Effects of the elevated temperature (+50 °C…140 °C) to the fastenings in concrete - A literature review

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Degree Programme in Construction Engineering | Abstract
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Concrete elements such as beams, columns, and walls, can be exposed for an extended period of time to elevated temperatures which will cause deterioration to the properties of concrete. This Bachelor's thesis aims to show the effect of elevated temperature on concrete and the effect on the capacity of the anchors. Extensive data shows that concrete will lose its compressive strength when subjected to heat. Compressive strength-to-temperature relationships should be taken into account when designing concrete fastener resistance. This thesis is commissioned by AFRY Finland Oy.

The background of this thesis is from SFS-EN 1992-4:2018 Design of concrete structures, - Part 4: Design of fastenings for use in concrete. In the standard, the effect of the elevated temperature is considered only for the case of bonded anchor, for this thesis cast in place anchor will be used.

This study is performed by a literature review, a collection of journals and studies from previous research will be reviewed. Based on the review conclusions will be drawn about whether the design capacity of the anchor should be reduced or not.

Keywords: Tensile resistance, embedded plates, elevated temperature, anchorage
Pages: 55 pages

Acknowledgment

The thesis topic is very interesting and it has wide application in the construction field. I would like to express my gratitude to Janne Ranz and Eija Lyytikäinen for allowing me to learn and better develop my skills on my path to becoming a structural engineer. This research would not be possible without Kimmo Ylinen, many thanks to him for providing this topic and for guiding and helping me throughout the research.

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Romeo Jr. Dacuno

List of symbols

$\mathrm{A}_{\mathrm{c,N}}$	is the actual projected area
$A^0{}_{c,N}$	is the reference projected of individual fastener area
$A_{c,v}$	is the area of the idealized concrete break-out body, limited by the overlapping concrete cones of adjacent fasteners
$A^0{}_{c,v}$	is the reference projected area of a single fastener
A_h	is the load-bearing area of the head of the fastener
c_1	edge distance in
f_{ck}	the characteristic value of compressive strength of concrete
f_{cd}	The design compressive strength
f_{ctm}	mean tensile strength
f_{ctd}	design tensile strength
$f_{ctk,0.005}$	5% characteristic axial tensile strength of concrete
$f_{c,\theta}$	compressive strength at specify temperature
h_{ef}	height of the cone
k_1	factor for cracked concrete cone failure
k_2	factor for fastener in cracked concrete pull-out failure
k_5	factor for cracked concrete blow-out failure
k_8	is a factor to be taken from the relevant European Technical Product Specification
k_9	factor for cracked concrete edge failure
$k_c(\theta)$	reduction factor of compressive strength
N_{Ed}	characteristic value of compressive strength of concrete

N_{Rd} design tension resistance

$N_{Rk,c}$ characteristic resistance in case of concrete cone failure under tension load

$N_{Rk.p}$ characteristic resistance in case of pull-out failure under tension load

$N_{Rk,cb}$ characteristic resistance in case of concrete blow-out failure

${N^0}_{Rk,c.Temp,M1}$ anchor tensile resistance exposed to temperature metho 1

${N^0}_{Rk,c.Temp,M2}$ anchor tensile resistance exposed to temperature method 2

${N^0}_{Rk,c}$ characteristic resistance of a single anchor

${N^0}_{Rk,cb}$ characteristic resistance of a single fastener for the concrete blow-out

${N^0}_{Rk.c,20\,°C}$ fastener resistance in normal temperature conditions T = 20 °C

R_b compressive strength

$R_{b,tem}$ compressive strength at specify temperature

T temperature

V_{Ed} design shear force

V_{Rd} design shear force resistance

$V_{Rk,cp}$ characteristic resistance in case of concrete pry-out failure under shear load

$V_{Rk,c}$ characteristic resistance in case of concrete edge failure under shear load

${V^0}_{Rk,c}$ characteristic concrete edge resistance of a single anchor

α_{cc} is the coefficient takes into account the long-term effect of compressive strength

α_{ct} is a coefficient taking account of long-term effects on the tensile strength

β normalized fastener capacity

µ normalized concrete tensile strength

γ_c the material safety factor of the concrete

γ_{bt} Coefficients of concrete working conditions in compression at concrete temperature

γ_{tt} Coefficients of concrete working conditions in tension at a concrete temperature

$\psi_{s,Nb}$ factor taking into account the disturbance of the distribution of stresses in the concrete due to the proximity of an edge in the concrete member in case of concrete cone failure

$\psi_{g,Nb}$ factor taking into account a group effect of a number of fasteners in a row parallel to the edge in case of concrete blow-out failure

$\psi_{ec,Nb}$ factor taking into account the group effect when different tension loads are acting on the individual fasteners of a group in case of concrete blow-out failure

$\psi_{s,N}$ factor taking into account the disturbance of the distribution of stresses in the concrete due to the proximity of an edge in the concrete member in case of concrete cone failure

$\psi_{re,N}$ is the shell spalling factor

$\psi_{ec,N}$ is the group effect when different tension loads are acting on the individual fasteners of a group in case of concrete cone failure

$\psi_{M,N}$ is the actor taking into account the effect of a compression force between the fixture and concrete in case of bending moments with or without axial force

$\psi_{s,V}$ is the factor takes account of the disturbance of the distribution of stresses in the concrete due to further edges of the concrete

$\psi_{h,V}$ is the factor takes account of the fact that the concrete edge resistance does not decrease proportionally to the member thickness

$\psi_{ec,V}$ is the factor takes into account a group effect when different shear loads are acting on the individual fasteners of a group

$\psi_{re,V}$ fastening in uncracked concrete and fastening in cracked concrete without edge reinforcement or stirrups

Contents

1 Introduction

1.1 Background of the study

Civil and structural engineering around the world is increasingly using modern fastening technology. Cast-in-place fastenings, which are put in the formwork before the concrete is poured, and post-installed fastening systems, which are put in concrete after it has hardened, are both widely used in construction. Anchor bolts transfer tension loads to the material they are attached to through mechanical interlock, friction, bond, or a mix of these three. No matter how the load is transferred, though fastening systems depend on the concrete tensile strength, this must be taken into account in both the design of the fastening and the design of the concrete member that is being held up by it. (Eligehausen et al., 2006, p. V)

Many studies have been made for designing concrete elements in fire situations. However, in the case of elevated temperatures, little research has been made. Figure 1 shows the headed studs which are welded to the plates, this assembly is called an embedded plate. Prior to concrete pouring these plates are installed in walls, floors, or ceilings to serve as an anchorage to other steel structures.

Figure 1. Embedded plates in different sizes. (Iter, 2013)

Every part of a fastening system is made to work best in a certain situation. When a fastening element is used for a purpose it was not designed for, it may not work as effectively as it should. To choose the right fastening system for a given application and to use the design of the fastening correctly, it is important to know how different fastenings behave. A fastener service life may include a variety of events that could influence both their structural integrity and behavior. It could manifest as corrosion, concrete deterioration, physical harm, chemical attacks, or unanticipated events like fire. (Eligehausen et al., 2006, p. V)

Most concrete constructions are typically only exposed to a temperature range that is no more than that imposed by ambient environmental conditions. But in some crucial situations, these structures might be subjected to even higher temperatures such as jet aircraft engine blasts, building fires, chemical and metallurgical industrial applications, in which the concrete is close to furnaces, and some nuclear power-related conditions. In these situations, whether or not the concrete will maintain its structural integrity may depend on how an increase in temperature affects specific mechanical and physical qualities. The thermal properties of concrete are more complicated than those of most materials because they depend on moisture and porosity, in addition to being a composite material with varied qualities. Concrete's mechanical and physical qualities are altered by exposure to high temperatures. A build-up of steam pressure could cause elements to warp and displace, and in certain circumstances, the concrete surfaces could spall. (Naus, 2005, p. 1).

1.2 Objective of the study

The objective of this thesis is to determine the effect of temperature on the capacity of the fastener and estimate the effect of temperature on the strength of concrete.

1.3 Limitation and assumption

This thesis is a review of related literature and no testing is done in this study. Most of the researchers who conducted the tests related to the effects of elevated temperatures were carried out using an electric furnace where the temperature ranged from 0 °C to over +1400 °C. However, this thesis will evaluate temperature ranges from +50 °C to +140 °C. Recycled and fresh aggregates are used for normal-strength concrete 20 MPa up to 50 MPa with long-term and cycle elevated temperature exposure. In this study, the effect of elevated temperature on cast-in-place headed anchors is examined.

2 Background of concrete anchorage

Anchorage is used to connect building components to the concrete structure. In concrete buildings, a variety of anchoring methods are routinely employed. These include, lifting inserts, anchor channels, and headed studs. The design of fasteners must ensure that they perform the intended function, are durable and robust, and have adequate load-bearing and deformation capacities. (Eligehausen et al., 2006, pp. 1-2)

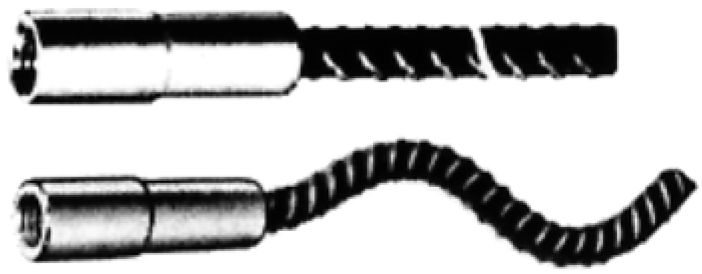
Fastenings that are important for life safety, whose failure could endanger life or cause considerable financial damage, must typically be chosen based on structural considerations and are typically designed and detailed by a structural engineer. The design determines whether the serviceability and ultimate limit state requirements are met. The serviceability limit state comprises limitations on deformation and durability criteria. At the ultimate limit state, it must be demonstrated that the design value of the actions does not surpass the design value of the fastening resistance ($N_{Ed} \leq N_{Rd}, V_{Ed} \leq V_{Rd}$). The optimal use of anchors is only possible if the design explicitly takes into account not only the direction of the load (tension, shear, combined tension, and shear), but also the modes of failure. A safe anchorage necessitates not only meticulous planning and design, but also anchor systems that function reliably in the field. (Eligehausen et al., 2006, p. 2)

Fasteners must align their placements with the reinforcing layout. They can also be easily put in structural parts that have been significantly strengthened. The benefit of cast-in-place systems is that because the location of the predicted external loads is known, the reinforced concrete member can be designed with the reinforcement in the correct places to account for it. The drawback of these systems is the additional layout and planning needed for them, as well as the possibility of incorrect placement. (Eligehausen et al., 2006, pp. 5–6)

Below is the list of the typical cast-in-place anchors in concrete buildings, these are various anchoring methods that are routinely employed.

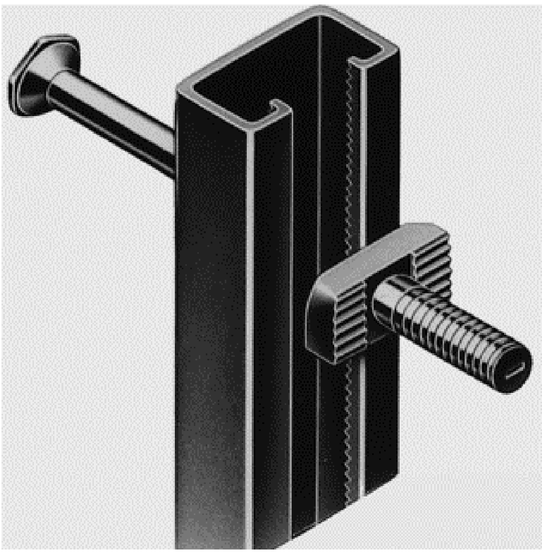
- Lifting inserts are used to move precast, and plain reinforced concrete components. To lift the concrete element from point to point, the lifting tackle is hooked in the threaded wire loop and screwed into lifting inserts with internally threaded sleeves. See Figure 2. (Eligehausen et al., 2006, p. 6)

Figure 2. Internally threaded lifting insert. (Eligehausen et al., 2006, p. 6)



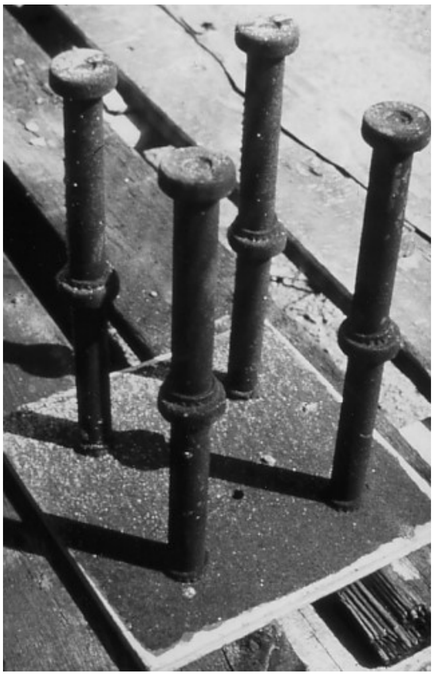
- An Anchor channel is a cold-formed or hot-rolled steel channel fitted with special anchor fittings. The Headed anchors are welded channels to transfer the load back to the concrete. After the concrete is cured, a T-bolt is inserted inside the channel and adjusted along the groove of the channel. After the T-bolt is adjusted to the specified distance, the external structure can now be attached and bolted into the T-bolt. See Figure 3. (Eligehausen et al., 2006, p. 7)

Figure 3. Anchor channel. (Eligehausen et al., 2006, p. 9)



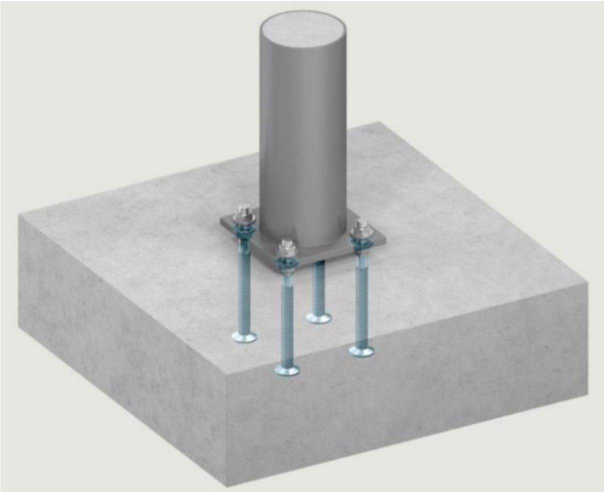
- An anchor plate with headed studs is made up of a steel plate with headed studs that are butt-welded. The steel plate is where the external structures are welded and
- transferring the load to headed studs and back to the concrete. See Figure 4 (Eligehausen et al., 2006, p. 9)

Figure 4. Steel-embedded plates with welded headed studs. (Eligehausen et al., 2006, p. 9)



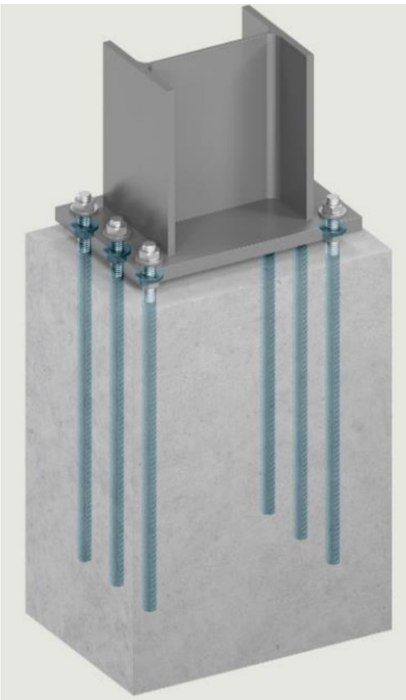
- An anchor bolt with a stud is used to connect concrete elements and steel frames to the foundation. The foundation bolt connection allows the transfer of axial, shear, and bending moment. The bolt comes in different sizes and lengths, and because of the very short length of the bolt, the application is suitable for a shallow foundation. The bolt diameter sized ranging from 16 mm up to 45 mm and the length is approximately 280 mm to 760 mm. The other end of the bolt is the head stud where the load is transferred back to the concrete. The headed stud should have enough edge distance and bolt length to prevent concrete edge failure and cone failure. See Figure 5. (Anstar rebar bolts, 2020, p. 6)

Figure 5. Anchor bolt with stud (Anstar rebar bolts, 2020, p. 4)



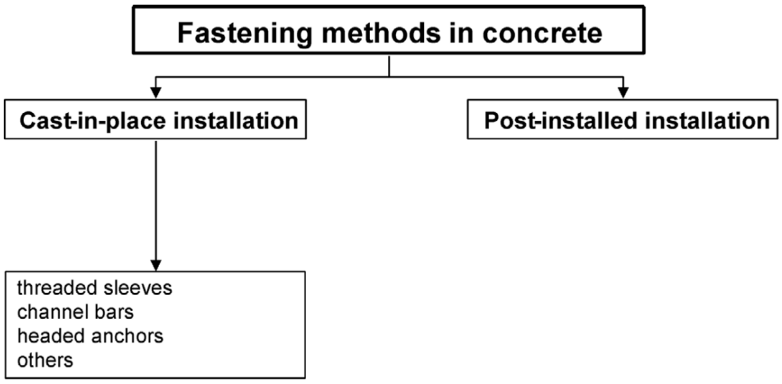
- An anchor bolt without a stud is a similar application to an anchor bolt with a stud. However, in the case of not enough distance between the stud and the concrete edge, an anchor bolt without a stud is suitable. Typically the bolt is placed near the edge of the foundation, depending on the design the bolt length may reach up to 3 m to effectively transfer the load through bonding of the steel bolt and concrete. See Figure 6. (Anstar rebar bolts, 2020, p. 7).

Figure 6. Anchor bolt without stud. (Anstar rebar bolts, 2020, p. 4)



The methods of fastening anchorage are shown in Figure 7. Cast-in-place installation is secured prior to casting, while in post-installation anchors are drilled into hardened concrete. (Eligehausen et al., 2006, p. 5)

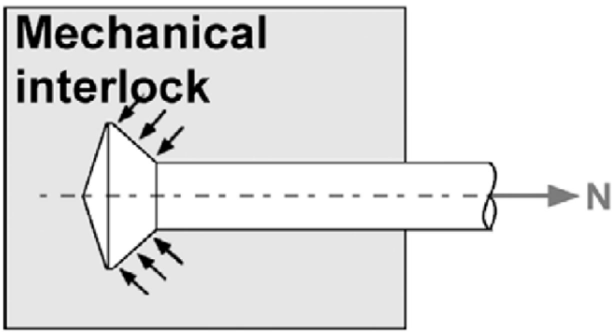
Figure 7. Fastening method in concrete. (Eligehausen et al., 2006, p. 1)



2.1 Load transfer of headed anchor

The headed anchor transfers its load through a mechanical interlock, which involves the transfer of load via a bearing interlock between the fastener and the base material, it is the most common name for load-transfer mechanisms used by headed anchors. Cast-in-place systems use a mechanical interlock between the embedded component and the concrete to transfer external tension into the base material. (Eligehausen et al., 2006, p. 5)

Figure 8. Headed anchor load transfer (Eligehausen et al., 2006, p. 5)



2.2 Application of cast-in-place headed anchor

Embedded plates are commonly used in construction, and in some applications they might encounter elevated temperatures. One good example is the project located in southern France called ITER where embedded plates were heavily used. The application of embedded plates started with installing of anchors before concrete pouring. Figure 9 shows the installation of anchors for the floor and Figure 10 shows the installation of anchors for the walls. When the concrete has hardened, the formwork is removed, and the fixtures and other structures can be installed by welding in the exposed plates, as shown in Figure 11

Figure 9. Embedded plates on the floor were installed prior to concreting. (iter, 2013)

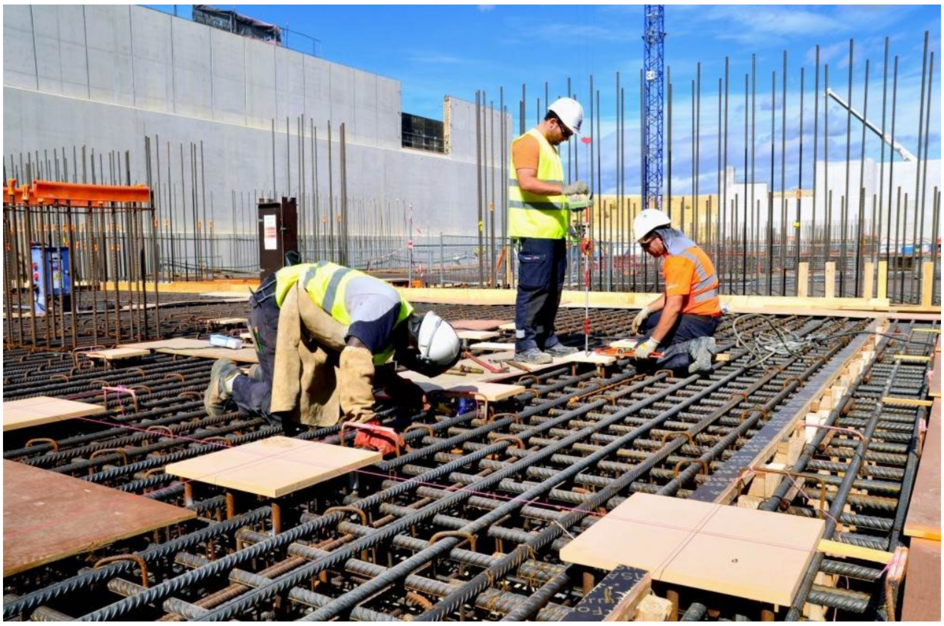


Figure 10. Each embedded plates on the wall were carefully aligned and secure. (iter, 2017)

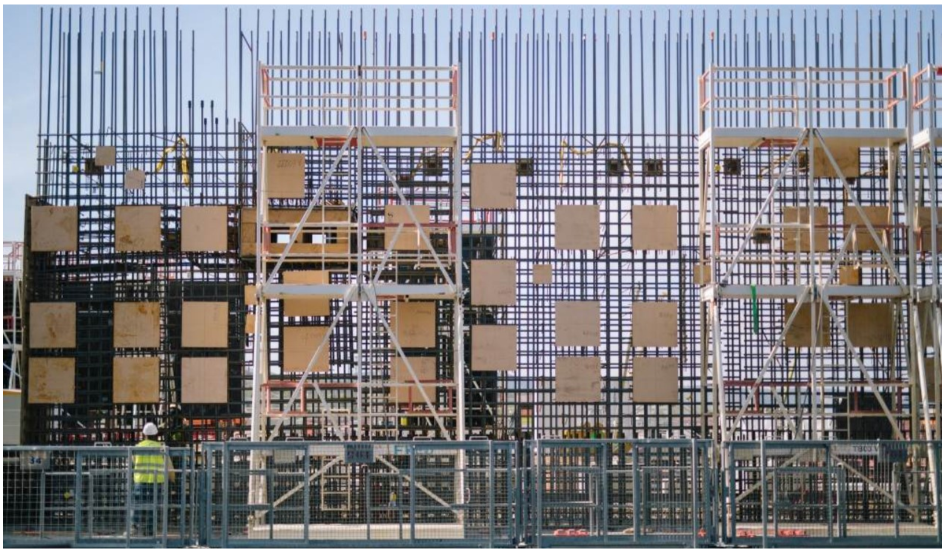
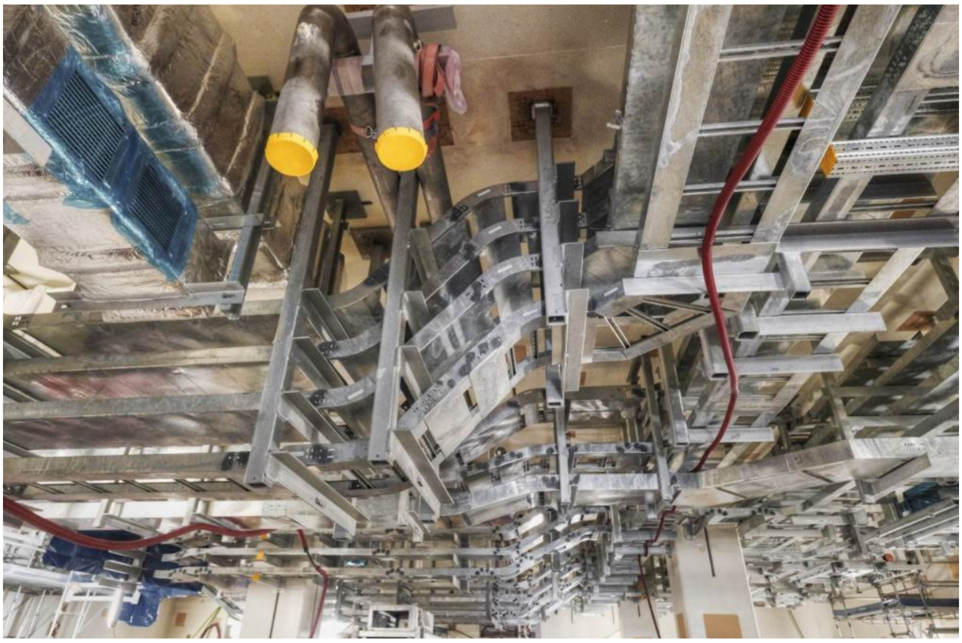


Figure 11. Supporting fixtures are welded into the exposed plates. (iter, 2021)



3 Concrete properties

3.1 Compressive strength

The compressive strength of concrete shown in Table 1. is obtained after curing in 28 days. For concrete cylinder and cube, it is denoted as f_{ck} and $f_{ck,\ cube}$.

Table 1. Compressive strength of concrete. (SFS-EN 1992-1-1, 2005, p. 29)

Compressive strength of concrete							
Strength class	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
f_{ck} (MPa)	20	25	30	35	40	45	50
$f_{ck,\ cube}$ (MPa)	25	30	37	45	50	55	60

The capacity to sustain large compression stresses is concrete's most crucial structural technical feature. The compressive strength of concrete is a crucial attribute to note from the perspective of designing and dimensioning of structures. Compressive strength is used to classify concrete into strength classes, from which other mechanical parameters for design are derived. According to the standard SFS-EN 1992-1-1, 2005, the compressive strength of the concrete is reported as the cylindrical strength when developing concrete structures in accordance with Eurocodes. In other words, the compressive strength of the structures is determined using test cylinders that are 150 mm in diameter and 300 mm in height. (Betonitieto, n.d.-a)

The strength that can be attained in the compression test is primarily influenced by the test piece's size, the caliber of the loading surfaces, and the loading speed, in addition to the material qualities. According to the Eurocode, every cube or cylinder test at the age of 28 days at a defined loading rate is used to determine the strength. If a non-standard cylinder is used to measure concrete's compressive strength, the strength must be converted to cylinder strength. The test results obtained with these test pieces, made of the same concrete a ratio of the strength of cylinder and cube ranges from 78 % to 85 % on average. In other words, the compressive strength determined by the cylinder test using the same concrete is marginally lower than the result determined by the cube test. The strength class affects the conversion factor slightly. (Betonitieto, n.d.-a)

Because of the variability in the shape, size, and proportions of the load area as well as the level of implementation, it is difficult to predict the compressive strength that may be attained in buildings. Experimental evidence has demonstrated that designing is safe when the compressive strength operating on the structure is equal to the strength determined by the cylinder test. However, compared to the real loading duration of the building, strength tests are completed quite rapidly. In comparison to the long-term test, the short-term test results in greater strength. Because of this, the factor α_{cc} lowers the strength acting on the structure. Its value has been set at 0,85 in Finland. (Betonitieto, n.d.-a)

The design value of compressive strength is obtained from the formula Equation (1)

$$f_{cd} = \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c} \tag{1}$$

where:

γ_c the material safety factor of the concrete

α_{cc} is the coefficient taking account of long term effect on the compressive strength. (SFS-EN 1992-1-1, 2004)

3.2 Tensile strength

The direct tensile strength, which is between 5 % and 8 % of the compressive strength and is comparatively larger in low-strength classes (8 %) than in high-strength classes (5 %) is the tensile strength utilized in the design. The strength class from the following formula is used by the Eurocode to compute the tensile strength. (Betonitieto, n.d.-b)

$$f_{ctm} = 0{,}30 \cdot (f_{ck})^{\frac{2}{3}} \qquad \text{when } f_{ck} \leq 50 \text{ MPa} \tag{2}$$

Equation (2) is the average tensile strength f_{ctm} for the strength class. Equation (3) is used to derive the particular strengths, the lower and upper limits of 5 % and 95 %, respectively.

$$f_{ctk,0.005} = 0{,}7\, f_{ctm} \qquad (3)$$

In design, the structure is not intended to operate in the failure mode relying on the tensile strength, rather the reinforcement must withstand the tensile stress. However, tensile strength is also used in indirect ways, such as in shear and anchoring design. (Betonitieto, n.d.-b)

The design value of the tensile strength is obtained from the formula

$$f_{ctd} = \alpha_{ct} \cdot \frac{f_{ctk,0,005}}{\gamma_c} \qquad (4)$$

where:

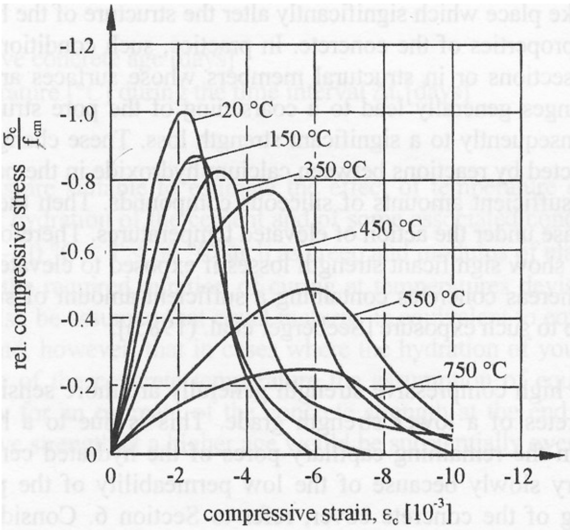
γ_c the material safety factor of the concrete

α_{ct} is a coefficient taking account of long term effect on the tensile strength. (SFS-EN 1992-1-1, 2004)

3.3 Effect of temperature on the properties of concrete

At the time of loading, the concrete compressive strength, and modulus of elasticity decrease as the temperature increases, while the plastic deformation and ultimate strain increase. (fib, 1999, p. 58).

Figure 12. Effect of temperature on compressive stress-strain (fib, 1999, p. 60)



Moisture level and the process of concrete curing have a significant effect on the strength of concrete. CEB-FIP MC 10 gives empirical relations to estimate the effect of temperature on the compressive strength of concretes. The effect of temperature in the range 0 °C ≤ T ≤ 80 °C on the compressive strength of normal strength and normal weight concrete may calculate as. (Paul Beverly, 2010, p. 95)

$$f_{cm}(T) = f_{cm} \cdot (1{,}06 - 0{,}003T) \tag{5}$$

where:

$f_{cm}(T)$ is the compressive strength in MPa at the temperature T in °C

f_{cm} is the compressive strength in MPa at the temperature T = 20 °C

T is the temperature (Paul Beverly, 2010, p. 95)

Phase transitions at elevated temperatures $T > 100$ °C can have a major impact on the mechanical properties of concrete. The amount and time of temperature exposure, the presence of moisture, the type of aggregates especially the quantity of sand, and the type and quantity of cement and additives all influence how much of these changes occur. Elevated temperatures may produce internal strains if the coefficients of thermal expansion of the aggregates and the cement paste differ sufficiently. However, if the concrete is constantly compressed while being subjected to high temperatures of up to 600 °C, there is no noticeable loss in strength as long as the compressive stresses exceed the internal tensile stresses. (fib, 1999, p. 59)

Hydrothermal reactions could occur, dramatically altering the structure of the hydrated cement paste and the mechanical properties of the concrete, if the drying of the concrete while exposed to high temperatures is not prevented. The pore structure of the hydrated cement paste typically becomes coarser as a result of the phase transitions, which results in a significant loss of strength. Compared to the concrete of a lesser strength grade, concretes with high compressive strength are typically more susceptible to high temperatures. This is because the residual capillary pores in the hydrated cement paste, which only very slowly expand due to the paste's poor permeability, are producing a high vapor pressure. Additionally, this can seriously spall the concrete cover. (fib, 1999, p. 59)

Elevated temperatures may produce a significant drop in concrete tensile strength at temperatures $T > 100$ °C, depending on the moisture state and rate of temperature change, due to internal stresses caused by non-linear temperature gradients and associated non-uniform shrinkage (fib, 1999). For the temperature range 0 °C ≤ T ≤ 80 °C according to CEB-FIP MC 10 temperature is significantly the tensile strength f_{ctm} of normal concrete and may be calculated as.

$$f_{ctm}(T) = f_{ctm} \cdot (1{,}16 - 0{,}008T) \qquad (6)$$

where:

$f_{ctm}(T)$ is the uniaxial tensile strength in MPa at temperature T in °C

f_{ctm} is the uniaxial tensile strength in MPa at T = 20 °C

T is the temperature (Paul Beverly, 2010, p. 95)

4 Fastener mode of failures

This chapter will present a short overview of the fastener failure modes given in Eurocode standard SFS-EN 1992-4 2018. Here, we will find the equations of the failure modes of the fastener. Fastener tensile resistance is dependent on the tensile strength of concrete, and later, in chapter 6, this relationship is presented the effect of elevated temperature on concrete and the effect on the capacity of the anchors is illustrated.

When designing fastener resistance it is important to understand the failure mechanism to provide a good solution to the problem. In Table 2, column one shows the two types of loading direction these are the axial forces acting on the fastener. The second column the failure mechanism, and column 3 clauses in EN 1992-4 for verification and calculation instruction.

Table 2. Verification required for tension and shear loading

Loading direction	Failure mode	Verification according to EN 1992-4
Tension loading	Concrete cone failure	Section 7.2.1.4
	Pull-out failure of fastener	Section 7.2.1.5
	Concrete blow-out failure	Section 7.2.1.8
Shear loading	Concrete pry-out failure	Section 7.2.2.4
	Concrete edge failure	Section 7.2.2.5

4.1 Tension failure

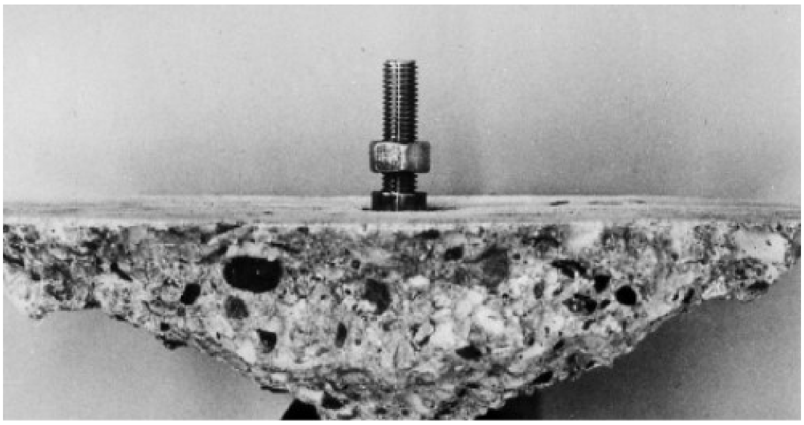
4.1.1 Concrete cone failure

Concrete cone failure happens when the bond in concrete to anchor is strong enough to resist the applied tension force, but the concrete tension resistance is low, and the length of the embedded anchor is very short so that t it can lift the concrete in the shape of a cone as shown in Figure 13. The failure mode of concrete cone breakout is characterized by the formation of a cone-shaped fracture surface. In the verification of conical concrete break-out, it is assumed

that the break-out cone begins at the bottom of the fastener's head and continues at a 35-degree angle. (Mallée et al., 2013, p. 93).

Equation (7) is the calculation of the resistance of the fastener $N_{Rk,c}$ concrete cone failure. (SFS-EN 1992-4, 2018, p. 51)

Figure 13. Concrete cone failure. (Eligehausen et al., 2006, p. 35)



$$N_{Rk,c} = N^0{}_{Rk,c} \cdot \frac{A_{c,N}}{A^0{}_{c,N}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{M,N} \tag{7}$$

where:

$N_{Rk,c}$ is the characteristic resistance in case of concrete cone failure under tension load

$N^0{}_{Rk,c}$ is the characteristic resistance of a single anchor

$A_{c,N}$ is the actual projected area

$A^0{}_{c,N}$ is the reference projected area

$\psi_{s,N}$ factor taking into account the disturbance of the distribution of stresses in the concrete due to the proximity of an edge in the concrete member in case of concrete cone failure

$\psi_{re,N}$ is the shell spalling factor

$\psi_{ec,N}$ is the group effect when different tension loads are acting on the individual fasteners of a group in case of concrete cone failure

$\psi_{M,N}$ is the actor taking into account the effect of a compression force between the fixture and concrete in case of bending moments with or without axial force

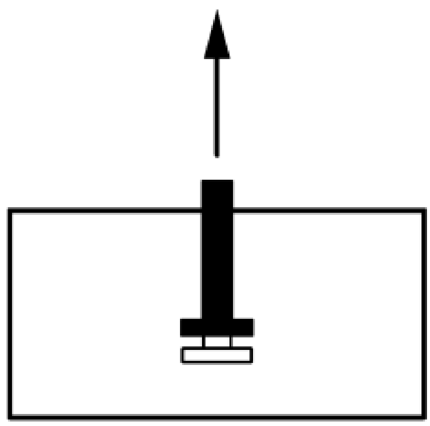
k_1 is the factor for cracked concrete

h_{ef} is the height is the anchor, not including the head

4.1.2 Pull-out failure

Pull-out failure, occurs when the entire fastener is dragged out of the borehole shown in Figure 14. Concrete near the surface could also sustain damage. (Mallée et al., 2013, p. 42). If the bearing resistance of the anchor head is adequate, failure mode may lead to concrete cone failure.

Figure 14. Pull-out failure. (SFS-EN 1992-4, 2018, p. 49)



Equation (8) is the calculation of the characteristic resistance of pull-out failure $N_{Rk,p}$ of headed fasteners. (SFS-EN 1992-4, 2018, p. 56).

$$N_{Rk,p} = k_2 \cdot A_h \cdot f_{ck} \tag{8}$$

where:

$N_{Rk,p}$ is the characteristic resistance in case of pull-out failure under tension load

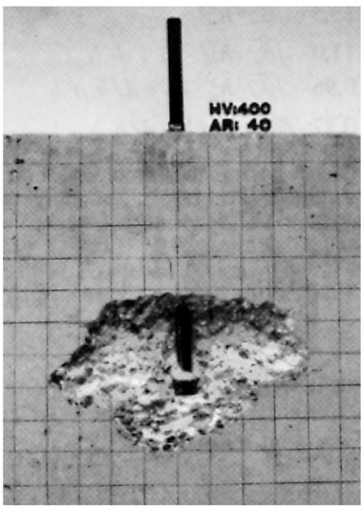
k_2 is the factor for cracked concrete

A_h is the load-bearing area of the head of a headed fastener

4.1.3 Blow-out failure

On tension-loaded headed studs, small edge distances can cause local blow-out failures near the head (see Figure 15). The concrete capacity of studs with a narrow edge distance (concrete cover) and a deep embedment will be governed by side blow-out failure. The quasi-hydrostatic pressure in the area of the stud's head is what causes local concrete side blow-out failure. (Eligehausen et al., 2006, p. 93)

Figure 15. Blow-out failure. (Eligehausen et al., 2006, p. 93)



Equation (9) and is the calculation of characteristic resistance $N_{Rk,cb}$ in case of concrete blow-out failure. (SFS-EN 1992-4, 2018, p. 61)

$$N_{Rk,cb} = N^0{}_{Rk,cb} \cdot \frac{A_{c,Nb}}{A^0{}_{c,Nb}} \cdot \psi_{s,Nb} \cdot \psi_{g,Nb} \cdot \psi_{ec,Nb} \tag{9}$$

where:

$N_{Rk,cb}$ is the characteristic resistance in case of concrete blow-out failure under tension load

$N^0{}_{Rk,cb}$ is the characteristic resistance of a single fastener, not influenced by adjacent fasteners or further edges The characteristic resistance of a single fastener

$A_{c,Nb}$ is the reference projected area for an individual fastener

$A^0{}_{c,Nb}$ is the actual projected area, limited by overlapping concrete break-out bodies of adjacent fasteners

$\psi_{s,Nb}$ factor taking into account the disturbance of the distribution of stresses in the concrete due to the proximity of an edge in the concrete member in case of concrete cone failure

$\psi_{g,Nb}$ factor taking into account a group effect of a number of fasteners in a row parallel to the edge in case of concrete blow-out failure

$\psi_{ec,Nb}$ factor taking into account the group effect when different tension loads are acting on the individual fasteners of a group in case of concrete blow-out failure

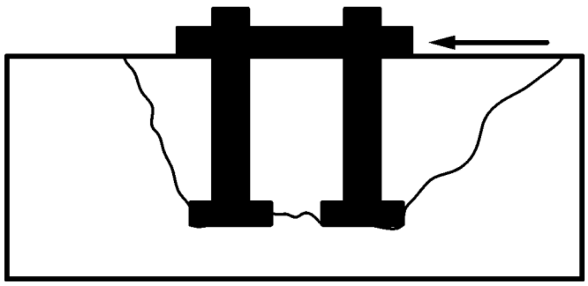
4.2 Shear failure

4.2.1 Concrete pry-out failure

Limited embedment anchors and shear studs loaded in shear can exhibit enough rotation to cause a pry-out fracture in which the primary fracture surface forms behind the point of load application. (Eligehausen et al., 2006, p. 105)

In Figure 16 illustrates the concrete pry-out failure. Fastenings may fail due to a concrete pry-out failure at the side opposite to the load direction. Pull-out failure may also occur due to a tension force introduced in the fasteners by the shear load. (SFS-EN 1992-4, 2018, p. 68)

Figure 16. Concrete pry-out failure. (SFS-EN 1992-4, 2018, p. 64)



Equation (10) is the calculation of characteristic resistance $V_{Rk,cp}$ in case of concrete pry-out failure under shear load. (SFS-EN 1992-4, 2018, p. 68)

$$V_{Rk,cp} = k_8 \cdot N_{Rk,c} \tag{10}$$

where:

$V_{Rk,cp}$is the characteristic resistance in case of concrete pry-out failure under shear load

$N_{Rk,c}$ is the characteristic resistance in case of concrete cone failure under tension load

k_8 is a factor to be taken from the relevant European Technical Product Specification

4.2.2 Concrete edge failure

A semi-conical fracture surface in the concrete that starts at the point of bearing and spreads to the free surface can cause anchors that are loaded in shear toward a nearby free edge to fail. (Eligehausen et al., 2006, p. 105)

Anchorages at edges that are subject to shear loads perpendicular to the edge may fail due to concrete fracture. The fracture crack on the concrete surface has, on average, a 35-degree angle with the edge. (Mallée et al., 2013, p. 68). Figure 17 studs are placed very close to the edge causing concrete edge failure.

Figure 17. Concrete edge failure. (Eligehausen et al., 2006, p. 112)

Equation (11) is the calculation resistance for $V_{Rk,c}$ concrete edge failure (SFS-EN 1992-4, 2018, p. 70)

$$V_{Rk,c} = V^0{}_{Rk,c} \cdot \frac{A_{c,V}}{A^0{}_{c,V}} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{ec,V} \cdot \psi_{re,V} \tag{11}$$

where:

$V_{Rk,c}$ is the characteristic resistance in case of concrete edge failure under shear load

$V^0{}_{Rk,c}$ is the characteristic resistance of a single fastener loaded perpendicular to the edge

$\psi_{s,V}$ is the factor takes account of the disturbance of the distribution of stresses in the concrete due to further edges of the concrete

$\psi_{h,V}$ is the factor takes account of the fact that the concrete edge resistance does not decrease proportionally to the member thickness

$\psi_{ec,V}$ is the factor takes into account a group effect when different shear loads are acting on the individual fasteners of a group

$\psi_{re,V}$ fastening in uncracked concrete and fastening in cracked concrete without edge reinforcement or stirrups

$A_{c,v}$.is the area of the idealized concrete break-out body, limited by the overlapping concrete cones of adjacent fasteners

${A^0}_{c,v}$ is the reference projected area of single fastener

5 Effect of elevated temperature to concrete-literature review

Numerous studies on the effects of increased temperature have been conducted based on the literature examined in this study. Some of the most important ones are provided below.

5.1 Y.F.Chang-Residual stress-strain relationship for concrete after exposure to high temperatures

Experimental research on the residual stress-strain relationship for concrete after exposure to high temperatures was conducted by Y.F.Chang in the Department of Architecture, National Cheng Kung University, Tainan, Taiwan. In his experiment, he prepared a standard concrete specimen that was subjected to an elevated temperature ranging from 100 °C up to 800 °C. His test result was then compared to Eurocode standard 1994-1-2 and EN 1992-1-2 and other experiments from different literature. Chang's research paper was published in the journals of Cement and Concrete Research 36 in May of 2006.

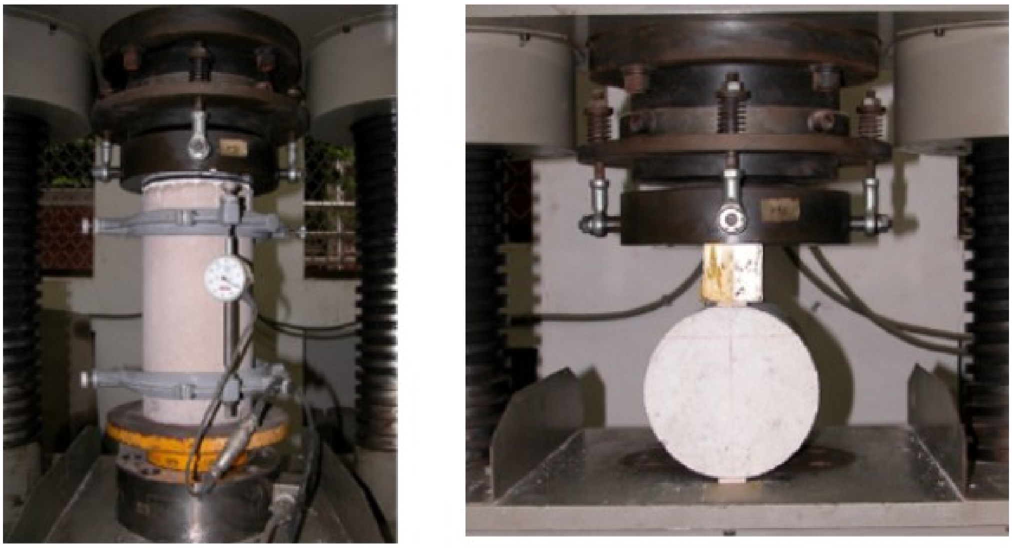
The full compressive stress-strain relationship for concrete after heating to temperatures between 100 and 800 °C was the subject of an experimental study. All concrete examples were siliceous aggregate-filled in a 15 cm 30 cm standard cylinders concrete mold. One month after being cooled to normal temperature, the heated specimens were tested. A single equation for the entire stress-strain curves of heated concrete is generated using the findings of 108 specimens with two original unheated strengths to take variation into account. The residual compressive strength, peak strain, and elastic modulus correlations between mechanical parameters and temperature are provided by regression analysis to fit the test data. In addition, 54 specimens are put through split-cylinder tests to investigate how splitting tensile strength relates to temperature. (Chang et al., 2006, p. 1)

Experimental work

All tests were performed on standard concrete cylinders measuring 15 cm in diameter by 30 cm in height. The specimens were made of a siliceous substance that was frequently used in Taiwan and Type I Portland cement. Compression tests evaluate two different types of compressive strengths. Using eight specimens of 40 MPa and four of 27 MPa, nine temperatures of room temperature, 100, 200, 300, 400, 500, 600, 700, and 800 °C were tested. Utilizing a total of 108 specimens, the entire stress-strain curves over a range of temperatures were measured. Two varieties of compressive strength were also tested for split-cylinder tests. The same nine temperatures were evaluated on three samples each at 32 MPa and 21 MPa. The splitting tensile strength was evaluated on a total of 54 samples at various temperatures. Heating rates of 1–10 °C/min were frequently used in earlier studies for heat treatments. Different maximum temperatures do not affect the rate. The heating rates, however, are set at 1 to 4,5 °C/min with a 0,5 °C/min increase, which corresponds to test temperatures of 100 °C to 800 °C with a 100 °C increment. It uses an electric furnace with a cross-section of 70 cm by 40 cm by 40 cm. (Chang et al., 2006, p. 2)

Five K-type thermocouples were used to measure the temperatures of the specimens, four were connected to the specimen surface at mid-height with equal spacing between them, and one was implanted at the center during casting. By varying the electrical furnace's heat output, the rising rates of average surface temperatures are matched as nearly as possible to the given heating rates. (Chang et al., 2006, p. 2)

Figure 18. a) Compression test b) Split cylinder test. (Chang et al., 2006, p. 2)



a b

Test procedures

The test setup is shown in Figure 18 with a 2000 kN universal testing machine. A compressometer was affixed to the specimen surface during the uniaxial compression test to quantify the average strains along a central 20 cm length. (Chang et al., 2006, p. 2)

By performing split-cylinder tests in accordance with ASTM C496 for unheated concrete, it was possible to determine the tensile strength of heated concrete for various temperatures. To prevent spalling, the specimens were heated for roughly 18 months following casting when their moisture level is low. In the furnace, three specimens were being heated simultaneously. The surface temperature was maintained constant for around 1,5–2,5 hours by changing the furnace's heat output until the center temperature achieves the same test temperature when the average surface temperature of the specimens reaches 20 °C above the required test temperature. The furnace then naturally cools down to normal to room temperature after that. To ensure that the residual strengths of heated concrete would be lowered to a minimum during compression tests, tests were conducted approximately one month afterward. (Chang et al., 2006, p. 2)

Experimental result

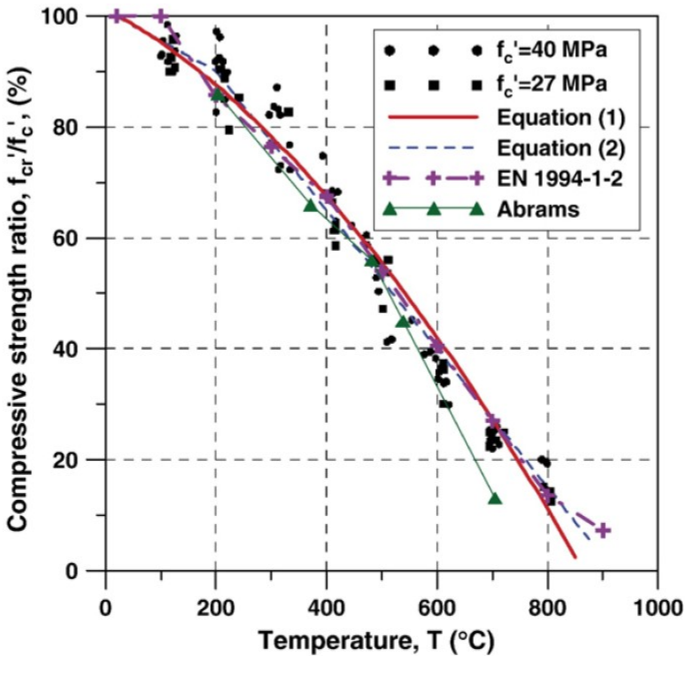
Chang's experiment on compressive strength f'_c as a function of temperature result was compared to Abrams investigation together with EN 1994-1-2 EN 1992-1-2 where siliceous aggregates were used and the result was very similar. The statistics in these graphs have all been averaged to reflect their original, unheated values. The corresponding average compressive strengths for unheated specimens were 27 MPa and 40 MPa, respectively. It should be noted that the EN 1994-1-2 curves are for heated concrete that has cooled to room temperature and curve EN 1992-1-2 is for concrete that is heated to elevated temperatures. (Chang et al., 2006, p. 3)

Compressive strength

The graph shown in Figure 19 in illustrates how compressive strength declines continuously, as the temperature rises the rate of decline in strength is slower below 200 °C. Furthermore, the normalized compressive strength after heating to different temperatures does not seem to be much impacted by the original strength f'_c. About 90% of the initial unheated value was still present in the residual strength at 200 °C. The equation represents the link between the residual normalized compressive strength f'_{cr}/f'_c and temperature T through regression analysis Chang proposed Equation (12) for temperature $20\ ℃ < T < 200\ ℃$ (Chang et al., 2006, p. 3).

$$\frac{f'_{cr}}{f'_c} = 1{,}01 - 0{,}00055T \quad 20\ ℃ < T < 200\ ℃ \tag{12}$$

Figure 19. Compressive strength after heating for different temperatures. (Chang et al., 2006, p. 3)

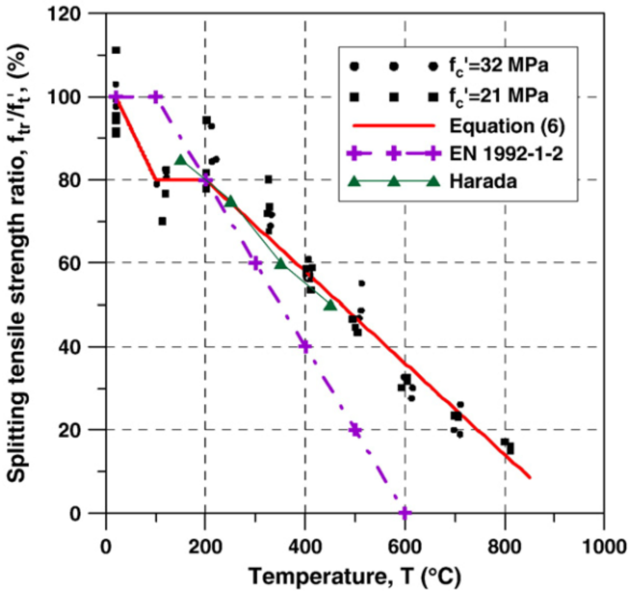


Tensile strength

The normalized tensile strength falls as temperature rises. Particularly for temperatures below 400 °C, the drop in tensile strength is larger than that in compressive strength for the same heat treatment. The normalized tensile strength shown in the following heating at

various temperatures does not seem to be much impacted by the initial strength f'_c. (Chang et al., 2006, p. 4)

Figure 20. Tensile strength after heating for different temperatures. (Chang et al., 2006, p. 3)



f'_{tr}/f'_t is the normalized tensile strength of concrete exposed at temperature *T.* (Chang et al., 2006, p. 5)

$$\frac{f'_{tr}}{f'_t} = \begin{cases} 1{,}05 - 0{,}0025T & ,20\ ℃ < T < 100\ ℃ \\ 0{,}8 & ,100\ ℃ < T < 200\ ℃ \end{cases} \tag{13}$$

Conclusion

Chang concluded his study by stating that his proposed model equation was agreeing with the experimental result. Furthermore, his proposed equation is very close to those of Abrams's experimental results, EN 1994-1-2, and EN 1992-1-2. The difference in compressive strength of the heated specimen compared to the normal unheated specimen was small at a temperature below 200 °C, in contrast, the tensile strength at elevated temperature drops faster than compressive strength.

1. According to Chang's experiment, the compressive strength decreased continuously. (Chang et al., 2006, p. 3)

2. For temperatures under 200 °C, the decrease rate of compressive strength was very slow. Compared to normal unheated concrete the loss in strength was only 10%. (Chang et al., 2006, p. 3)

3. Chang's experiment for concrete under elevated temperature was very similar to an earlier experiment by Abrams, and EN 1994-1-2. (Chang et al., 2006, p. 4)

5.2 Koichi Matsuzawa-Influence of recycled aggregates on the properties of concrete exposed to elevated temperature up to 175 °C

Koichi Matsuzawa is a senior research engineer in the department of building materials and components at the building research institute of Japan. He stated that the concrete exposed to elevated temperature can maintain its structural integrity if the temperature is not greater than 65 °C. In his experiment he investigated the effect of elevated temperature on the properties of concrete using recycled aggregate. He uses the local standard in Japan in the preparation of concrete specimens, compression, and tensile strength test. His research paper was then published in the journal SMIRT on 26 July 2022 in Berlin Germany.

Matsuzawa tested 4 different concrete specimens with different types of recycled aggregates. The test was conducted at sustained temperatures up to 175 °C for 90 days. The test result showed that the concrete that was altered by heat was the subject of active damage. Compressive strength was said to be affected by rising temperatures and the accompanying moisture level, as it declined more and more as the temperature rose. (Matsuzawa et al., 2022, p. 1)

According to the quality of aggregates in Japan, four different types of recycled aggregates 30 MPa were specified. The recycled aggregates were divided into four classes according to JIS standards: JASS 5N, JIS A 5021 Class H, JIS A 5022 Class M, and JIS A 5023 Class L. The NPP, Class H concrete materials were made from recycled JASS 5N aggregate and used for architectural structures. Class M can be used in building materials in reinforced concrete that did not need to be strong enough against dry shrinkage (Matsuzawa et al., 2022, p. 1).

The materials are listed in Table 3, and the factors and levels are listed in Table 4. The concrete mixed was made with two different kinds of coarse materials: crushed stone and recycled aggregate. The exposure temperature was set to 65, 110, and 175 °C, and the water-to-cement ratio was between 50 % and 60 %. For comparison, tests were also carried out on specimens that had not been heated (20 °C, 60 %R.H.). The ratios of the mixture are shown in Table 5. The concrete's water content was set at 178 kg/m^3. After placement, specimens were demolded, cured for 13 weeks in a chamber at a constant 20 °C, then exposed to heating and testing (Matsuzawa et al., 2022, p. 2).

Table 3. Materials. (Matsuzawa et al., 2022, p. 2)

Materials	Mark	Detail
Cement	MPC	Moderate-heat Portland cement, Density=3,21 g/cm^3
Fine aggregate	S	River sand by Kakegawa, Specific gravity=2,58 g/cm^3, Fineness Modulus F.M. =2,72
Coarse aggregate	G	Crushed stone by Oune, Specific gravity=2,64 g/cm^3 Absolute Volume=59,1 %
	RG	Recycled aggregate, Specific gravity=2,58 g/cm^3 Absolute Volume=60,5 %
Admixture	Ad	Air-entraining and water-reducing admixture

Table 4. Factors and levels. (Matsuzawa et al., 2022, p. 2)

Factor	Level
Water cement ratio (W/C)	50% (50), 60% (60)
Coarse aggregate	Crushed stone (G), Recycled aggregate (RG)
Exposure temperature	20 °C 60%R.H., 65 °C, 110 °C, 175 °C

Table 5. Moisture proportions. (Matsuzawa et al., 2022, p. 2)

Mark	W/C (%)	W (kg/m^3)	MPC (kg/m^3)	S (kg/m^3)	RG (kg/m^3)	G (kg/m^3)	Ad (C×%)
50G	50	178	356	781	960	-	0,012
50RG	50	178	356	781	-	938	0,010
60G	60	178	297	828	960		0,015
60RG	60	178	297	828	-		0,010

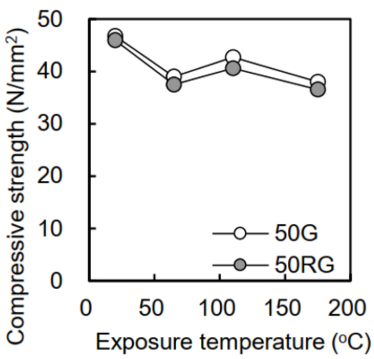
The specimen was on maximum temperatures for each pair of specimens were 65, 110, and 175 °C. Additionally, tests were performed on unheated samples (20 °C, 60 % R.H.). There was a 90-day exposure period. After exposure, the samples were analyzed after being gradually cooled to room temperature, and a test was done following the standard JIS A 1108 for compression and BS-1881 Part-117 for tensile strength test standard JIS A 1108 and BS-1881 (Matsuzawa et al., 2022, pp. 2–3).

After the test, the result shown in Figure 21 illustrates the compressive and tensile strength and exposure to temperature. The compressive strength is greater at a 50 % water-cement ratio than at 60 %. The compressive strength decreased at 65 °C, increased slightly at 110 °C, and then decreased once more at 175 °C. In general, however, heating tends to decrease compressive strength. At a water-cement ratio of 50 %, the compressive strength of concrete made with recycled aggregate is lower, but the difference is not great. At a water-cement ratio of 60 %, the compressive strengths are the same. (Matsuzawa et al., 2022, p. 4)

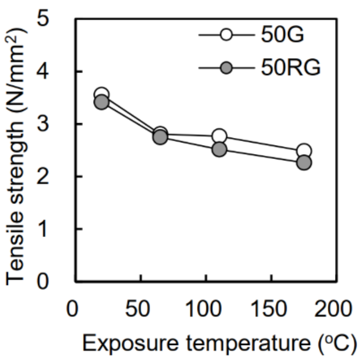
From 0.5 to 1 N/mm^2 decrease was seen in tensile strength when the temperature reaches 65 °C and continued to decrease slightly from 100 °C to 175 °C (Matsuzawa et al., 2022, p. 6)

Figure 21. Strength-exposure temperature relationship (Matsuzawa et al., 2022, pp. 5–6)

A) Compressive B) Tensile



A)



B)

Conclusion

Matsuzawa found that the compressive strength for both recycle aggregates lost its strength slightly. At 65 % of the water-cement ratio, concrete with recycled aggregates tends to have lesser strength than virgin aggregates. However, at a 50 % water-cement ratio the result in strength was the same. The tensile strength difference between aggregates, Matsuzawa did not observe. (Matsuzawa et al., 2022, p. 8)

5.3 El-Zohairy-Temperature effect on the compressive behavior and constitutive model of plain hardened concrete

Ayman El-Zohairy is a researcher from the University of Commerce Texas, USA. The scope of the test was to measure the effect of temperature on the compressive strength of concrete. The test was conducted using the American Society for Testing and Materials (ASTM) standard, with different temperature levels and two concrete compressive strengths. The experimental result was then compared to the other existing tests done by researchers Chang, Jau, Li, and Purkis. His research paper is submitted to the journals of Materials 2020 under the section Construction and Building Materials. The article was published on June 22, 2020.

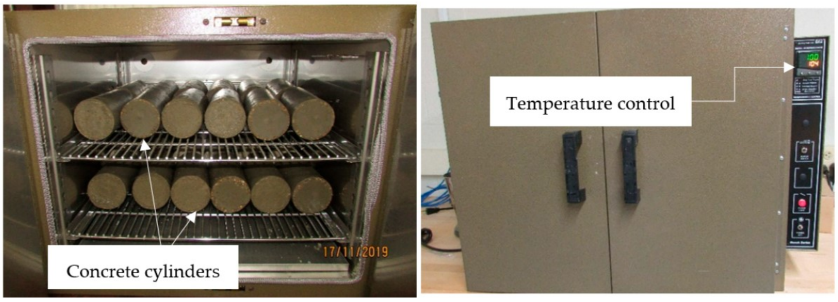
A total of 150 pieces of concrete cylinders were prepared to examine the effect of temperature conditions on compressive strength. This evaluation utilized two concrete grades, C20 (f_c = 20 MPa) and C27 (f_c = 27 MPa), and 75 concrete specimens for each concrete grade experiment. Samples were prepared in accordance with ASTM C31/C31 M. To account for the temperature effect alone, no admixtures were added to the concrete mixture to counteract the change in workability caused by temperature effects. As shown in Table 6, the mix ratio of two distinct concrete grades with different w/c ratios was used, the coarse aggregate consisted of continuously graded gravel aggregates with a maximum aggregate size of 10 millimeters. (El-Zohairy et al., 2020, p. 3)

Table 6. Concrete mix ratios. (El-Zohairy et al., 2020, p. 3)

Concrete Grade (MPa)	f_{ck} (MPa)	Aggregate size (mm)	Slump (mm)	w/c	Gravel (kg/m^3)	Sand (kg/m^3)	Portland cement (kg/m^3)	Fly Ash (kg/m^3)
C20	20	10	127,2	0,55	1067,9	830,6	334,6	30
C27	27	10	82,55	0,45	1067,9	830,6	334,6	30

The specimens were prepared according to ASTM C31/C31M specifications. The plastic mold cylinders were covered to preserve the concrete's water content, and the specimens of concrete were kept in the molds for 24 hours. The specimens were then removed from the plastic molds and the curing process began at the desired temperature for seven days. The specimens were cured and stored at five different temperatures, 0 °C, 21 °C, 40 °C, 121 °C, and 260 °C, to completely explore the impact of temperature on the mechanical properties of hardened concrete. As shown in Figure 22, the concrete samples were kept in temperature-controlled ovens at the desired temperature until the day of testing. (El-Zohairy et al., 2020, p. 4)

Figure 22. Temperature control ovens. (El-Zohairy et al., 2020, p. 4)



Once the specimen reached the thermal equilibrium, the specimen was placed into a standard compression test. Figure 23 shows the set-up of the specimen sample under compression test. To determine the mechanical characteristics of the concrete, a closed-loop servo-controlled is used, standard compression machine was operating under force control at a loading rate of 9,0 kN/min. These tests were performed in accordance with ASTM C39/C39M-20. At 7, 14, 28, 56, and 90 days, compression tests were performed. For each group, three samples were analyzed, and the average results were used. (El-Zohairy et al., 2020, p. 4)

Figure 23. Test set-up. (El-Zohairy et al., 2020, p. 5)

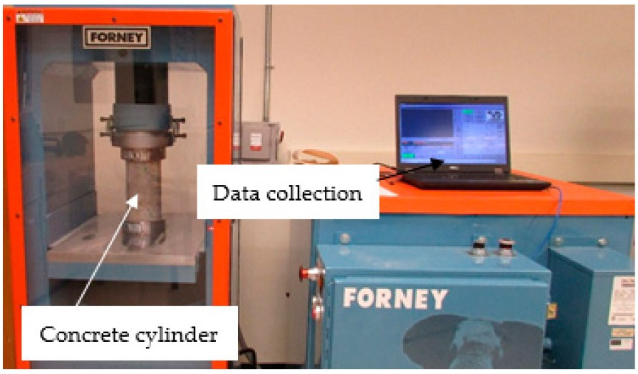


Table 7 show the average compressive strength of each tested group, with a standard deviation range of 2,79 % to 3,15 %, the statistics are the average outcomes of three concrete cylinders evaluated for each group at a specific age.

Table 7. Compressive strength of concrete. (El-Zohairy et al., 2020, pp. 8–9)

Days (d)	f'_c (MPa)	0 °C	21 °C	40 °C	121 °C	260 °C
7-day	C20	4,47	15,27	16,49	15,42	15,83
	C27	10,08	22,21	20,46	18,44	17,4
14-day	C20	2,97	17,93	15,05	15,29	14,64
	C27	8,17	26,96	22,06	20,95	20,47
28-day	C20	4,35	19,98	16,45	13,25	12,67
	C27	9,19	27,68	19,81	19,12	18,85
56-day	C20	3,73	24,01	18,16	14,47	14,33
	C27	8,43	31,34	23,19	20,66	17,89
90-day	C20	3,79	25,61	17,93	14,24	12,4
	C27	8,41	31,03	23,37	17,74	18,4

According to El-Zohairy's experimental result on the compression test, the specimen cured only for 7 to 14 days lost its strength of around 25 % after exposure to a moderate temperature of 21 °C, but as the concrete aged concrete gained its strength, the highest strength gained results were seen in the samples kept at 21 °C. (El-Zohairy et al., 2020, p. 8)

The summary of earlier constitutive models from the literature to predict the compressive strength of concrete under the influence of temperature was described by the following equations below:

Equation (14) was the model defined by Li and Purkiss. (El-Zohairy et al., 2020, p. 11)

$$f_{cT} = f'_c \cdot \left[0{,}00165\left(\frac{T}{100}\right)^3 - 0{,}03\left(\frac{T}{100}\right)^2 + 0{,}025\left(\frac{T}{100}\right) + 1{,}002\right] \tag{14}$$

Equation (15) proposed a model by Jau. (El-Zohairy et al., 2020, p. 11)

$$f_{cT} = f'c(1 - 0{,}001T) \tag{15}$$

Equation (16) described by Chang (El-Zohairy et al., 2020, p. 11)

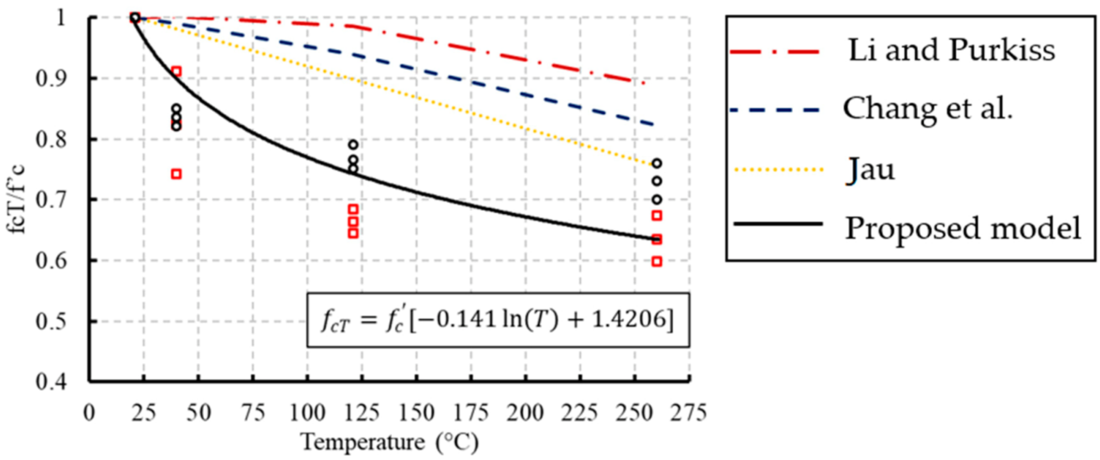
$$f_{cT} = f'_c \cdot \left[1{,}008 + \frac{T}{450\ln\left(\frac{T}{5800}\right)}\right] \tag{16}$$

These models were validated using the experimental data, and a new relationship was suggested by El-Zohairy as shown in Equation (17). The analysis was done from 21 °C to 260 °C temperature. The summary of all models is shown in Figure 24, which compares the proposed equation by El-Zohairy and existing models with the difference in compressive strength caused by temperature. (El-Zohairy et al., 2020, p. 10)

Equation (17) El-Zohairy proposed an equation. (El-Zohairy et al., 2020, p. 10)

$$f_{cT} = f'_c \cdot [-0{,}14\ln(T) + 1{,}4206] \qquad 20℃ < T < 260℃ \tag{17}$$

Figure 24. Comparisons between the effect of elevated temperature on concrete compressive strength (El-Zohairy et al., 2020, p. 10)



Conclusion

The experimental study done by El-Zohairy investigated how temperature affected the concrete. To investigate the effects of compression on concrete, cylinders were cast, cured, and aged in a variety of temperature-controlled environments. The Concrete was tested from 7 days to 90 days of age to determine its compressive strength. Previous constitutive models were verified using the experimental results, and new models were developed to predict the mechanical characteristics of concrete as a function of temperature.

1. According to actual results and the suggested model, concrete lost between 10 % and 20 % of its initial compressive strength at 100 °C and between 30 % and 40 % at 260 °C.

2. Slight increase of strength at early exposure to temperature, but gradually decrease as temperature increases.

The result from El-Zohairy’s experiment yielded a high reduction due to the method of experiment and concrete curing. 24 hours after concrete casting, the samples were removed from the mold and put into the electric furnace. The concrete samples were subjected to elevated temperatures resulting in fast dehydration of cement paste causing a significant loss of concrete strength.

5.4 Anypama Krishna-Effect of Elevated Temperatures on the Mechanical Properties of Concrete

Krishna is a researcher from Trivandrum college of engineering in Kerala, India. She investigated the effect of elevated temperature on the mechanical properties of normal-strength concrete. Her goal is to predict the concrete compressive and tensile strength and incorporate her model into the current standard use in India. Krishna used an electric heating furnace to simulate the effect on the concrete in case of a fire situation. The test results are then compared to the values in the published literature of American Society of Civil Engineers and Eurocode EN 1991 -1-2. Krishna's research paper was published in Procedia Structural Integrity 14 the year 2019.

Krishna used regular Portland cement (OPC 53), which complied with IS 12269-2013. Both coarse aggregate and fine aggregate underwent sieve analysis. Testing on new concrete was done following IS 8142-1976. The final mix proportions obtained are presented in Table 8 for the M20 grade of concrete. The mix design was completed under Indian standard IS 10262-2009, with a slump value of 60 mm, exposed to harsh conditions, and with a target mean strength of 28,25 MPa. The measured test strength was 32,4 MPa. (Krishna et al., 2019, p. 3)

Table 8. Mix proportion for the concrete. (Krishna et al., 2019, p. 3)

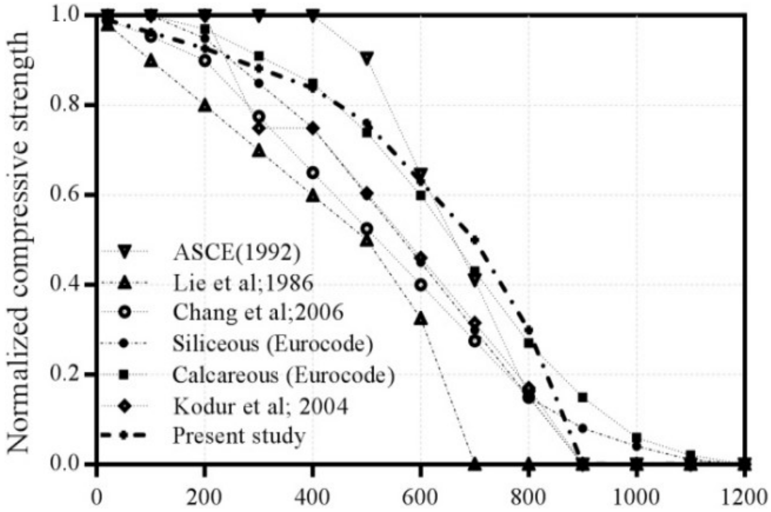
Property	Mix
Cement content (kg/m^3)	364,5
Fine aggregate (kg/m^3)	546,75
Coarse aggregate (kg/m^3)	1093,5
Water (l/m^3)	164

Standard-size cubes 150 mm x 150 mm x 150 mm cylinders in diameter and 300 mm in height were cast. The samples were cured for 28 days, air-dried in the lab for 24 hours, and then dried for 24 hours in an oven at 105 °C. The specimens were then heated for 1 hour to a final temperature between 100 and 1000 °C, the samples were then examined and air-cooled.

Two cooling techniques were used to measure the residual strength of concrete (1) through air cooling, which involved exposing the heated specimens to room temperature, and (2) water cooling, which involved quenching the specimens in water immediately after being exposed to high temperatures. (Krishna et al., 2019, p. 3)

The primary interest of the experiment is to find the effect of temperature on the compressive strength of concrete. Compressive strength at high temperatures was significantly affected by the heating rate, and batch mix binders such as silica fume, fly ash, and slag. Figure 25 shows the normalized compressive strength change with temperature for various models together with the current study by Krishna. (Krishna et al., 2019, p. 4)

Figure 25. Variation of normalized compressive strength with temperature from various models. (Krishna et al., 2019, p. 5)



The experimental data from the testing of 120 cube specimens, for the present research were fitted into a linear regression model. The modified equations take into account the linear dependency between the two explanatory variables, compressive strength, and temperature, as in the published literature, the ASCE model from 1992, and the Eurocode 2 model [EN 1992-1-2-2004]. (Krishna et al., 2019, p. 5)

From the literature and standards, the equations below are the summary of the influence of temperature on the compressive strength of concrete that was gathered by Krishna to compare her findings.

Equation (18)(19) as described by Krishna. (Krishna et al., 2019, p. 5)

$$f'_{cT} = f'_c \cdot (1{,}0032 - 0{,}00044 \cdot T) \qquad 20\ ℃ < T < 400\ ℃ \tag{18}$$

$$f'_{cT} = f'_c \cdot (1{,}4163 - 0{,}0016 \cdot T) \qquad 400\ ℃ < T < 800\ ℃ \tag{19}$$

Equation (20) (21) (22) was from EN 1992-1-2-2004. (Krishna et al., 2019, p. 5)

$$f'_{cT} = f'_c \cdot\ (1{,}067 - 0{,}00067 \cdot T) \qquad 100\ ℃ < T < 400\ ℃ \tag{20}$$

$$f'_{cT} = f'_c \cdot\ (1{,}44 - 0{,}0016 \cdot T) \qquad T > 400\ ℃ \tag{21}$$

$$f'_{cT} = f'_c \cdot T \quad 100\ ℃ < T \tag{22}$$

Equation (23), (24) was from the American Society of Civil Engineers (ASCE). (Krishna et al., 2019, p. 5)

$$f'_{cT} = f'_c \cdot T \qquad 20\ ℃ < T < 450\ ℃ \tag{23}$$

$$f'_{cT} = f'_c \cdot \left(2{,}011 - 2{,}353 \cdot \left(\frac{T-20}{1000}\right)\right) \qquad 450\ ℃ < T < 874\ ℃ \tag{24}$$

Equation (25), and (26) was proposed by Chang. (Krishna et al., 2019, p. 5)

$$f'_{cT} = f'_c \cdot (1{,}01 -\ 0{,}00055 \cdot T) \quad 20\ ℃ < T < 200\ ℃ \tag{25}$$

$$f'_{cT} = f'_c \cdot (1{,}15 -\ 0{,}00125 \cdot T) \quad 200\ ℃ < T < 800\ ℃ \tag{26}$$

Equation (33) (27), (28), (29) from Kodur. (Krishna et al., 2019, p. 5)

$$f'_{cT} = f'_c \cdot \big(1 -\ 0{,}003125 \cdot (T - 20)\big) \quad T < 100\ ℃ \tag{27}$$

$$f'_{cT} = 0{,}75 \cdot f'_c \qquad 100\ ℃\ < T < 400\ ℃ \tag{28}$$

$$f'_{cT} = f'_c \cdot (1{,}33 - 0{,}00145 \cdot T) \quad T > 400\ ℃ \tag{29}$$

Equation (30), (31) from Lie. (Krishna et al., 2019, p. 5)

$$f'_{cT} = f'_c \cdot (1 - 0{,}001 \cdot T) \quad T < 500\ ℃ \tag{30}$$

$$f'_{cT} = f'_{c} \cdot (1{,}375 - 0{,}00175 \cdot T) \quad 500\,℃ < T < 700\,℃ \tag{31}$$

The concrete tensile strength is substantially lower than compressive strength, as it is around 10 % of its compressive strength. At room and elevated temperatures the calculation of tensile strength is usually ignored, however, it is an important property because cracking in concrete is generally due to tensile stresses, and the structural damage of the member in tension is often generated by progression in micro-cracking (Krishna et al., 2019, p. 6)

Krishna's experimental model in her study, the rate of decrease of tensile strength is greater when the temperature is above 400 °C. At a temperature close to 800 °C concrete loses all its tensile strength. The tensile strength at elevated temperatures may calculated as shown in Equation (32)

Equation (32) is Krishna's proposed model for tensile strength

$$f_{cr,T} = f_{cr} \cdot (-0{,}0009(T) + 0{,}9678) \qquad 20℃ < T < 400℃ \tag{32}$$

Conclusion

Krishna concluded that the developed model from the experimental studies for the compressive strength of concrete at elevated temperatures is in good agreement with the existing literature. (Krishna et al., 2019, p. 9)

5.5 SFS EN 1992-1-2:2004 Design of concrete structures. Part 1-2: General rules. Structural fire design

SFS EN 1992-1-2:2004 provided instruction that the concrete structure should maintain its structural integrity if the structure is exposed to fire (SFS-EN 1992-1-2, 2004, p. 14). Though this study does not include structural integrity under fire, fire causes heat and high temperatures. Section 4.2.4 of SFS EN 1992-1-2, strength reduction should be applied to the characteristic compressive strength of concrete to take into account the effect of high temperature. (SFS-EN 1992-1-2, 2004, p. 31)

As shown in Figure 26, the graph of strength reduction of concrete as a function of temperature used for siliceous and calcareous aggregates, it is visible that the coefficient of reduction $k_{c(\theta)}$ decreases as the temperature *T* increases. As a result, concrete's compressive strength decreases. At room temperature *T* = 20 °C to 200 °C the reduction of strength is less than 10 %, a significant drop is seen at *T* = 400 °C and it continually drops faster until it reaches the highest temperature.

Equation (33) shows that the compressive strength of concrete in temperature $f_{c,\theta}$ is direclty proportional to the coefficient of reduction $k_{c(\theta)}$ multiplied by concrete strength f_{ck}.

$$f_{c,\theta} = k_{c(\theta)} \cdot f_{ck} \tag{33}$$

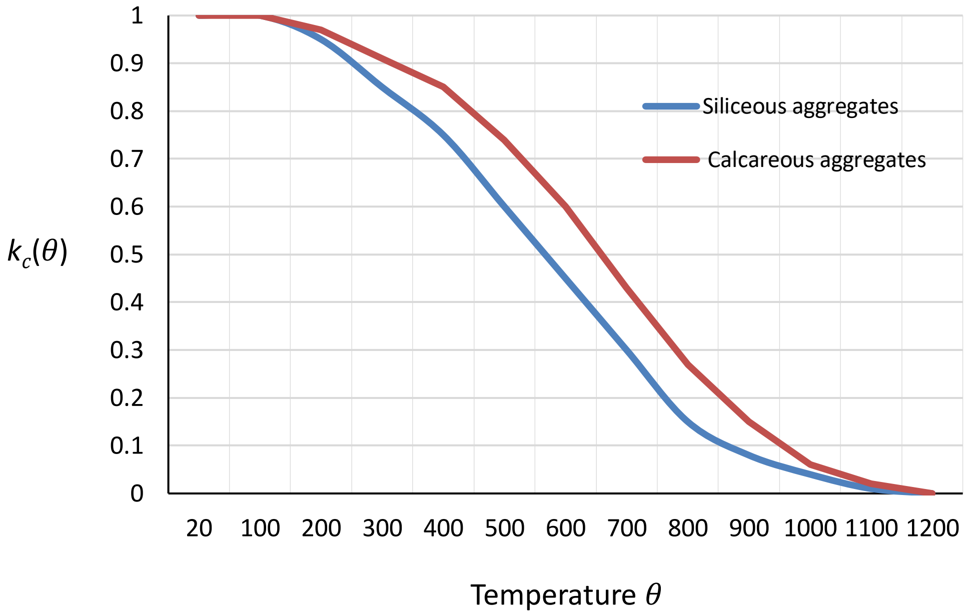
where:

$f_{ck(\theta)}$ is the characteristic compressive strength in temperature

$k_{c(\theta)}$ reduction coefficient in compression

f_{ck} compressive strength at room temperature

Figure 26. Coefficient $k_{c(\theta)}$ allowing for decrease in the characteristic strength f_{ck} of concrete. (SFS-EN 1992-1-2, 2004, p. 31)



Eurocode also gives provisions to reduce the tensile strength. Conservatively the tensile strength is ignored in the calculation, but it is necessary to take account of the tensile strength when using an advanced calculation method given in Eurocode, as shown in Equation (34) (SFS-EN 1992-1-2, 2004, p. 21)

$$f_{ck,t(\theta)} = k_{ck,t(\theta)} \cdot f_{ck,t} \tag{34}$$

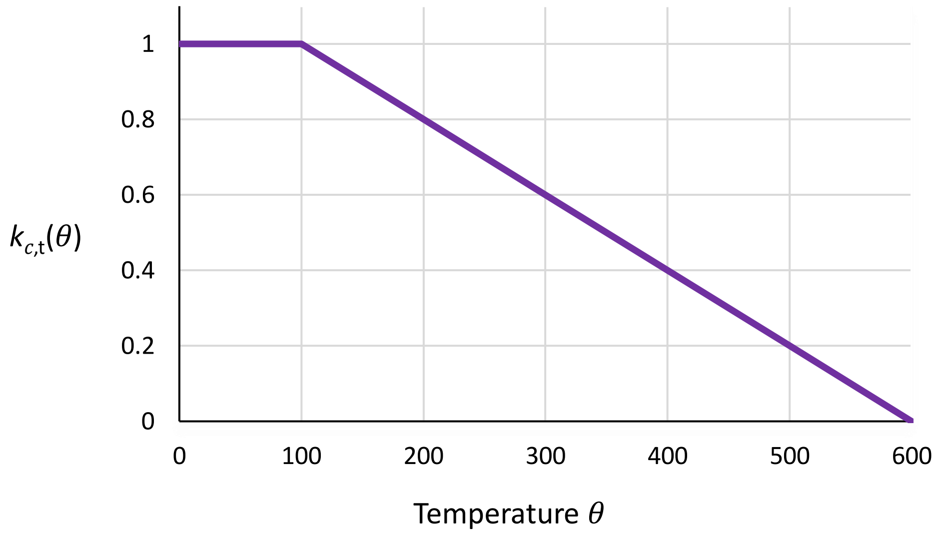
where:

$f_{ck,t(\theta)}$ is the characteristic tensile strength in temperature

$k_{ck,t(\theta)}$ is the reduction coefficient of tensile strength

$f_{ck,t}$ is the tensile strength at room temperature

Figure 27. Tensile strength reduction coefficient. (SFS-EN 1992-1-2, 2004, p. 22)



The graph shown in Figure 27 depicts the relationship between temperature to the normalized tensile strength of concrete. At the beginning of temperature load application $0 < T < 100$ °C, there is no reduction in concrete tensile strength, but when the temperature reaches $T > 100$ °C, the reduction drops linearly up to 600 °C.

5.6 SP 27.13330.2017-Concrete and reinforced concrete structures intended for the service in elevated and high temperature

In Russian standards, the calculation of the compressive strength of concrete exposed to elevated temperature is given in SP 27.13330.2017.

From the SP 27.13330.2017 a set of rules applies for concrete and reinforced concrete structures exposed to effects of elevated from 50 ° C to 200 ° C. The requirements for the design of these structures should be made of ordinary concrete with medium density from 2200 to 2500 kg / m^3 (SP 27.13330, 2017, p. 3).

In the design of concrete exposed to elevated temperature, heating duration, and exposure to moisture and water should take into account. Short-term heating refers to the first heating of the structure to the design temperature during its manufacture. Long-term heating is the effect of the design temperature during operation (SP 27.13330, 2017, p. 3).

According to Russian standards, in designing concrete exposed to an elevated temperature ranging from 50 ° C to 250 ° C, ordinary portland cement should be used with a density of 2200 kg/m^3 to 2500 kg/m^3 and with compressive strength of B20 (f_c = 20 MPa) to B60 (f_c = 60 MPa). Depending on the application: two types may be chosen; fast drying with slag and fast drying without slag. (SP 27.13330, 2017, p. 9)

When calculating concrete and reinforced concrete structures intended for operation under conditions of elevated and high temperatures, the effect of temperature on the change in the strength characteristics of concrete in compression is taken into account by multiplying them by the corresponding coefficients for the working conditions of concrete in compression by γ_{bt}. (SP 27.13330, 2017, p. 14)

The values of the coefficient of the working conditions of concrete in compression γ_{bt} and tension γ_{tt} can be found in Table 9.

Table 9. Coefficient working conditions. (SP 27.13330, 2017)

Concrete type	Type of heating	Coefficients of concrete working conditions in compression γ_{bt} and tension γ_{tt} concrete temperature °C			
		50 °C	70 °C	100 °C	200 °C
γ_{bt}	Short term	1	0,85	0,90	0,80
	Long term	1	0,85	0,90	0,80
	Long lasting with moisture	1	0,65	0,40	0,60
	Short term in water	0,97	0,85	0,65	-
γ_{tt}	Short term	1	0,70	0,70	0,60
	Long term	1	0,70	0,70	0,50
	Long lasting with moisture	1	0,50	0,30	0,40
	Short term in water	0,95	0,77	0,60	-

Equation (35) defines the compressive strengths of concrete $R_{b,tem}$ taking into account the temperature effect and R_b is the axial compressive (SP 27.13330, 2017)

$$R_{b,tem} = R_b \cdot \gamma_{bt} \tag{35}$$

where:

R_b = concrete compression strength (SP 27.13330, 2017, p. 13)
γ_{bt} = reduction factor in compression (SP 27.13330, 2017, p. 14)

Similarly, designing tension members exposed to elevated temperature the change in the characteristics of the tensile strength of concrete must take into account by multiplying the tensile strength R_{bt} by their respective working condition γ_{tt}. The influence of temperature on the tensile strength of concrete is denoted by R_{btt} and maybe calculated as shown in Equation (36)

$$R_{btt} = R_{bt} \cdot \gamma_{tt} \tag{36}$$

where:

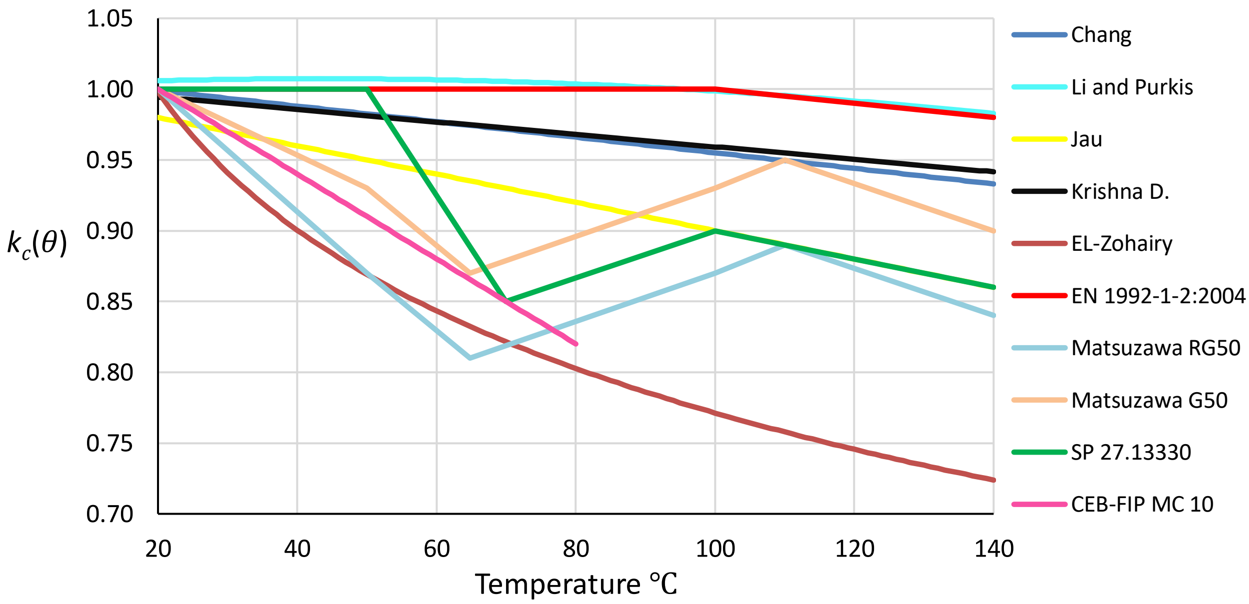
R_{bt} = concrete tensile strength (SP 27.13330, 2017, p. 13)
γ_{tt} = reduction factor in tension (SP 27.13330, 2017, p. 14)

6 Effect of elevated temperature on the strength of concrete

6.1 Compression strength

The graph shown in this study is relevant to this thesis, it is illustrated the reduction in concrete strength as the temperature increases. The graph is the compiled findings from the literature review regarding the effect of elevated temperature on the compressive strength of the concrete. The significance of this graph is to determine whether the concrete strength should be reduced or not. It can be noticed that the graph's general trend is that the reduction in concrete strength increases as the temperature increases. Therefore, the characteristic compressive strength should be reduced to safely design the fastener and to avoid fastener failure. It should be mentioned that EN 1992 1-1 and Russian standard SP 27.13330 are used in the actual design.

Figure 28. Reduction coefficient $k_c(\theta)$



The test experiments done by Chang and Krishna have been compared to the Eurocode EN 1992-1-2 and the result was almost similar. On the other hand, the experimental result by Kiochi Matzusawa on the concrete using fresh and recycled aggregates has a close similarity to the Russian standard with recycled aggregates yielding a higher reduction at around 0,81 and 0,84 from the Russian standard at a temperature of around 65 °C.

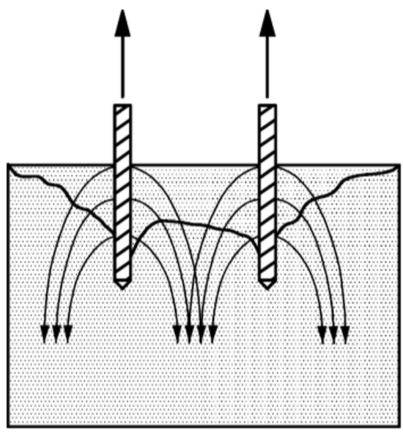
From El-Zohary's result in his proposed model was also compared to Eurocode EN 1992-1-2 and other earlier researchers from Jau, Li, and Purkis, his result yielded the lowest reduction factor however it is considered invalid because the concrete lost its strength at an early stage. The specimen was subjected to elevated temperature causing phase change leading to coarsening of the pore structure of the hydrated cement resulting in significant strength loss.

Researchers commonly compared their findings to EN 1992-1-2 because Eurocode is used in actual design. In a comparison of EN 1992-1-2 to Russian standard SP.27.13330, the Rusian standard gives a higher reduction factor. CEB-FIB MC 2010 is also a reliable source, their recommended values may be used for temperatures less than 80 °C. For this thesis, the calculation for reduction for compressive and tensile strength in the Russian standard is logical to use.

6.2 Tension strength

For the concrete tensile resistance, the tension force from the anchor is transferred to the concrete and tries to resist the force from splitting. In designing the fastener, according to Eurocode, the concrete tensile capacity is directly used to transfer the load into the concrete component (SFS-EN 1992-4, 2018, p. 9).

Figure 29. Fastener design theory (SFS-EN 1992-4, 2018, p. 10)



By the definition of fastener design theory, the tension force from the fastener is the same as the tension resisted by the concrete to maintain equilibrium. Using the Russian standard for reduction coefficient in tensile, which can be found in Table 9, the normalized tensile strength capacity of concrete can be calculated from Equation (37).

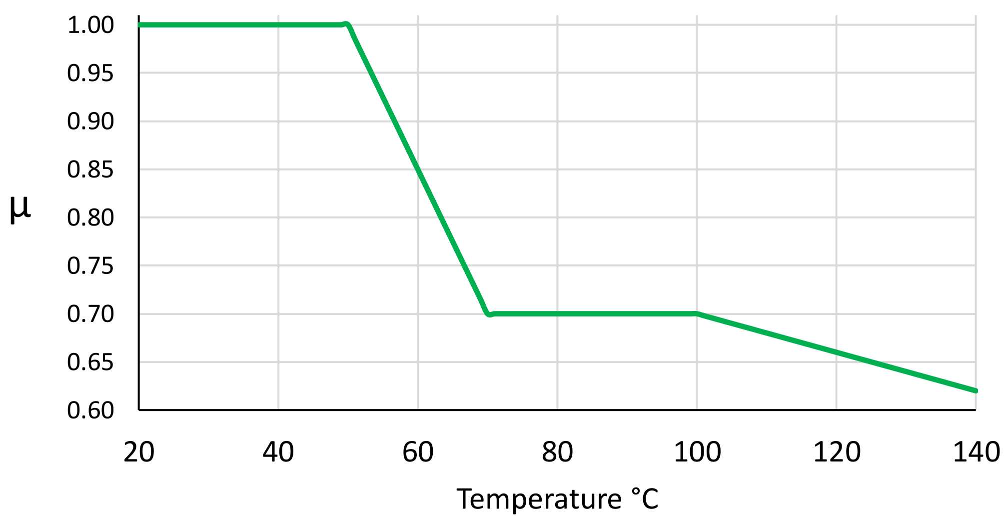
$$\mu = \gamma_{tt} \tag{37}$$

where:

μ = normalized concrete tensile capacity (unitless)

γ_{tt} = reduction coefficient of the working condition (SP 27.13330, 2017, p. 15)

Figure 30. Normalized concrete tensile strength (SP 27.13330, 2017, p. 15)



The graph shown in Figure 30 illustrates the normalized tensile strength of concrete as the temperature increases. At temperature T = 20 °C no reduction has been seen, but when temperature T = 50 °C a sudden drop of tensile strength reaches up to 30 % and begins to stabilize as the temperature increases from 70 °C to 100 °C and continues to drop again of about 10 % when it reaches 140 °C

7 Influence of temperature on the strength capacity of fastener

Fastener modes of failure are discussed in chapter 4, and fastener resistance is dependent on the tensile strength of concrete. For simplicity, consider the failure mode of cone failure of a single fastener $N^0{}_{Rk,c} = k_1 \cdot \sqrt{f_{ck}} \cdot h_{ef}{}^{1.5}$ and by neglecting the constants k_1 and $h_{ef}{}^{1.5}$ the new notation for fastener tensile resistance will now be $N^0{}_{Rk,c.Temp} = \sqrt{f_{ck}}$

Taking into account the effect of the temperature, a reduction factor should be introduced. Correct placement of the reduction factor will significantly affect the fastener's tensile resistance. Here we explore two methods of solution, Method 1 shown in Equation (38), and Method 2 shown in Equation(39).

$$N^0{}_{Rk,c.Temp,M1} = \gamma_{tt} \cdot \sqrt{f_{ck}} \tag{38}$$

$$N^0{}_{Rk,c.Temp,M2} = \sqrt{\gamma_{bt} \cdot f_{ck}} \tag{39}$$

The reduction factor γ_{tt} and γ_{bt} is taken from the Russian standard SP 27.13330.2017 using the long-term type of heating. Since the standard SP 27.13330.2017 is used in the real designing practice of concrete structures exposed to elevated temperatures, it is wise to use this standard.

Notice in Method 1 that the reduction factor in Equation (38) is outside the square root symbol, meaning that the effect of temperature on the fastener and the tensile capacity is reduced. Whereas, in Method 2 the reduction factor is placed inside the square root symbol reducing the concrete compressive strength.

Method 1 is the correct way of expressing the effect of temperature on the resistance of the fastener because the fastener tensile strength is reduced, from the fastener design theory, fastener tensile resistance is directly used to transfer the load.

To normalize the strength capacity of the fastener, Equation (38) should be divided by the single fastener resistance in normal room temperature $N^0{}_{Rk.c,20\,°C} = \sqrt{f_{ck}}$. The normalized strength resistance of the fastener exposed to elevated temperature is denoted by β and can be calculated using Equation (40).

$$\beta = \frac{N^0{}_{Rk,c.Temp,M1}}{N^0{}_{Rk.c,20\,°C}} \tag{40}$$

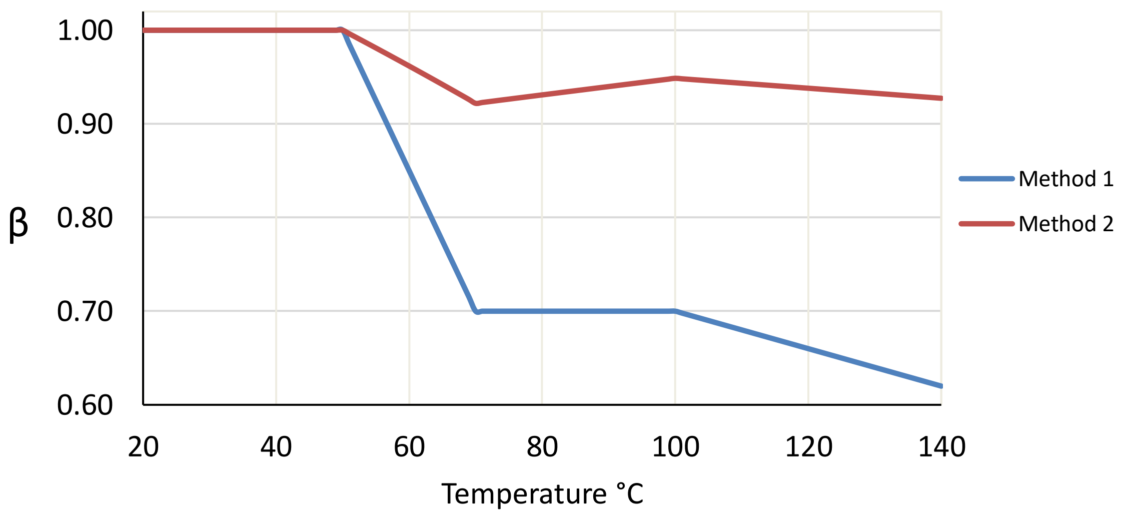
where:

β = Normalized fastener capacity (unitless)

$N^0{}_{Rk,c.Temp,M1} = \gamma_{tt} \cdot \sqrt{f_{ck}}$ anchor tensile resistance exposed to temperature

$N^0{}_{Rk.c,20\,°C} = \sqrt{f_{ck}}$ fastener resistance in normal temperature conditions T = 20 °C

Figure 31. Normalized fastener capacity



The reduction factor taken from Russian standard SP 27.13330 is Long term heating type, by substituting the values to Equation (40), it is obvious that $\sqrt{f_{ck}}$ will cancel out and will be left with $\beta = \gamma_{tt}$. Figure 31 is the graphical representation of how the fastener resistance is influenced by the elevated temperature. Method 1 shows a higher reduction because it's the tensile resistance formula itself is reduced, unlike Method 2, which yields a lower reduction because the reduction factor is inside the square root ($\sqrt{\gamma_{bt}}$). When designing fastener strength, fastener tensile resistance should be directly used.

8 Result

The results showed that elevated temperatures significantly affect both the tensile resistance of fasteners and the mechanical characteristics of concrete. In the concrete test performed in this thesis, the specimen's compressive and tensile strengths decreased as the temperature increased, and the extent of the damage varied depending on the temperature and the length of exposure to high temperatures.

9 Conclusion and discussion

The findings of this study have significant implications for concrete structure and fastener design. According to the data, concrete's mechanical properties can have significant damage at temperatures as high as 140 °C, which can reduce the material's compressive strength and fastener tensile strength. This information can be used to improve the performance of concrete and to develop new materials and construction techniques that can better withstand elevated temperatures. It should be mentioned that amongst the reviewed materials EN 1992-1-2 which are intended for fire, only and Russian standard SP 27.13330.20017 used for concrete structures designed to work in conditions of exposure to high and high temperatures, are the only ones that are in use in actual practice. The Russian standard gives a more conservative value than Eurocode EN 1992-1-2. According to both standards when designing a concrete structure exposed to elevated temperatures reduction to concrete strength should be applied. Similarly, reduction to the fastener tensile capacity should be reduced using the fastener direct formula.

In conclusion, concrete's stability and strength can be significantly affected by high temperatures. Concrete can expand and crack when it is exposed to high temperatures because the hydration process, which creates the structure of the concrete, can be sped up by heat. High temperatures can also cause the water in the concrete to evaporate, which will

result in additional shrinkage and lower strength. The concrete may also lose its structural integrity and fail if exposed to high temperatures over an extended period of time. Further research is needed to fully understand the mechanisms of damage in concrete at elevated temperatures and to develop new materials and construction techniques that can better withstand elevated temperature.

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