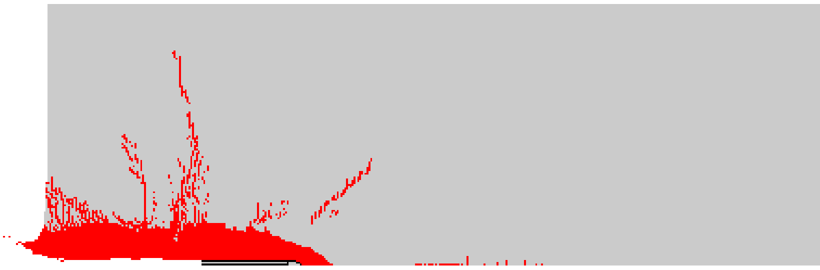
In Figure 14 the crater size and depth of penetration from the analyses are shown. The depth of penetration, maximum crater diameter and the lateral damage are greater for higher impact velocities. In these analyses the yield strength of the steel in the projectile was 1 448 MPa. The crater size is smaller for the light projectile than in experiments with the heavier projectile; the maximum crater diameter was between 0.20 and 0.40 m depending on the impact velocity.



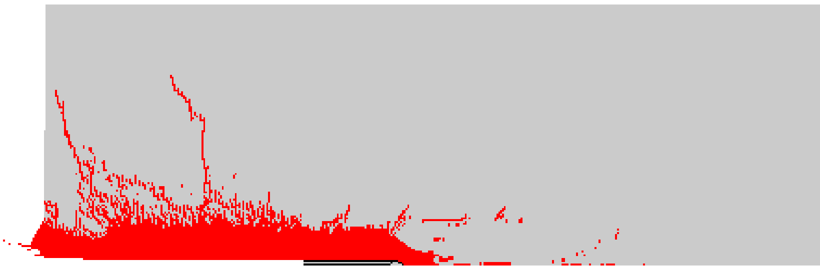
$V_s = 277$ m/s



$V_s = 499$ m/s



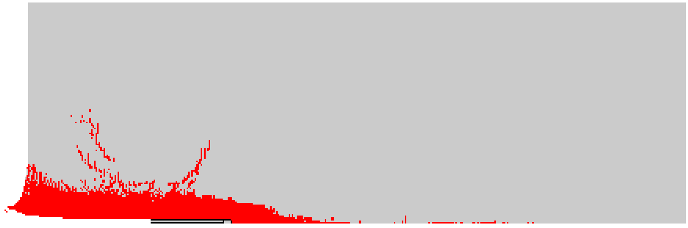
$V_s = 642$ m/s

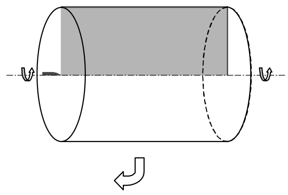


$V_s = 800$ m/s

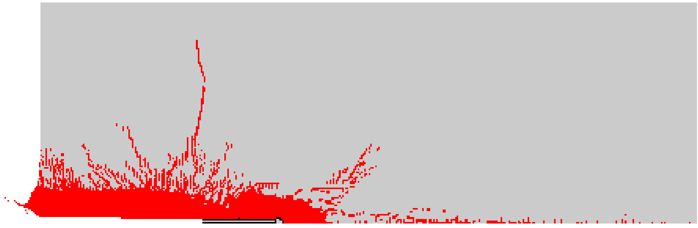
Figure 14 – Computed cratering and the depth of penetration of four projectile striking velocities. Lagrangian mesh: erosion criterion = 140 %. The yield strength for the steel of the projectile in the analyses was 1 448 MPa.

In analyses where the yield strength of the steel used for the projectile was 972 MPa, the results were very similar at low impact velocities, i.e. 277 m/s and 499 m/s. For the experiments with higher impact velocities, i.e. 642 m/s and 800 m/s, the depth of penetration is less when using the yield strength than when using the ultimate strength of steel in the analyses, see Figure 13. The results of the analyses are shown in Figure 15 for the two higher impact velocities.





V_s = 642 m/s



V_s = 800 m/s

Figure 15 – Computed cratering and the depth of penetration. Lagrangian mesh: erosion criterion = 140 %. The yield strength for the steel of the projectile in the analyses was 972 MPa.

For the higher impact velocities, the steel strength of the projectile is important. The projectile deforms when using the true yield strength of the material in the material model (von Mises), as shown in Figure 16. However, when using the ultimate strength of the steel, the projectile did not deform. Steel has, as does concrete, a strain rate dependency. In this paper, the strain rate dependency is not taken into account for the projectile. Therefore, when using a material model that does not take into account the hardening and strain rate effects, it is proposed that the true ultimate strength be used, instead of the true yield strength, as the yield strength in the material model (here von Mises, no hardening and no strain rate effects).

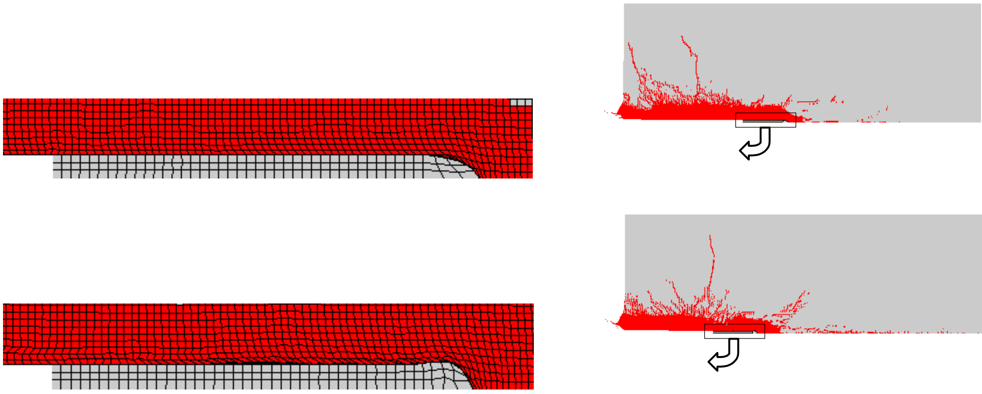


Figure 16 – Projectile deformations for two yield strengths of steel. Impact velocity of 800 m/s. Above: The yield strength for steel of the projectile in the analysis was 1 448 MPa. Below: The yield strength was 972 MPa.

4.5 Discussion

To achieve a reliable model, results from several experiments must be reproduced, for both the crater size and the depth of penetration. For example, accurate results for depth of penetration can be obtained by changing the residual strength or the erosion criterion (with the Lagrangian method). In this paper, numerical comparisons with experiments by Hansson [20] were made with both Lagrangian and Eulerian methods. The erosion criterion, which is the instantaneous geometric strain for Lagrangian analyses, was calibrated to fit the experimental results. This erosion criterion was then used for further comparison with another experimental series, in Forrestal *et al*. [3], with four impact velocities for the projectile. For these experiments, the projectile was modelled with the von Mises material model. Since this model has no hardening, the difference between the ultimate strength and the yield strength of the steel is so great that, by using the yield strength of the material, the depth of penetration becomes underestimated. Therefore, analyses using the ultimate strength as the yield strength in the model were also carried out. This gives lower and upper limits (if the strain rate effect is not taken into account) according to the strength of the steel in the projectile. At low impact velocities, the difference in depth of penetration is negligible, but for the higher impact velocities the higher steel strength is important as shown in Figure 13. In the experiments, non-deforming projectiles were used. As shown in Figure 16, in the analysis the projectile deforms when the increase in steel strength is not modelled. Hence, modelling the steel accurately, i.e. including hardening and strain rate effects in the material model, is important.

5. CONCLUSIONS

The behaviour of concrete changes under dynamic loading: the initial stiffness, as well as the ultimate strength, in both compression and tension, increase. Furthermore, the fracture of a concrete member changes under dynamic loading and multiple fracture planes are obtained.
When the behaviour of concrete under tri–axial stress states (failure surface, residual strength etc.) is known, and with sophisticated material models, such as the RHT model in AUTODYN, the depth of penetration and the crater size can be computed.

In this paper numerical analyses were compared with two test series, with different projectile weights and impact velocities; in all cases the depth of penetration was simulated well. The Eulerian method is preferable. With the Lagrangian method, by using the erosion algorithm, elements are removed from the model and, thus, also mass and strain energy, which yields non-physical results. However, the Lagrangian method is faster and may still give reliable results by using large erosion criteria.

The main material parameters that influence the depth of penetration are the concrete compressive strength, the strain rate dependency for compression, and the level of residual strength. The material parameters that have the most influence on the size of the crater are the tensile strength, the fracture energy and the strain rate dependency for tension.

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