Kristian Hertz

DOCUMENTATION FOR CALCULATIONS OF STANDARD FIRE RESISTANCE OF SLABS AND WALLS OF CONCRETE WITH EXPANDED CLAY AGGREGATE

REPORT

Revised November 2002

Preface

The Danish producers of expanded clay concrete elements and blocks have shown a remarkable initiative by foreseeing the need for reliable methods for calculating the load bearing capacity of their elements exposed to fire.
Since expanded clay aggregate concrete mainly differ from the traditional heavy concrete qualities by the weight and porosity, but not by other substantial mechanical differences, it is logical to presume, that the basis for calculation of expanded clay aggregate concrete constructions accord with the basis for calculating the fire resistance of heavy constructions. This was previously established by the author and for example expressed in chapter 9 of the Danish Standard DS 411 from 1999 [3] or the simplified calculation method in chapter 4.3 of the CEN code printed as ENV 1992-1-2 from 1995 [4].
The basis for the calculations of expanded clay aggregate concrete constructions was written in the report "Calculation method for fire safety design of constructions of expanded clay aggregate concrete" [1] from 1997 as a preliminary proposal for a text for a Danish code of practice for expanded clay aggregate concrete. During the project a number of full-scale tests has been made in order to provide a reasonable documentation for the application of the calculation methods, and the present report deals with the second phase of full scale testing made in 2002.
The report comprises also the results of the first part of full-scale tests with reference to the report "Fire safety design of expanded clay aggregate concrete - Calculation of fire resistance time" [2] from 2001, and the results of the second phase of full-scale tests have been added. In addition the results from a Norwegian full scale test on a Scan Brann Blokk wall has been released for the purpose of documenting the calculation methods, and are adopted in this report.

Lyngby, May 2002

In the revised edition a small printing error of no significance for the conclusions has been corrected in one of the spreadsheets used for calculating walls, and the numerical values of the wall calculations are changed a few percent.

Lyngby, November 2002
Kristian Hertz

Acknowledgements

I would like to record my sincere thanks to the society of Danish producers of expanded clay aggregate concrete elements called Dansk Betonindustriforening (DBI), Element fraction (BIH) and Block fraction (BIB) and to Scan Brann Blokk for financing a number of full-scale tests, which can not only serve as a documentation for the calculation of expanded clay aggregate elements but contribute to the documentation for calculation of fire exposed concrete constructions in general.

Kristian Hertz

Contents

Summary

A number of full-scale tests are made in order to document calculation methods for fire-exposed slabs and walls derived during a previous project on fire exposed light-weight aggregate concrete constructions.
The calculation methods are derived, and thus have a logical connection with the calculation methods used for other load cases.
In addition the methods are shown to be valid for heavy concrete constructions by cooperation with tests for beams and columns, and a few slabs and walls.
The two test series phase 1 and 2 of this report can therefore be seen as a necessary supplement to show that the methods are applicable for slabs and walls of light weight aggregate concrete.
It is shown that the temperatures for standard fire exposed cross sections can be calculated, that the ultimate moment capacity can be calculated for slabs, and that the anchorage capacity and the shear tension capacity can be calculated for slabs and that a support of only 70 mm is sufficient for slabs with deformed bars and the actual loads.
It is also shown, that the load bearing capacity can be modeled for walls, if a detailed model for the thermal expansion is used, and if the calculation is made in time steps taking the transient strains into account.

Materials

The concretes used for the tests of this report is based on expanded clay aggregate and manufactured according to the requirements of the Danish Standards DS 414 and DS 420.
The values for compressive strength, tensile strength and E-modulus are assessed as average values in a hot condition in order make the calculations comparable to the test results. This means that differences between calculations and test results can only be ascribed to differences between calculation model and test method, and should not be influenced by a difference between a characteristic and average values of the material properties.
For the application of the calculation methods in practice, the characteristic values should be applied in stead, and therefore the load bearing capacities must be expected to be somewhat smaller.
Fixed values of the thermal conductivity used in simple temperature calculations are assessed as the values at 500°C.
For a 600 kg/m^3 concrete with plastered surfaces (as used in phase 2 of the project) and mortar between the blocks the final average density is 880 kg/m^3.

Temperature calculations

The temperature calculations are made using a simplified method developed by the author and adopted in the Danish concrete code DS 411 [3] representing an exact solution to the Fourier equation for heat conduction for a sinus variation of the surface temperature. This solution is used approximating the first quarter of the sinus cycle with the variation of temperature at the surface of a concrete specimen exposed to a standard fire. Hence this solution is valid only for a standard fire exposure and another calculation must be adopted in case the load bearing capacity should be calculated for an element exposed to a fully developed fire.
The expression is given as

Simple temperature calculation for a wall:

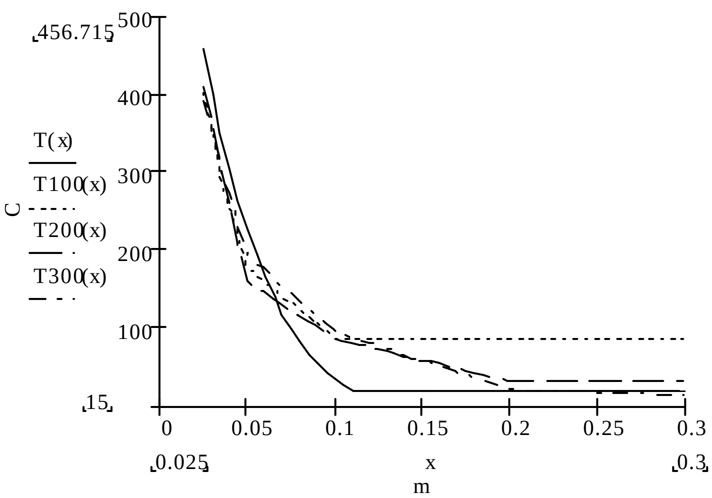
$$T_0(x,t) := 312 \cdot \log(8 \cdot t + 1) \cdot \exp(-1.9 \cdot k(t) \cdot x) \cdot \sin\left(\frac{\pi}{2} - k(t) \cdot x\right) \qquad k(t) := \sqrt{\frac{\pi \cdot \rho \cdot c_p}{750 \cdot \lambda \cdot t}}$$

where t is the time in minutes, ρ the density in kg/m^3, c_p the specific enthalpy and λ the conductivity of the concrete. The empirical temperature factor 312 is chosen to follow the surface temperature of a standard fire exposed concrete taking the effect of evaporation of water from the concrete into account.

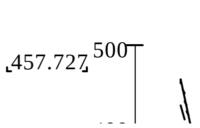
In the **first phase** of full scale tests reported in Hertz and Hansen [2], a wall constructed of tiles of quality 600, 1200 and 1800 kg/m^3 and thickness 100, 200 and 300 mm was tested (Andersen [7]) and temperature profiles were recorded and compared to calculated temperature distributions for the time 60 minutes giving a reasonable agreement between calculation and test.

____ T(x) Calculated simple formula
…….. T100(x) Measured in 100 mm wall
__ __ T200(x) Measured in 200 mm wall
__ . __T300(x) Measured in 300 mm wall

t = 60 min
ρ= 1800 kg/m^3
λ= 0.9 W/m°C
c_p=1000 J/kg°C

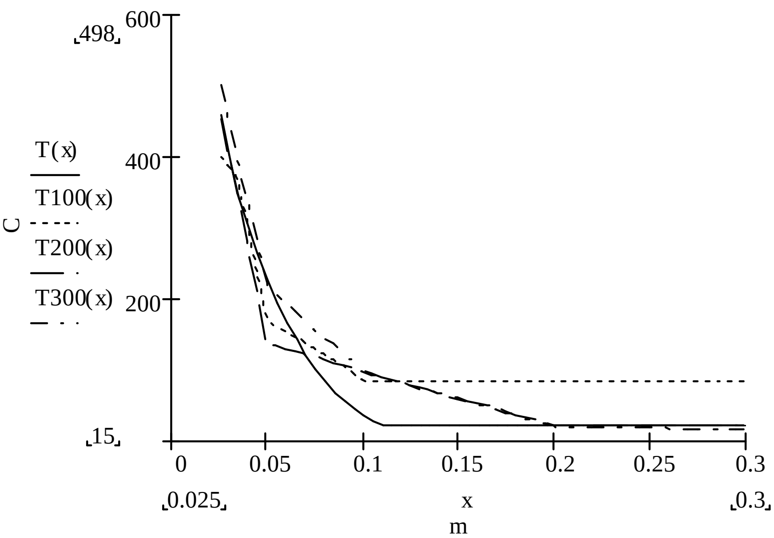


____ T(x) Calculated simple formula
…….. T100(x) Measured in 100 mm wall



t = 60 min
ρ= 1200 kg/m^3
λ= 0.6 W/m°C
c_p=1000 J/kg°C

____ T(x) Calculated simple formula
…….. T100(x) Measured in 100 mm wall
__ __ T200(x) Measured in 200 mm wall
__ . __T300(x) Measured in 300 mm wall



t = 60 min
ρ= 600 kg/m^3
λ= 0.3 W/m°C
c_p=1000 J/kg°C

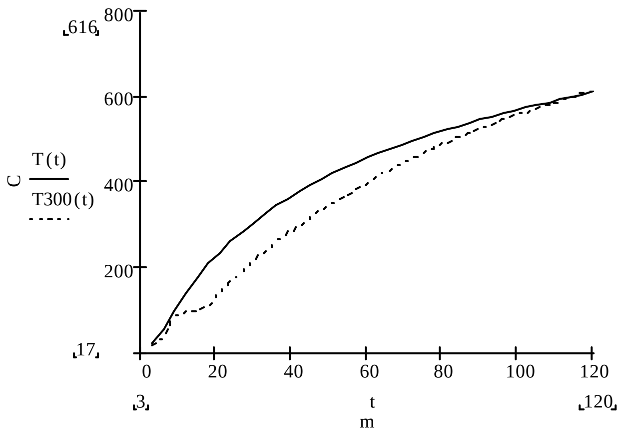
It is seen that the curves calculated by the simple formula gives too low values at temperatures less than 100°C, because the formula is derived from an exact solution for the harmonic oscillation, which would proceed into the negative part, but is cut off. The other deviations are comparable with the uncertainty of the measurements. The deviations in the temperature region less than 100°C have no effect on the load bearing capacities, because no or small strength reductions take place here.

From the following temperature curved in fixed depths is seen that the measured temperature is constant due to evaporation at 100°C, and that the simple formula is modified for this effect seen for heavy as well as for expanded clay aggregate concrete.

Temperature development in the depth 25 mm for density 1800 kg/m^3.

Temperature-time curve from a 300 mm tile with

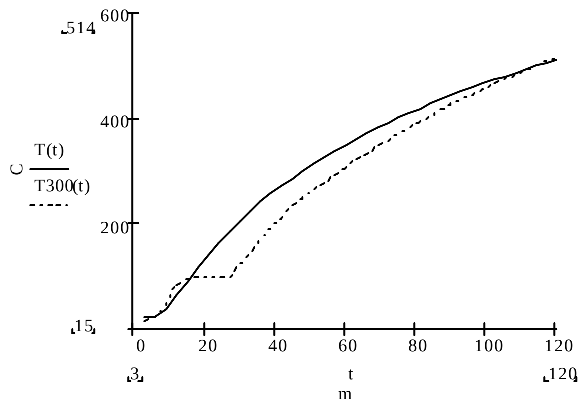
x = 25 mm
ρ= 1800 kg/m^3
λ= 0.9 W/m°C
c_p=1000 J/kg°C



Temperature development in the depth 35 mm for density 1800 kg/m^3.

Temperature-time curve from a 300 mm tile with

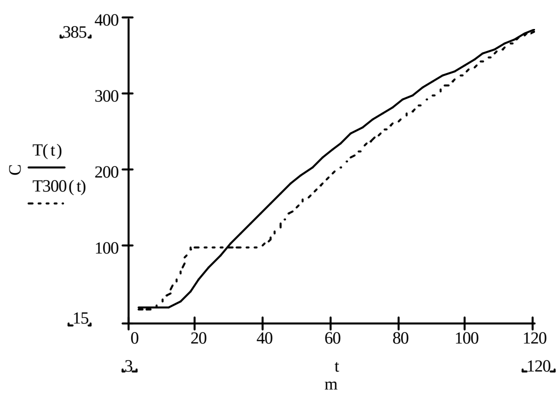
x = 35 mm
ρ= 1800 kg/m^3
λ= 0.9 W/m°C
c_p=1000 J/kg°C



Temperature development in the depth 50 mm for density 1800 kg/m^3.

Temperature-time curve from a 300 mm tile with

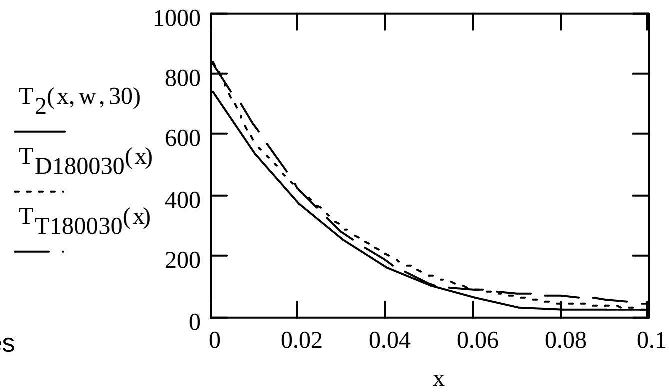
x = 50 mm
ρ= 1800 kg/m^3
λ= 0.9 W/m°C
c_p=1000 J/kg°C



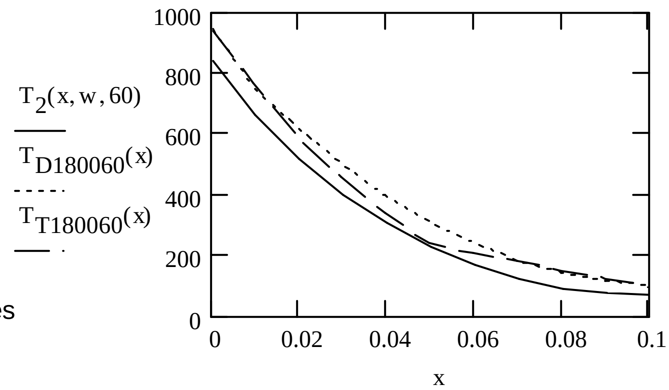
In the **second phase** of the full scale tests temperatures were measured at the reinforcing bars of deck elements and at different depths of walls.
Unfortunately the exact positions of the points of measurement were not recorded, but still the results can serve as a further documentation by defining the depth obtaining some agreement between calculated and test recorded temperature curve as a function of time and then comparing the temperature profiles made by the recorded temperatures at the derived depths against the calculated temperature profiles for different fixed times.

These results are shown for concrete quality 1800, 1200 and 600 kg/m^3 from Andersen [11], [12] and [13] on the following pages, where the temperatures calculated by the simple method are called T_2 and the temperatures from the tests are called T_T and in addition temperatures calculated by a finite difference method are shown called T_D.

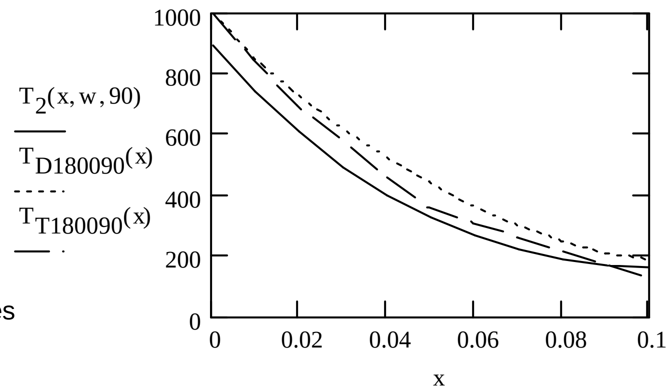
Temperature profiles
for 1800 wall
at 30, 60 and 90 minutes
calculated by proposed
method T2 and by
a Finite Difference
method TD and measured
at test TT.



30 minutes

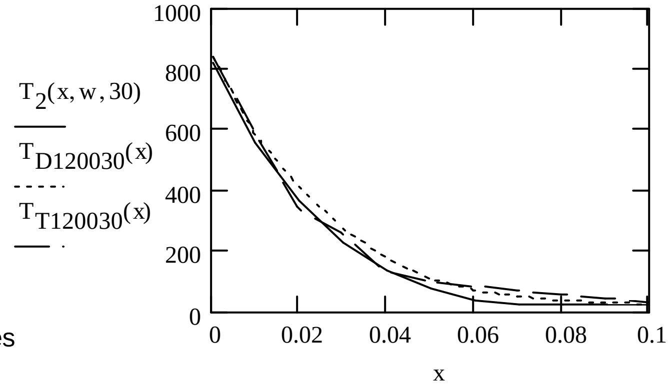


60 minutes

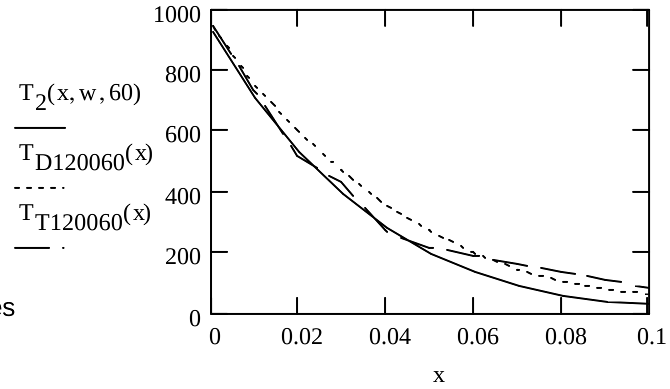


90 minutes

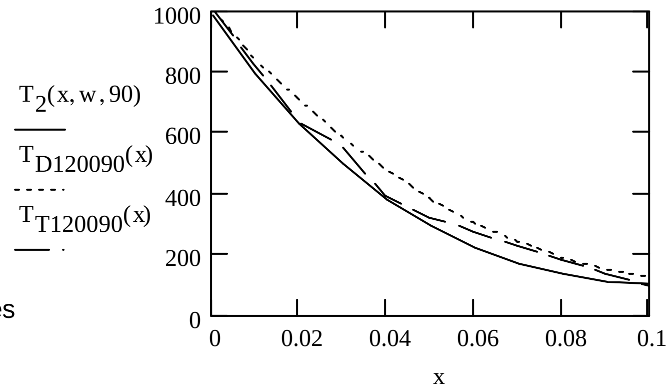
Temperature profiles
for 1200 wall
at 30, 60 and 90 minutes
calculated by proposed
method T2 and by
a Finite Difference
method TD and measured
at test TT.



30 minutes

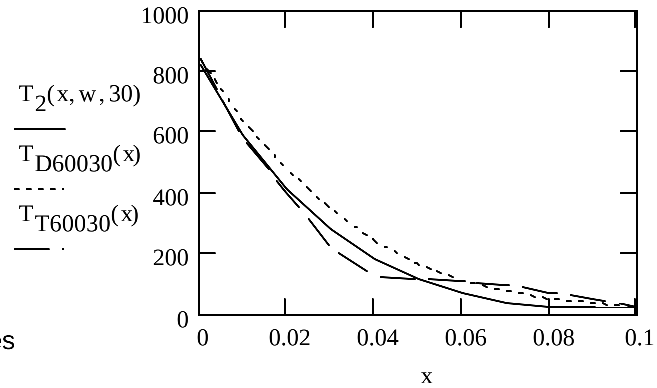


60 minutes

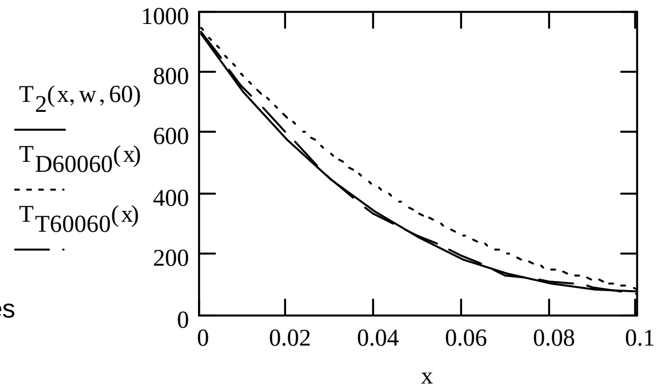


90 minutes

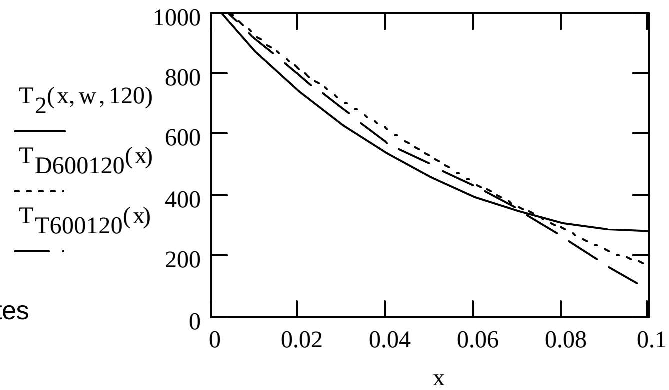
Temperature profiles
for 600 wall
at 30, 60 and 90 minutes
calculated by proposed
method T2 and by
a Finite Difference
method TD and measured
at test TT.



30 minutes



60 minutes



120 minutes

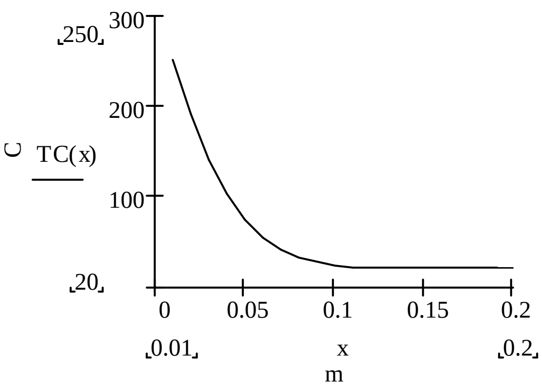
Anchorage calculations for slab elements

By means of the temperature calculation program HEAT2 the temperature conditions are calculated in the corner, where a light weight aggregate concrete slab of thickness 200 mm rests on a support of a 200 mm thick expanded clay aggregate concrete wall
The anchorage strength of a reinforcing bar with centre line 20 mm above the bottom of the slab is calculated.
A net of 10 mm masks is used extended 400 mm in the wall as well as in the slab.
The following in-data are used: Expanded clay aggregate concrete with λ= 0.6 W/mK, ρ = 1775 kg/m^3, c_p = 1000 J/kgK. Border conditions t = 60 minutes. No heat flux across surfaces except the two fire exposed inner surfaces, which are exposed by a surface temperature development according to the one used for the simple temperature calculation from the previous chapter.
The temperature development in °C is given by: 20 + 836sin(2π(t-0s)/14400s),
where 836°C is 312*log(8*60+1)°C and 14400s = 4*60*60s = 4 hours = the time for a full period, where 1 hour is a quarter of a period equal to the heating period.
The resistance of heat transfer at the surface is set to be rather low such as 0.01m^{2}K/W, in order to make the surface temperature vary as a sinus of max 846°C
by the expression above, which is defined as a surface temperature variation.

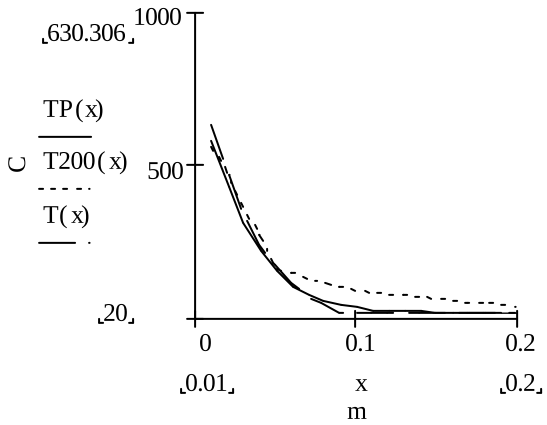
On the next page the result is presented as isotherms.

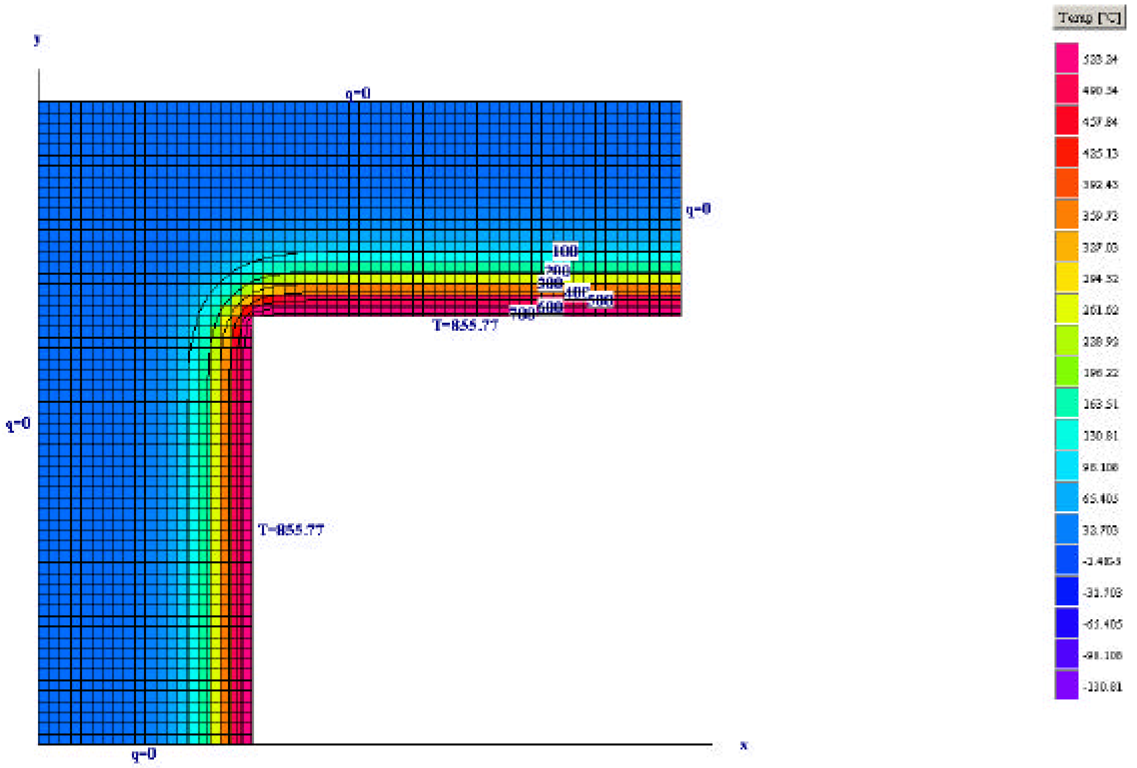
Utilizing the facilities of HEAT2 the temperature profile in the depth 20 mm along the reinforcing bar is derived counted from the corner = 0.00 m and inwards. In addition the temperature profile in the distance 300 mm from the corner is derived, where the isotherms are parallel. The last profile is compared with the measured profile from a 200 mm thick wall with λ = 0.6 W/mK, ρ = 1800 kg/m^3 and c_p = 1000 J/kgK, i.e. for approximately the same concrete.

Temperature profile along a reinforcing bar in the depth 20 mm above the bottom of a slab calculated by HEAT2 from the corner and inwards after 60 min standard fire at light aggregate concrete slab 1775 kg/m^3.



Temperature profile through 200 mm light aggregate concrete slab 1775 kg/m^3 after 60 min standard fire TP calculated by HEAT2 compared with simple calculation T and with measured temperatures T200 from 200 mm wall 1800 kg/m^3





Temperature distribution in a corner of a slab and a wall after 60 minutes standard fire calculated by HEAT2 for expanded clay aggregate concrete of λ= 0.6 W/mK, ρ = 1775 kg/m^3, c_p = 1000 J/kgK.

Along the reinforcing bar the following temperatures are calculated in the centre line at the distance 0.020m from the bottom of the slab and in the mid points of 3 lamellas of 5 mm thickness as a function of the depth from the corner and above the wall:

Dybde	T 0.020	T 0.0125	T 0.0075	T 0.0025
m	°C	°C	°C	°C
0.00	311	396	451	561
0.01	250	310	349	410
0.02	191	231	256	593
0.03	141	167	183	206
0.04	102	118	129	143
0.05	73	84	90	99
0.06	54	60	64	69
0.07	40	44	46	50
0.08	32	34	35	37
0.09	27	28	29	30
0.10	24	24	25	25

The anchorage capacity is assessed as the minimum of the bond capacity for pulling out the bar of the concrete and the splitting capacity for formation of splitting cracks along the bar. The theory is presented in Hertz [5].

The bond strength is reduced along the bar by the reduction of the compressive strength of the concrete ξ_c.

Depth	T 0.020	ξc
m	°C	-
0.00	311	0.926
0.01	250	0.967
0.02	191	1.000
0.03	141	1.000
0.04	102	1.000
0.05	73	1.000
0.06	54	1.000
0.07	40	1.000
0.08	32	1.000
0.09	27	1.000
0.10	24	1.000

Depth	T 0.0125	ξc	T 0.0075	ξc	T 0.0025	ξc	ξc average
m	°C		°C		°C		
0.00	396	0.869	451	0.833	561	0.759	0.820
0.01	310	0.927	349	0.901	410	0.860	0.896
0.02	231	0.979	256	0.963	293	0.938	0.960
0.03	167	1.000	183	1.000	206	0.996	0.999
0.04	118	1.000	129	1.000	143	1.000	1.000
0.05	84	1.000	90	1.000	99	1.000	1.000
0.06	60	1.000	64	1.000	69	1.000	1.000
0.07	44	1.000	46	1.000	50	1.000	1.000
0.08	34	1.000	35	1.000	37	1.000	1.000
0.09	28	1.000	29	1.000	30	1.000	1.000
0.10	24	1.000	25	1.000	25	1.000	1.000

From these numbers it can be seen, that the reduction of the bond capacity for an anchorage length of 50 mm on average will be 0.979.

The reduction of the concrete contribution to the splitting capacity for an anchorage length of 50 mm on average will be 0.933.

The splitting capacity of a 50 mm bar can then be estimated as
$2\pi*0.05m*0.015m*0.933*f_{ct20}$ = 13.2 kN, if the tensile strength of the concrete at 20°C is assumed to be f_{ct20} = 3.0 MPa.
The bond strength for a 50 mm Ø10 mm bar can be estimated as
$0.05m*0.979*1.3*f_{cc20}*\pi*D/2$ = 19.9 kN, if the concrete compressive strength is f_{cc20} = 19.9 MPa at 20°C.
For a slab with 8 reinforcing bars, the maximum shear capacity will be 8*13.2 = 105.2 kN, where the load is 25 kN for a 5.5 m slab

If the preconditions for this calculation are valid, no shear failure should be seen in a massive slab of a expanded clay aggregate concrete of quality 1800 kg/m^3 with a 5.5 m span and a with of 1.2 m loaded with 13.25 kN in each of two quarter points even if the anchorage length is reduced to 50 mm in the second phase from 200 mm in the first phase.
One precondition is that the support is uniformly distributed at the 50 mm and that the supporting construction is of the same thermal quality as the slab.

Slab elements

In phase 1 of full-scale tests (in the following marked 2000) a massive slab and a sandwich slab were tested both having a depth at support of 200 mm. The ultimate moment was shown to be decisive for the load bearing capacity, and this varied as shown on the following figures according to the calculations.

After these tests have been made the testing lab made some tests on prestressed heavy concrete slabs, which proved to fail very quickly after the start of the fire test. This gave reason to some debate, although there was a simple and predominant reason for the early failure: the slabs rested on bearing knots, and no concrete was casted between them. Therefore the knots were able to expand side-wards and splitting cracks could develop along the reinforcing bars leading to a tension shear failure.
Calculations based on the principles shown in the previous chapter show that an early failure should occur due to splitting even if the good bond properties of deformed bars were used. The debate caused by these tests seem therefore to be quite irrelevant, although the bond strength of the prestressed wire still have to be determined if a precise calculation of the anchorage capacity should be made where splitting will not occur, i.e. where concrete has been casted properly between the bearing knots.

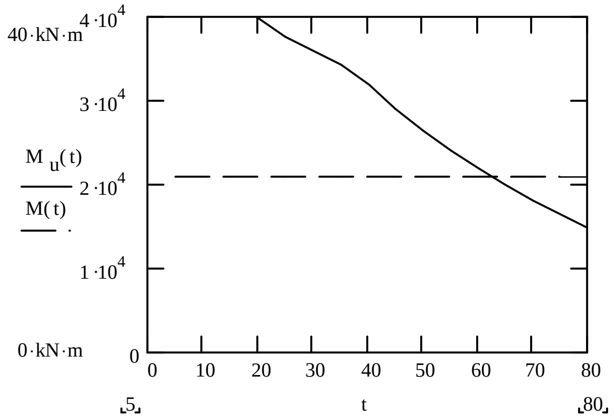
However, caused by the debate, there was an understandable wish to demonstrate that a usual depth of 70 mm at the bearing is sufficient for the light weight aggregate slabs. Therefore a calculation of the anchorage capacity of the deformed bars was made showing that 70 mm should be more than sufficient to avoid a shear tension failure.
The two decks were placed at the oven with the calculated bearing depth of only 70 mm, and both of them clearly failed in bending almost at the prescribed time proving that the shear and anchorage capacity can be calculated for these fire exposed decks.

During the testing some vertical tensile cracks were observed above the level of the reinforcement at the end sections of the slabs. These cracks did also occur inbetween the reinforcing bars and therefore they can not be initiated be them.
It was observed from the color of the concrete how the moisture were conducted by the cracks.
The obvious reason for the development of the cracks is thermal stresses, where this depth above the bottom is subjected to tension while the bottom is compressed laterally during the test. Later during the cooling phase after the test the picture is reversed, and the cracks penetrate down to the bottom surface.

The tests of phase 1 are reported in Andersen [10] and of phase 2 in Andersen [14] and [15].

Sandwich slab 2000 phase 1

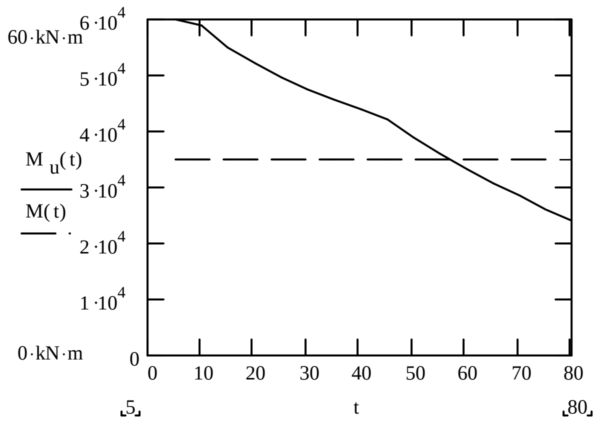
1.2 m wide consisting of 23 mm of 1550 kg/m^3 and f_{cc20} = 17 MPa at top, 182 mm of 625 kg/m^3 and f_{cc20} = 2.8 MPa and f_{ct20} = 1.0 MPa at middle and 35 mm of 1500 kg/m^3 and f_{ct20} = 3.0 MPa and λ=0.6 W/mK at bottom.
The span was L=5.5 m and cover thickness d = 15 mm at 8 Y 8 mm bars of 550 MPa.
The deck was loaded by 8.33 kN (2.5 kN/m^2) at a each 1/4 point of the slab. 200 mm support.



The calculated fire resistance time was 63 minutes.
The tested fire resistance time was more than 61 minutes.

Solid slab 2000 phase 1

1.2 m wide consisting of 200 mm of 1775 kg/m^3 and f_{cc20} = 19.9 MPa and f_{ct20} = 3.0 MPa and λ=0.6 W/mK at bottom.
The span was L=5.5 m and cover thickness d = 15 mm at 8 Y 10 mm bars of 550 MPa.
The deck was loaded by 13.25 kN (4.0 kN/m^2) at a each 1/4 point of the slab. 200 mm support.



The calculated fire resistance time was 57 minutes.
The tested fire resistance time was 61 minutes.

Sandwich slab Phase 2 at 70 minutes. Deflection and support.

Sandwich slab 2002 phase 2

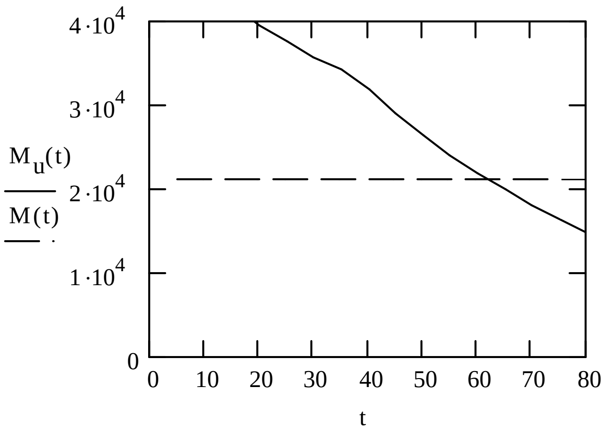
1.2 m wide consisting of 23 mm of 1550 kg/m^3 and f_{cc20} = 15.25 MPa at top, 182 mm of 625 kg/m^3 and f_{cc20} = 2.8 MPa and f_{ct20} = 0.3 MPa at middle and 35 mm of 1500 kg/m^3 and f_{ct20} = 2.7 MPa and λ=0.6 W/mK at bottom.
The span was 5.63 m and cover thickness d = 15 mm at 8 Y 8 mm bars of 550 MPa.
The slab was loaded by 8.11 kN (2.4 kN/m^2) at a each 1/4 point of the slab.
At the time 60 minutes the reinforcement temperature was measured to be 518°C and 486°C which gives an average of 502°C, where the calculation has foreseen 503°C. The calculation results are given below, where Q_u means the ultimate shear force and Q the shear load. 70 mm support. Fire resistance time at test was 73 minutes.

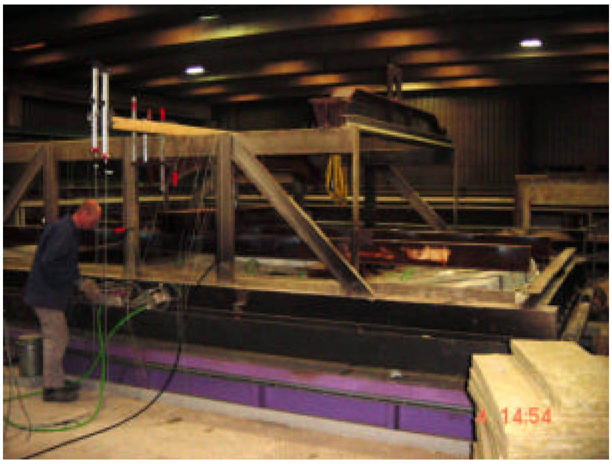
$Q(t) = 15\,kN$ $\quad M(t) = 21.2\,kN\cdot m$

$Q_u(61) = 36\,kN$ $\quad M_u(61) = 21.4\,kN\cdot m$ $\quad$ Calculated fire resistance time **61 minutes**

t =	$Q_u(t)$ = (kN)	$M_u(t)$ = (kN·m)
5	38.7	47.54
10	38.7	45.17
15	38.8	42.00
20	38.9	39.49
25	38.9	37.42
30	39.0	35.67
35	39.0	34.16
40	38.4	31.80
45	37.8	28.94
50	37.3	26.36
55	36.8	24.00
60	36.4	21.84
65	35.9	19.85
70	35.5	17.99
75	35.2	16.26
80	34.8	14.64

(An increase of cover thickness from 15 to 25 mm would give rise to an increase of fire resistance time to 103 minutes.)





Solid slab Phase 2 at 80 min.
Deflection and support.

Solid slab 2002 phase 2

1.2 m wide consisting of 200 mm of 1775 kg/m^3 and f_{cc20} = 20 MPa and f_{ct20} = 3.2 MPa and λ=0.6 W/mK at bottom.
The span was 5.63 m and cover thickness d = 15 mm at 8 Y 10 mm bars of 550 MPa.
The deck was loaded by 8.11 kN (2.4kN/m^2) at a each 1/4 point of the slab.
At the time 60 minutes the reinforcement temperature was measured to be 452°C, where the calculation has foreseen 465°C. The calculation results are given below, where Q_u means the ultimate shear force and Q the shear load. 70 mm support.
Fire resistance time at test was 79 minutes.

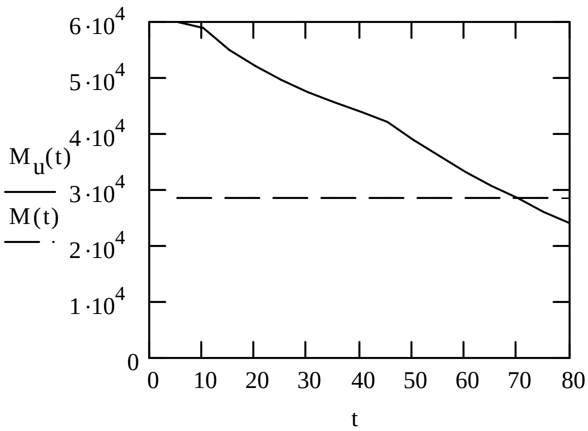
$Q(t) = 20.1\,kN$ $M(t) = 28.4\,kN \cdot m$ $t := 5, 10..\, 80$

$Q_u(69) = 112\,kN$ $M_u(69) = 28.7\,kN \cdot m$

Calculated fire resistance time **69 minutes**

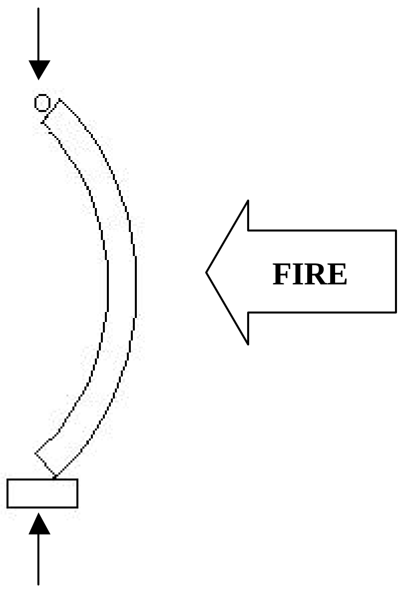
t =	$Q_u(t)$ = (kN)	$M_u(t)$ = (kN·m)
5	160.5	59.72
10	150.7	58.62
15	142.8	54.95
20	137.1	51.95
25	132.5	49.43
30	128.8	47.28
35	125.7	45.39
40	122.9	43.73
45	120.5	42.10
50	118.4	38.85
55	116.5	35.87
60	114.7	33.13
65	113.1	30.59
70	110.9	28.22
75	108.2	26.00
80	105.6	23.92

(An increase of cover thickness from 15 to 25 mm would give rise to an increase of fire resistance time to 114 minutes.)



As seen all tests show a good agreement with the ultimate load-bearing capacities calculated for shear, anchorage and bending, and it is possible to conclude that the calculation methods seem to be well documented for bending as well as for shear and anchorage failure of these light weight aggregate slabs exposed to fire.

Wall elements



In order to calculate the load bearing capacity of a fire exposed concrete wall, some general problems have to be solved. Because the wall is exposed to fire at only one side, it will deflect into the fire, giving rise to a considerable eccentricity, which must be taken into account.

At the same time the concrete section is damaged at the fire exposed side giving an eccentricity counteracting the eccentricity of the thermal deflection.

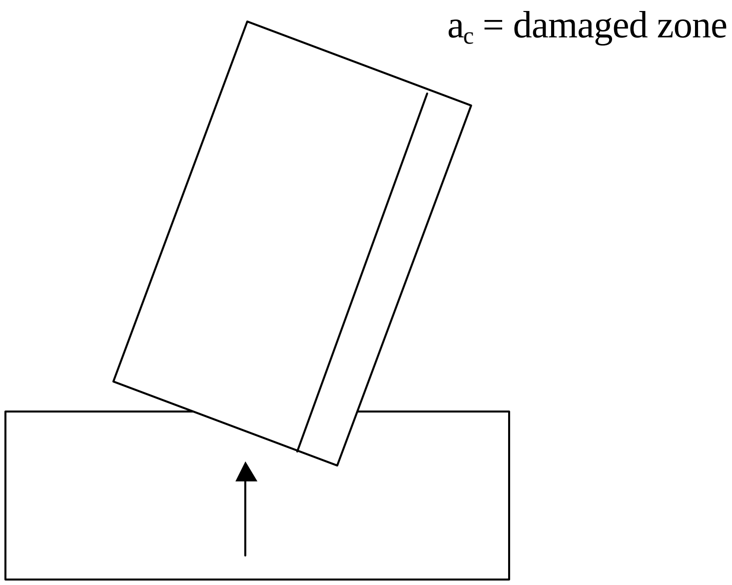
Both effects will influence the distribution of stresses in each time step of the fire, and the resulting stress distribution again determines the transient thermal expansion. This is the free thermal expansion minus the strain, which can not take place because the concrete is loaded.

This means that the stress distribution at a certain time is a function of the deflection but also influences the new thermal expansion and thereby the new deflection and the new stress distribution.

It was therefore concluded in the first phase of the project, that a calculation has to be made in time steps. And the walls tested in this part of the project had deliberately an initial eccentricity of the load, which gave compression towards the fire and therefore contributed the most to the transient strain. On the other hand this eccentricity also gave the most stable conditions because the external load counteracted the thermal deflection.

In the second phase it was therefore decided to use load with an eccentricity away from the fire, which tend to increase the thermal deflection, and which must be expected to be the worst case and therefore the decisive load case for a wall.

Doing this the test results would not only serve as a check for the calculations, but can also be used as a direct documentation for the application of the specific walls.

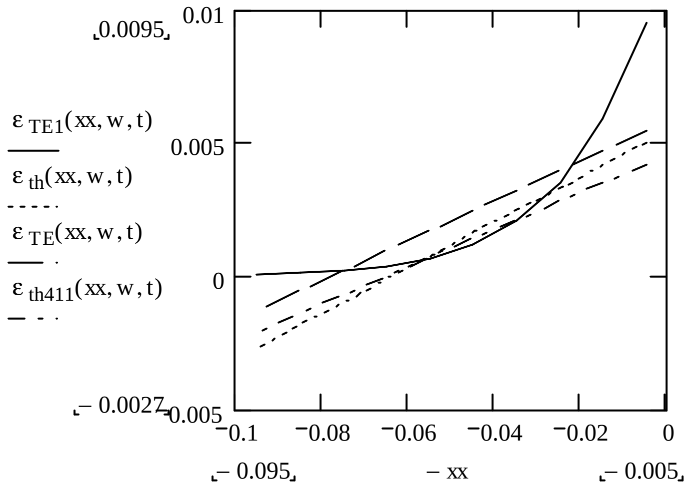


In the report of the first phase the support was modeled as a hinge at top and bottom, but it was learned that in the test the walls had rested flat at the bottom support. This gives rise to an eccentricity, which was not calculated correct. Since it seems to be the most correct model of the support in practice the supporting conditions were maintained in the test of the second phase, but the calculation model was modified in order to model it, and a quite new calculation method was derived, and it was used calculating all the tests. This means that the tests of phase 1 were recalculated with the new border conditions, and that the calculations are not equal to those in the first report [2].
The resulting eccentricity at the bottom is found as follows: First the width of the supporting strip is calculated as the load divided by the compressive strength of the reduced cross section. Then the eccentricity from the middle of this strip to the centre of the reduced cross section is calculated, where the strip starts in the depth of the damaged zone from the fire exposed side of the wall. This gives the moment load and the curvature at the bottom.

The curvature at top κ_t and at bottom κ_b is determined, at the deflection caused by this is calculated as a function of the height z as

$$u(z, \kappa_t, \kappa_b, L) := (L - z)\cdot\frac{z}{6\cdot L}\cdot\left[(2\cdot L - z)\cdot\kappa_t + (L + z)\cdot\kappa_b\right]$$

where L is the total height of the wall.

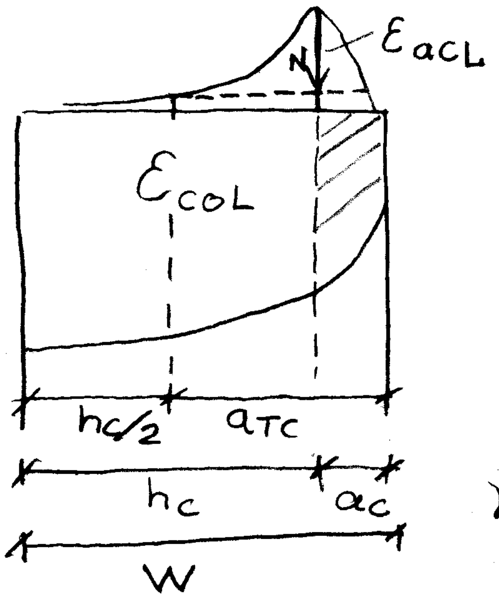


In order to find a reliable method of calculating the thermal curvature, a wall was divided into 10 lamellas, and the temperature, the initial thermal strain ε_{TE1} and the strength reduction and stiffness reduction was calculated for each lamella. The resulting curvature for a plane cross section with internal thermal stresses was calculated ε_{TE}, and compared to a more simple expressions such as the one given as a guide line for columns in DS411 [3] ε_{th411} and the proposed expression ε_{th} used in this report. The results is shown above for a 100 mm wall of quality 1800 kg/m^3 at the time 60 minutes.

The expression used here and in the following calculations for the increase of the thermal curvature in a time step is

$$\frac{-2\cdot(\Delta\varepsilon acL\cdot ptraL - \Delta\varepsilon c0L\cdot ptr0(w,tL))\cdot a_{Tc}(w,tL)}{h_c(w,tL)^2}$$

w is the width of the cross section, and tL the time.
ΔεacL is the increase of the free thermal strain in the depth of the inner edge of the damaged zone, Δεc0L is the increase of the free thermal strain at the middle of the reduced cross section, ptraL and ptr0 are the reductions of these due to transiens, a_{Tc} is the depth of the centre of the reduced cross section from the fire exposed surface, and h_c is the with of the reduced cross section.



Force from hindered expansion

$$N \sim \frac{1}{3}(\varepsilon_{acL} - \varepsilon_{coL}) \cdot a_{Tc} \cdot E$$

Moment from this

$$M \sim \frac{1}{3}(\varepsilon_{acL} - \varepsilon_{coL}) a_{Tc} E \frac{h_c}{2}$$

Curvature from this

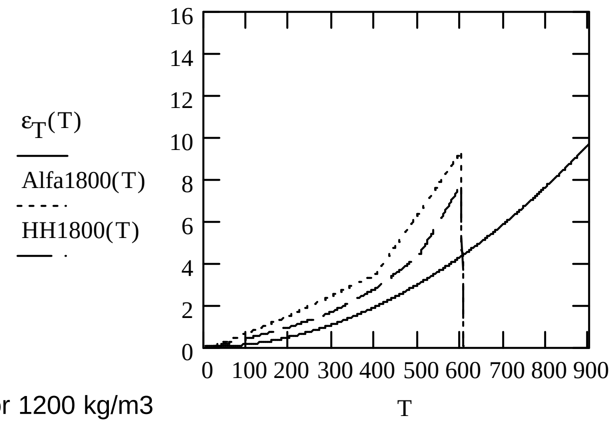
$$\kappa \sim \frac{\frac{1}{6}(\varepsilon_{acL} - \varepsilon_{coL}) a_{Tc} E h_c}{\frac{1}{12} h_c^3 E} = \frac{2(\varepsilon_{acL} - \varepsilon_{coL}) a_{Tc}}{h_c^2}$$

The formula is derived assuming, that the transient thermal strains (or the equivalent stresses if hindered) are distributed following a parabola from the centre of the reduced cross section to the edge of the damaged zone, and reduced from this level and out to the surface, such that the area under the curve in the last part (in the damaged zone) is equal to the area in the first part.
The moment of the hindered thermal stresses is found and divided by the stiffness of the reduced cross section.

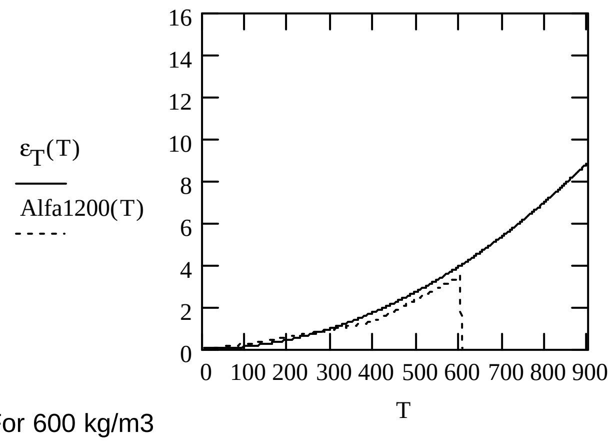
As seen from the graph, the expression gives a good approximation to the more complicated calculated curvature.

Since the thermal deflection is outmost important for the calculation, it has been necessary to adjust the simple formula ($1.1*10^{-5}*T$) previously used for most concretes, and in stead use a parabolic expression, which gives a better fit to the observed values. The thermal expansion has been measured at the beginning of the project by different producers, and the following approximate formulas are derived for the 3 qualities.
In the graphs the expressions are compared to measured expansions.

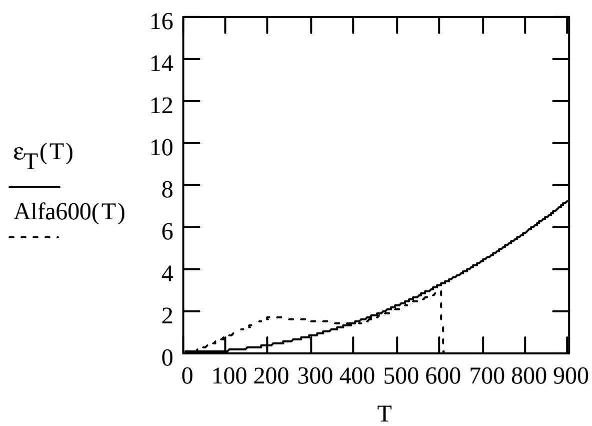
For 1800 kg/m3 $\varepsilon_T(T) := 1.2 \cdot 10^{-5} \cdot (T)^2$ per 1000



For 1200 kg/m3 $\varepsilon_T(T) := 1.1 \cdot 10^{-5} \cdot (T)^2$ per 1000



For 600 kg/m3 $\varepsilon_T(T) := 0.9 \cdot 10^{-5} \cdot (T)^2$ per 1000



In the following the calculations of the eccentric loaded walls are compared to the results from the full-scale tests. The tests called 2001 are those from phase 1 are all of them have an initial eccentricity towards the fire. This gives the most complicated technical problem, but is on the safe side compared to an eccentricity away from the fire, which must be regarded as decisive, and therefore this was used the tests of phase 2.
The tests of phase 1 are reported in Andersen [6]-[9], and of phase 2 in Andersen [11]-[13].

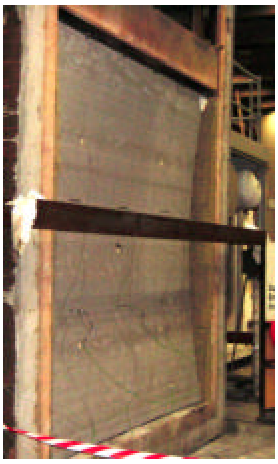
All test were made with a flat foot bearing at the bottom except the 1200 kg/m^3 wall of phase 2. It was the idea that this wall should have a hinge at the bottom in order to verify the calculation method for this simpler border condition. However, the hinge constructed by the testing lab proved not to be able to incline sufficient, and after 45 minutes the bottom could be regarded as a flat foot support for this element also. This is taken into account in the successive calculation, where the moment at the bottom is assessed to be fixed during the first 45 minutes, and following the flat foot principle thereafter.

The 3 m high wall of quality 600 kg/m^3 of phase 1 failed after only 36 minutes. Calculating the development of the load bearing capacity of this wall, the resistance time is found to be larger. The author has not been present at the tests of phase 1, but it can be seen from the photos that the wall is made of blocks without a plaster added to the surface, such as it has been done at phase 2. Therefore, it can be assumed, that one reason for the difference between calculation and test is the missing filling of edges of the joints between the blocks. A second calculation is therefore made where the joints are presumed to miss 10 mm filling at the surface, reducing the cross section used for the calculation of the Navier load bearing capacity.

At phase 2 the 600 kg/m^3 wall was only 2.4 m high, loaded with only 7.5 kN/m, but with an eccentricity away from the fire. In this case the wall proved to have a fire resistance of more than 120 minutes, and the calculation is in agreement with the test.

The graphs show the calculated load bearing capacity F_{ult} as a function of time, and the calculated deflection at the middle of the wall u_m and the eccentricity of the load related to the centre of the reduced cross section e_{una} and the measured deflections D.
All calculations are made in time steps of 10 minutes, and each page compare a calculation with a full-scale test.

It seems that there is a good agreement between calculations and test results, and that the tests can serve as documentation for the calculation methods.



1800 kg/m3, 2002 Phase 2

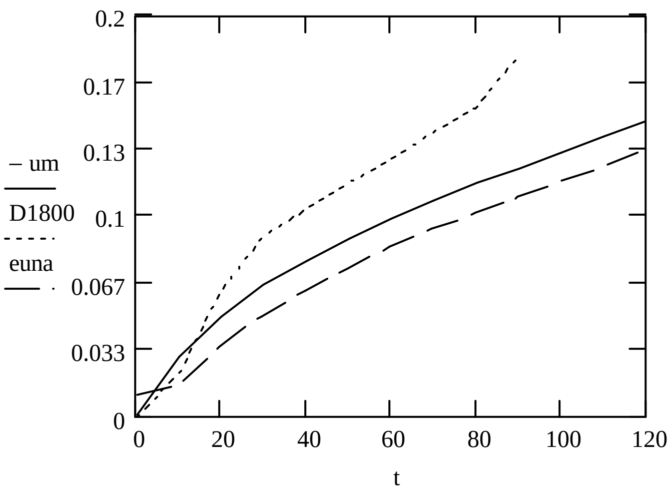
Wall supported by a hinge at top and a flat foot at the bottom 2002-10-30

Load	$P = 40.0$ kN/m		$f_{cc20} = 20.00$ MPa	$\lambda = 0.90$	W/mC
Width of wall	$w = 0.100$ m		$E_{c20} = 18.00$ GPa	$\rho = 1800$	kg/m3
Eccentricity top	$e_{top} = -0.020$ m	(Positive towards the fire)	$f_{ct20} = 3.20$ MPa	$c_p = 1000$	kJ/kgC
Height	$L = 3.00$ m		$t_{max} = 120$ min	$\Delta t = 10$	min

Test stoped at 85 minutes 168 mm deflection 40 kN/m   Thermal expansion e(T) = 1.2*T2

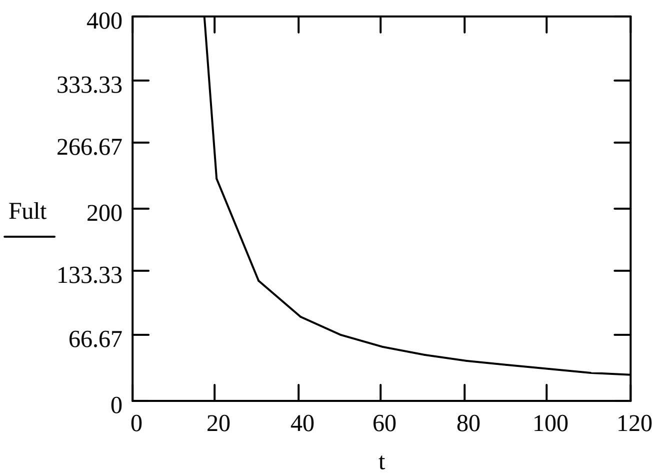
Deflection in m calculated um, internal euna and measured D and Fult in kN/m as function of time t in min.

$t_i =$	$um_i =$	$Fult_i =$
0	-0.000	902.6
10	-0.030	809.7
20	-0.050	230.2
30	-0.065	124.0
40	-0.078	87.1
50	-0.089	68.0
60	-0.099	56.1
70	-0.108	47.6
80	-0.116	41.6
90	-0.124	36.3
100	-0.132	31.6
110	-0.139	28.3
120	-0.148	25.4



$$t_u := 80 + \frac{(41.6 - 40)}{41.6 - 36.3} \cdot 10 \qquad t_u = 83$$

Calculated resistance time 83 minutes



Photos after 80 minutes and at the time of brake after 85 minutes.

1800 kg/m3, 2001 Phase 1

Wall supported by a hinge at top and a flat foot at the bottom 2002-10-30

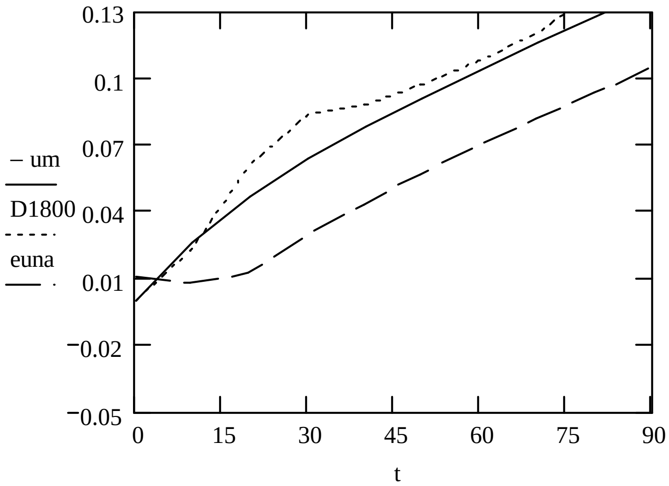
Load	$P = 70.0$	kN/m		$f_{cc20} = 20.00$	MPa	$\lambda = 0.90$	W/mC
Width of wall	$w = 0.100$	m		$E_{c20} = 18.00$	GPa	$\rho = 1800$	kg/m3
Eccentricity top	$e_{top} = 0.020$	m	(Positive towards the fire)	$f_{ct20} = 3.20$	MPa	$c_p = 1000$	kJ/kgC
Height	$L = 3.00$	m		$t_{max} = 120$	min	$\Delta t = 10$	min

Test stoped at 78 minutes 137 mm deflection 70 kN/m

Thermal expansion e(T) = 1.2*T2

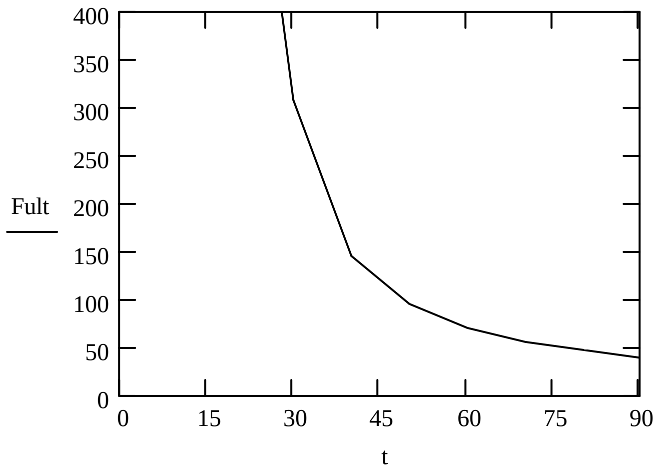
Deflection in m calculated um, internal euna and measured D and Fult in kN/m as function of time t in min.

$t_i =$	$um_i =$	$Fult_i =$
0	0.000	902.6
10	-0.026	809.7
20	-0.047	768.3
30	-0.063	307.5
40	-0.078	144.9
50	-0.091	95.2
60	-0.104	70.7
70	-0.116	55.7
80	-0.127	45.9
90	-0.139	38.0
100	-0.151	31.6
110	-0.164	27.0
120	-0.179	23.2



$$t_u := 60 + \frac{(70.0 - 70)}{70.7 - 55.7} \cdot 10 \qquad t_u = 60$$

Calculated resistance time 60 minutes



1200 kg/m3, 2002 Phase 2

Wall supported by a hinge at top and a flat foot at the bottom 2002-10-30

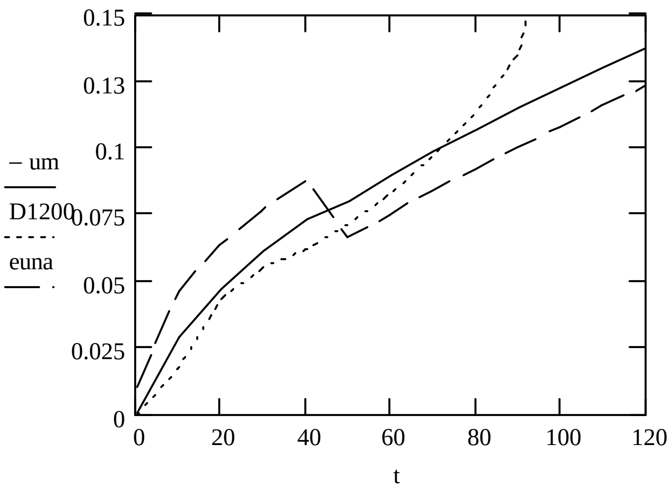
Load	$P = 25.0$	kN/m		$f_{cc20} = 10.50$	MPa	$\lambda = 0.45$	W/mC
Width of wall	$w = 0.100$	m		$E_{c20} = 8.00$	GPa	$\rho = 1200$	kg/m3
Eccentricity top	$e_{top} = -0.020$	m	(Positive towards the fire)	$f_{ct20} = 2.20$	MPa	$c_p = 1000$	kJ/kgC
Height	$L = 3.00$	m		$t_{max} = 180$	min	$\Delta t = 10$	min

Test stoped at 96 minutes 173 mm deflection 25 kN/m

Thermal expansion $e(T) = 1.1*T2$

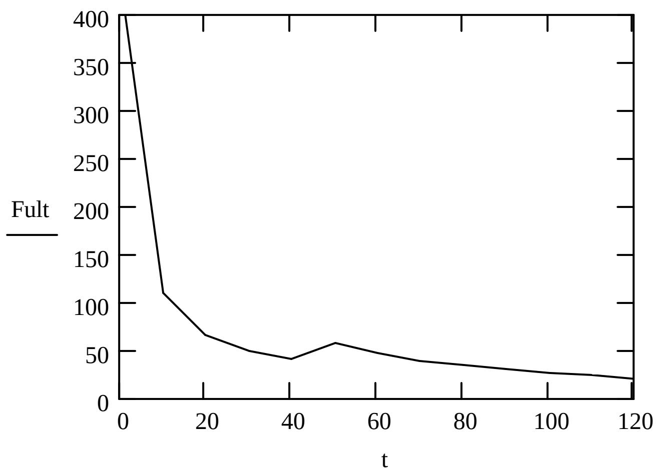
Deflection in m calculated um, internal euna and measured D and Fult in kN/m as function of time t in min.

$t_i =$	$um_i =$	$Fult_i =$	$D1200_r =$
0	-0.000	431.0	0.000
10	-0.029	108.8	0.018
20	-0.047	66.2	0.043
30	-0.061	49.7	0.055
40	-0.073	40.6	0.062
50	-0.080	56.5	0.072
60	-0.090	46.0	0.083
70	-0.098	38.9	0.097
80	-0.107	33.7	0.114
90	-0.115	29.6	0.136
100	-0.122	26.0	0.210
110	-0.130	23.1	
120	-0.138	20.5	



$$t_u := 100 + \frac{(26 - 25)}{26 - 23.1} \cdot 10 \qquad t_u = 103.4$$

Calculated resistance time 103 minutes



Photos of the wall after 95 minutes from outside and after the test from inside the furnace.

1200 kg/m3, 2001 Phase 1

Wall supported by a hinge at top and a flat foot at the bottom 2002-10-30

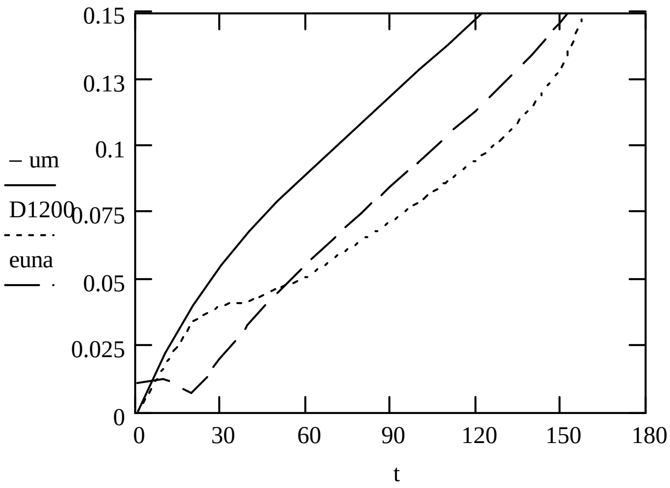
Load	$P = 35.0$	kN/m		$f_{cc20} = 10.50$	MPa	$\lambda = 0.45$	W/mC
Width of wall	$w = 0.100$	m		$E_{c20} = 8.00$	GPa	$\rho = 1200$	kg/m3
Eccentricity top	$e_{top} = 0.020$	m	(Positive towards the fire)	$f_{ct20} = 2.20$	MPa	$c_p = 1000$	kJ/kgC
Height	$L = 3.00$	m		$t_{max} = 180$	min	$\Delta t = 10$	min

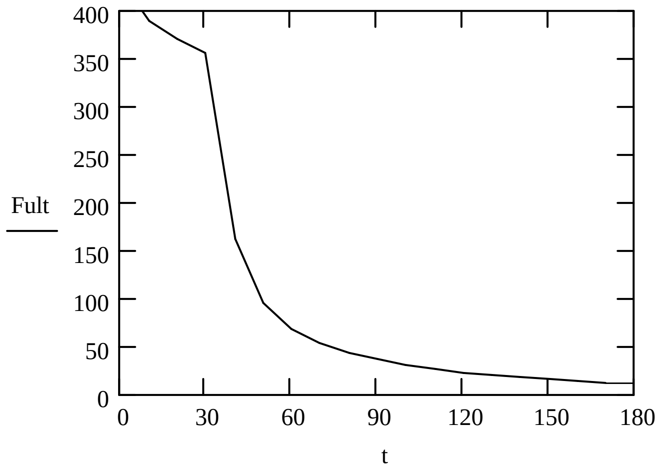
Test stoped at 154 minutes 130 mm deflection 35 kN/m

Thermal expansion e(T) = 1.1*T2

Deflection in m calculated um, internal euna and measured D and Fult in kN/m as function of time t in min.

$t_i =$	$um_i =$	$Fult_i =$	$D1200_r =$
0	0.001	431.0	0.000
10	-0.022	389.3	0.018
20	-0.040	370.7	0.034
30	-0.055	354.9	0.040
40	-0.068	160.5	0.042
50	-0.079	95.4	0.046
60	-0.090	68.1	0.051
70	-0.100	52.8	0.058
80	-0.109	43.1	0.065
90	-0.119	36.0	0.072
100	-0.128	30.5	0.079
110	-0.138	26.1	0.087
120	-0.148	22.5	0.095
130	-0.158	19.4	0.104
140	-0.168	17.2	0.115
150	-0.181	15.2	0.129
160	-0.194	13.5	0.154
170	-0.209	12.0	
180	-0.224	10.6	





$$t_u := 90 + \frac{(36 - 35)}{36 - 30.5} \cdot 10 \qquad t_u = 91.8$$

Calculated resistance time 91 minutes

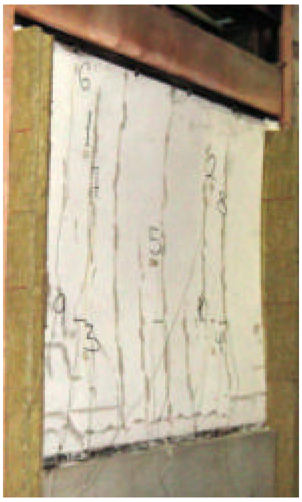


Photo wall 600 after 124 min

600 kg/m3, 2002 Phase 2

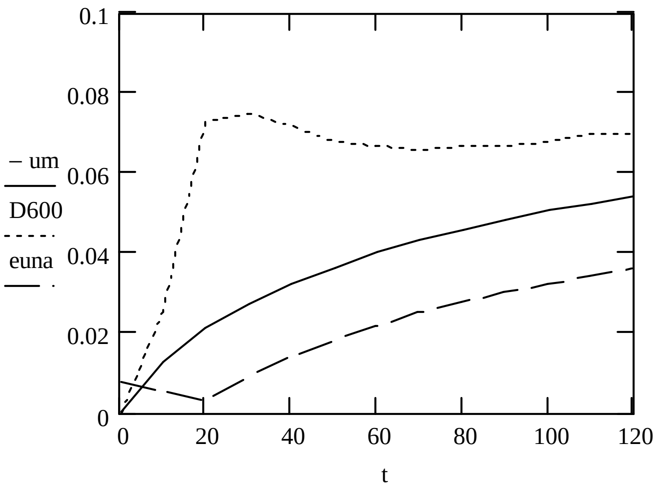
Wall supported by a hinge at top and a flat foot at the bottom 2002-10-30

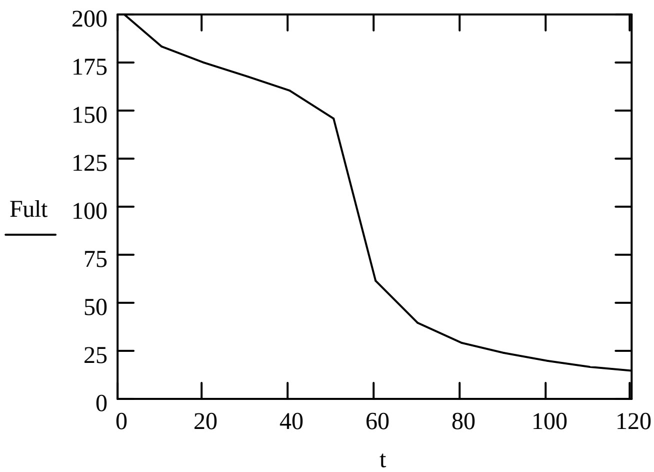
Load	$P = 7.5$	kN/m		$f_{cc20} = 3.75$	MPa	$\lambda = 0.30$	W/mC
Width of wall	$w = 0.105$	m		$E_{c20} = 2.50$	GPa	$\rho = 600$	kg/m3
Eccentricity top	$e_{top} = -0.015$	m	(Positive towards the fire)	$f_{ct20} = 0.30$	MPa	$c_p = 1000$	kJ/kgC
Height	$L = 2.40$	m		$t_{max} = 200$	min	$\Delta t = 10$	min

Test stoped at 120 minutes 70 mm deflection loaded up to 17.5 kN/m Thermal expansion e(T) = 0.9*T2

Deflection in m calculated um, internal euna and measured D and Fult in kN/m as function of time t in min.

$t_i =$	$um_i =$	$Fult_i =$	$D600_r =$
0	-0.000	201.6	0.000
10	-0.013	182.6	0.026
20	-0.021	174.2	0.073
30	-0.027	166.9	0.075
40	-0.032	160.3	0.072
50	-0.036	145.4	0.068
60	-0.040	60.6	0.067
70	-0.043	39.0	0.066
80	-0.046	29.1	0.067
90	-0.048	23.2	0.067
100	-0.051	19.1	0.068
110	-0.052	16.1	0.070
120	-0.054	14.2	0.070
130	-0.056	12.6	
140	-0.058	11.3	
150	-0.060	10.1	
160	-0.062	9.3	
170	-0.064	8.7	
180	-0.065	8.0	
190	-0.066	7.4	
200	-0.068	6.8	





At 120 minutes the wall is calculated to have a load bearing capacity of 14.2 kN/m, and it is calculated to fail after 188 minutes

600 kg/m3, 2001 Phase 1

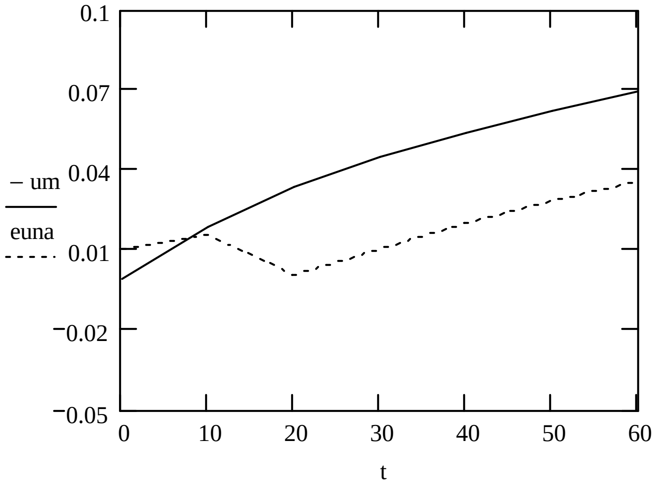
Wall supported by a hinge at top and a flat foot at the bottom 2002-10-30

Load	$P = 10.0$	kN/m		$f_{cc20} = 3.75$	MPa	$\lambda = 0.30$	W/mC
Width of wall	$w = 0.100$	m		$E_{c20} = 2.50$	GPa	$\rho = 600$	kg/m3
Eccentricity top	$e_{top} = 0.020$	m	(Positive towards the fire)	$f_{ct20} = 0.30$	MPa	$c_p = 1000$	kJ/kgC
Height	$L = 3.00$	m		$t_{max} = 120$	min	$\Delta t = 10$	min

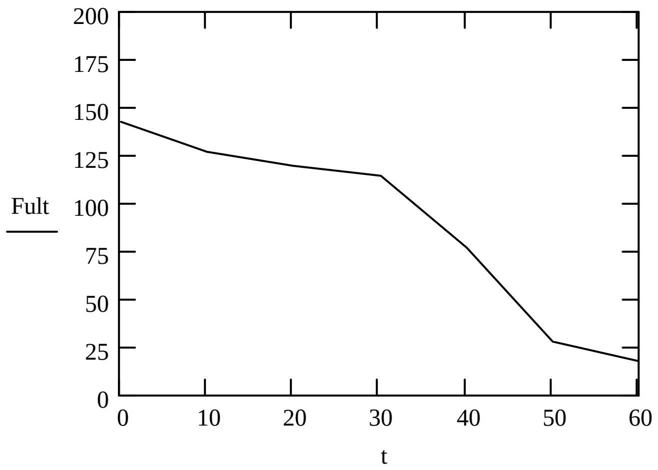
Test stoped at 36 minutes ? mm deflection loaded up to 10.0 kN/m   Thermal expansion e(T) = 0.9*T2

Deflection in m calculated um, internal euna and measured D and Fult in kN/m as function of time t in min.

$t_i =$	$um_i =$	$Fult_i =$
0	0.001	142.0
10	-0.019	126.4
20	-0.033	119.5
30	-0.045	113.8
40	-0.054	76.8
50	-0.062	27.9
60	-0.070	17.2
70	-0.077	12.4
80	-0.083	9.8
90	-0.089	7.9
100	-0.095	6.5
110	-0.100	5.6
120	-0.107	4.8



Calculated resistance time 79 minutes



600 kg/m3, 2001 Phase 1 corrected for joints by reduced thickness

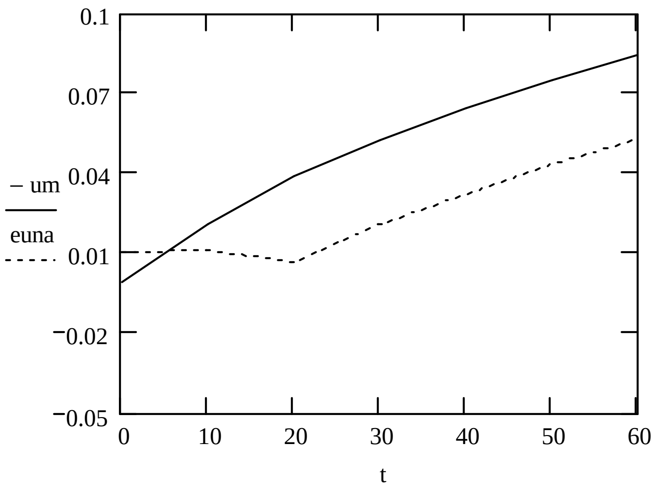
Wall supported by a hinge at top and a flat foot at the bottom 2002-10-30

Load	$P = 10.0$	kN/m		$f_{cc20} = 3.75$	MPa	$\lambda = 0.30$	W/mC
Width of wall	$w = 0.090$	m		$E_{c20} = 2.50$	GPa	$\rho = 600$	kg/m3
Eccentricity top	$e_{top} = 0.020$	m	(Positive towards the fire)	$f_{ct20} = 0.30$	MPa	$c_p = 1000$	kJ/kgC
Height	$L = 3.00$	m		$t_{max} = 120$	min	$\Delta t = 10$	min

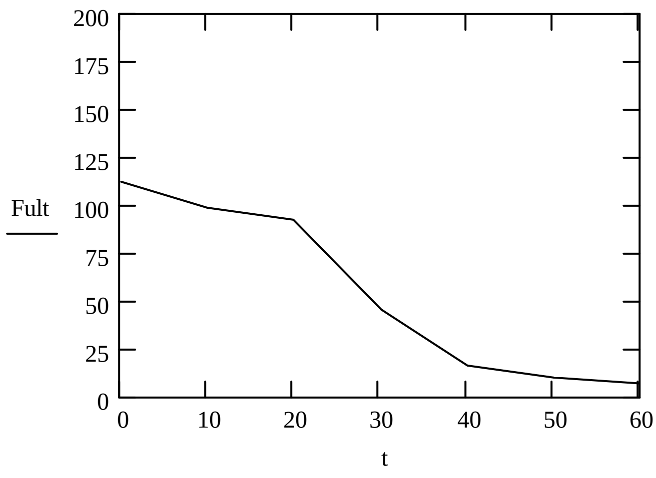
Test stoped at 36 minutes ? mm deflection loaded up to 10.0 kN/m   Thermal expansion e(T) = 0.9*T2

Deflection in m calculated um, internal euna and measured D and Fult in kN/m as function of time t in min.

$t_i =$	$um_i =$	$Fult_i =$
0	0.001	111.5
10	-0.021	98.0
20	-0.039	91.9
30	-0.052	45.5
40	-0.064	16.3
50	-0.075	10.0
60	-0.085	7.2
70	-0.093	5.6
80	-0.102	4.5
90	-0.111	3.7
100	-0.122	3.1
110	-0.133	2.6
120	-0.143	2.2



Calculated resistance time 50 minutes



900 kg/m3, 1999

Wall of blocks supported by a hinge at top and flat foot at bottom 2002-10-30

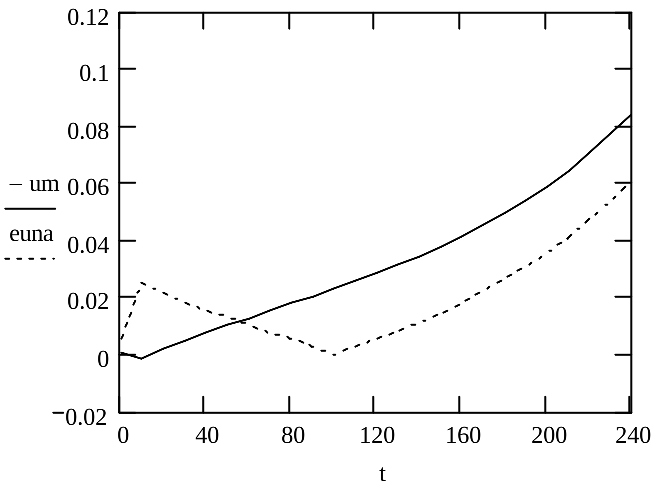
Load	$P = 108.0$	kN/m		$f_{cc20} = 3.00$	MPa	$\lambda = 0.40$	W/mC
Width of wall	$w = 0.150$	m		$E_{c20} = 3.00$	GPa	$\rho = 900$	kg/m3
Eccentricity top	$e_{top} = -0.010$	m	(Positive towards the fire)	$f_{ct20} = 1.00$	MPa	$c_p = 1000$	kJ/kgC
Height	$L = 2.50$	m		$t_{max} = 240$	min	$\Delta t = 10$	min

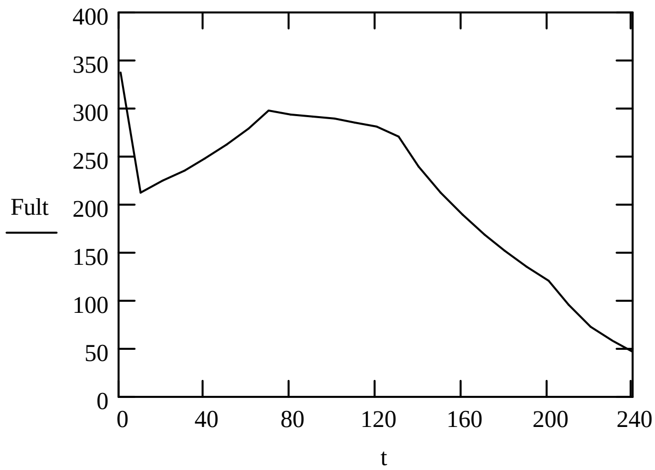
Test stoped at 198 minutes -5 mm deflection 108 kN/m

Thermal expansion e(T) = 1.0*T2

Deflection in m calculated um, internal euna and Fult in kN/m as function of time t in min.
FNavt is Navier tension and FNavc is Navier compression criterion and FR is the rankine capacity.

$t_i =$	$um_i =$	$Fult_i =$	$FNavt_i =$	$FNavc_i =$	$FR_i =$
0	-0.001	336.4	0	0	336
10	0.002	211.6	3554	212	316
20	-0.002	223.2	-1512	223	312
30	-0.005	234.5	-665	235	309
40	-0.008	247.1	-430	247	305
50	-0.010	260.7	-319	261	302
60	-0.013	277.1	-252	277	299
70	-0.015	296.1	-207	296	296
80	-0.018	293.2	-175	319	293
90	-0.020	290.6	-150	348	291
100	-0.023	287.9	-131	386	288
110	-0.026	284.7	-145	345	285
120	-0.028	280.5	-168	304	281
130	-0.031	269.1	-206	269	277
140	-0.035	238.7	-274	239	273
150	-0.038	211.9	-438	212	269
160	-0.041	188.5	-1363	189	265
170	-0.045	168.1	995	168	261
180	-0.050	150.0	339	150	257
190	-0.054	133.7	193	134	253
200	-0.059	119.2	130	119	250
210	-0.065	94.7	95	106	246
220	-0.071	72.0	72	94	241
230	-0.077	56.3	56	83	237
240	-0.085	45.0	45	74	233





$$t_u := 200 + \frac{(119.2 - 108)}{119.2 - 94.7}\cdot 10 \quad t_u = 204.6$$

Calculated resistance time 204 minutes

Notice that the internal eccentricity e_{una} of this wall first grows, then declines and finally grows again, and that the ultimate capacity is first caused by compression failure (FNavt), then Rankine instability (FR) and finally tension Navier failure (FNavt).

Conclusions

The simplified temperature calculation method seems to be verified by comparison with measured temperature profiles and developments in a special block wall with varying thickness and concrete quality and by comparison with temperature profiles and developments measured in the tested wall elements and at the reinforcing bars of the tested slabs. In addition the simplified method prove a good agreement with the results of finite difference calculations.
The simplified temperature calculation seems therefore to be well documented.

For **slabs** the calculations of ultimate **bending capacity** seems to be well documented, and it seems to be verified that even very small bearing depths can be foreseen to give a sufficient **shear and anchorage** resistance for deformed bars.

On the precondition that the calculations are made in reasonable time steps, and a detailed assessment is used for the transient thermal strain it can be concluded that there is a good agreement between calculations and test results for **walls**, and that the tests can serve as documentation for the calculation methods.

Further the test results can serve as a direct documentation for the actual slabs and walls with the prescribed loads.

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Appendix 1
Test data recorded manually by the author

Test data sandwich deck manually recorded at test 2002-01-30 at DIFT

Time	Deflection	Reinforcement temperature	Comments
min	mm	°C	
12	30	95	
20	59	150	
30	85	250	
40	118	360	
50	132	440	One piston replaced
60	143	502 (average of 518 and 486)	One piston replaced
70	160+217	561 (average of 577 and 545)	Accelerating deflection without load defines the brake

Test data massive deck manually recorded at test 2002-02-04 at DIFT

Time	Deflection	Temperatures						Comments
min	mm	°C						
10	38	71	60	80	76	63	78	
20	83,6	167	121	176	160	127	173	
30	114,8	282	194	262	249	204	264	
40	141,8	352	271	356	325	295	352	
50	166,6	404	332	416	385	361	411	
60	198,0	452	394	449	440	415	452	
70	226,6	473	434	474	481	458	491	
75	233							
78	239	503	449	500	509	482	518	
80	241							Accelerating deflection without load defines the brake After test vertical cracks were observed from the bottom at the supports also between the reinforcing bars. Two days later the cracks were closed. (Thermo cracks) During test moisture seems to spread from these cracks.

Test data for block wall 600 kg/m3 manually recorded at test 2002-01-30 at DIFT

Time	Deflection	Temperatures					Comments
min	mm	°C					
10	29	1	437	361	65	34	Hinge at top. Flat foot at bottom.
18	57	14	549	481	81	73	
20	73	58	621	561	118	80	
25	76	79	676	621	176	86	Vertical crack in the middle
30	77	80	713	663	220	107	
40	71	81	769	724	321	186	
50	68	88	814	769	396	260	
60	68	111	849	804	453	321	
70	67	150	881	840	505	377	
80	67	190	903	863	547	421	Horizontal crack 15 cm up
90	67	229	929	891	586	461	Horizontal crack 30 cm up
100	70	260	946	912	623	501	
110	72	300	966	962	654	533	Horizontal crack 45 cm up
120	72	328	976	946	683	562	

After 2 hours the wall was loaded up from 7.5 kN/m (150bar hydraulic pressure) and it broke into the oven at 17.5 kN/m (350 bar hydraulic pressure).

Test data for wall 1800 kg/m3 manually recorded at test 2002-02-04 at DIFT

Time	Deflection	Temperatures						Comments
min	mm	°C						
10	23	34	90	80	100	11	9	Hinge at top. Flat foot at bottom.
20	63	90	207	187	220	80	79	
25	80							Vertical cracks
30	88	159	297	274	300	83	86	
40	102	214	370	347	382	124	126	
50	115	267	427	404	439	168	170	
55	122	289	453	430	465	196	195	
60	128	341	475	453	488	218	215	
65	134							
70	141	356	522	500	536	259	252	
75	147							
80	154	395	557	535	569	296	284	
85	166							The wall broke into the oven. The line of fracture was 0.7 times the total height from the bottom.

Test data for wall 1200 kg/m3 manually recorded at test 2002-02-04 at DIFT

Time	Deflection	Temperatures						Comments
min	mm	°C						
10	20	33	93	78	108	12	16	Hinge at top and bottom at first.
15	35							
20	44	78	214	197	233	78	79	Vertical cracks
35	59							
40	63	185	365	348	384	127	156	
45	68							
50	73	243	426	408	445	177	207	
55	78							
60	84	286	473	455	493	217	250	
65	91							
70	98	330	520	502	540	255	291	
75	106							
80	114	370	562	544	582	297	332	
90	137	405	596	578	615	333	369	
95	159							The deflections accelerated and the test was stopped. It was observed that the bottom hinge was not capable of taking the maximum inclination A calculation show that the limit of movement has been reached at a deflection of 64 mm, i.e. after 45 minutes, where the support must be considered to be an inclined flat foot with eccentricity +50 mm in stead of -20 mm.

Appendix 2
Example of a slab calculation

Calculations of fire exposed light concrete structures. K. Hertz 2002-01-29

At this first page, the units are defined and the basic formulas are given for later use.

$N := 1 \cdot newton \quad kN := 10^3 \cdot N \quad MN := 10^6 \cdot N \quad kPa := 10^3 \cdot Pa \quad MPa := 10^6 \cdot Pa \quad GPa := 10^9 \cdot Pa$

$\lambda := 0.60$ W/mC $\quad \rho := 1775$ kg/m3 $\quad c_p := 1000$ J/kgC $\qquad k(t) := \sqrt{\dfrac{\pi \cdot \rho \cdot c_p}{750 \cdot \lambda \cdot t}}$

$$T_0(x,t) := 312 \cdot \log(8 \cdot t + 1) \cdot \exp(-1.9 \cdot k(t) \cdot x) \cdot \sin\left(\frac{\pi}{2} - k(t) \cdot x\right)$$

T1(x,t) is the temperature in the depth x of a semi infinite specimen at the time t calculated by the simple formula:

$$T_{10}(x,t) := \text{if}\left(x < \frac{\pi}{2 \cdot k(t)}, T_0(x,t), 0\right)$$

$$T_1(x,t) := \text{if}\left(T_{10}(x,t) > 20, T_{10}(x,t), 20\right)$$

T2(x,w,t) is the temperature in the depth x of a two sided exposed wall or web of thickness 2w at the time t:.

$$T_{20}(x,w,t) := \left(T_{10}(x,t) + T_{10}(2 \cdot w - x, t)\right) \cdot \frac{T_{10}(0,t)}{T_{10}(0,t) + T_{10}(2 \cdot w, t)}$$

$$T_2(x,w,t) := \text{if}\left(T_{20}(x,w,t) > 20, T_{20}(x,w,t), 20\right)$$

The reduction of the **0.2%** strength ξs(T) of the slack reinforcement:

$$\xi_{si}(T) := \text{if}(T \le 100, 1, 0) \quad \xi_{sj}(T) := \text{if}\left(100 < T \le 400, 1 - 0.3 \cdot \frac{T - 100}{300}, 0\right)$$

$$\xi_s(T) := \xi_{si}(T) + \xi_{sj}(T) + \text{if}\left(400 < T \le 650, 0.7 - 0.6 \cdot \frac{T - 400}{250}, 0\right) + \text{if}\left(650 < T \le 1200, 0.1 - 0.1 \cdot \frac{T - 650}{550}, 0\right)$$

The reduction of the compressive strength ξc(T) of a Danish light aggregate concrete:

$$\xi_c(T) := \text{if}(T \le 200, 1, 0) + \text{if}\left(200 < T \le 800, 1 - 0.4 \cdot \frac{T - 200}{600}, 0\right) + \text{if}\left(800 < T \le 1000, 0.6 - 0.6 \cdot \frac{T - 800}{200}, 0\right)$$

The reduction in each of 5 zones of the half of a two sided exposed wall w.

$$\xi_{c1}(w,t) := \xi_c\left(T_2\left(\frac{w}{10}, w, t\right)\right) \quad \xi_{c2}(w,t) := \xi_c\left(T_2\left(\frac{3 \cdot w}{10}, w, t\right)\right) \quad \xi_{c3}(w,t) := \xi_c\left(T_2\left(\frac{w}{2}, w, t\right)\right)$$

$$\xi_{c4}(w,t) := \xi_c\left(T_2\left(\frac{7 \cdot w}{10}, w, t\right)\right) \quad \xi_{c5}(w,t) := \xi_c\left(T_2\left(\frac{9 \cdot w}{10}, w, t\right)\right)$$

The average of the concrete strength reductions in 5 zones of the half of a two sided exposed wall w:

$$\xi_{cave}(w,t) := \frac{1 - \frac{0.2}{5}}{5} \cdot \left(\xi_{c1}(w,t) + \xi_{c2}(w,t) + \xi_{c3}(w,t) + \xi_{c4}(w,t) + \xi_{c5}(w,t)\right)$$

The temperature TM(w,t) and the strength reduction xcM(w,t) in the centre line of this wall:

$$T_M(w,t) := T_2(w,w,t) \qquad \xi_{cM}(w,t) := \xi_c\left(T_2(w,w,t)\right)$$

The reduction ab(w,t) and ac(w,t) of a cross section of width w at the time t of a standard fire exposure ís then:

$$a_b(w,t) := w \cdot \left(1 - \frac{\xi_{cave}(w,t)}{\xi_c\left(T_2(w,w,t)\right)}\right) \quad \text{for a beam}$$

$$a_c(w,t) := w \cdot \left[1 - \left(\frac{\xi_{cave}(w,t)}{\xi_c\left(T_2(w,w,t)\right)}\right)^{1.3}\right] \quad \text{for a column}$$

In some textbooks the stress distribution factor h is used in stead of a, and this is:

$$\eta(w,t) := \frac{\xi_{cave}(w,t)}{\xi_c\left(T_2(w,w,t)\right)}$$

SOLID SLAB Time of standard fire $t := 60$ min

Free span $L := 5.63 \cdot m$ Width $b := 1.20 \cdot m$ Height $h := 0.200$ m $w := 2 \cdot h$

Concrete strength $f_{cc20} := 20.0 \cdot MPa$ $f_{ct20} := 3.2 \cdot MPa$

Steel 0.2% strength $f_{s20} := 550 \cdot MPa$ Reinforcement $D := 0.010$ m $n_s := 8$

Cover thickness $c_y := 0.015$ m $c := c_y + \frac{D}{2}$ $c = 0.0200$ m Anchorage $l_a := 0.070 \cdot m$

$cs := c + \frac{D}{2}$ $cs = 0.025$ m

External load $q_b := 2.4 \cdot \frac{kN}{m^2}$ $Q_b := q_b \cdot 1.2 \cdot m \cdot L$ $Q_b = 16.2\,kN$

External load in each 1/4 point for each slab element $F := \frac{Q_b}{2}$ $F = 8.11\,kN$

Dead load per slab $G := b \cdot h \cdot 17.75 \cdot \frac{kN \cdot m}{m^3}$ $G = 4.26\,\frac{kN}{m}$

Shear force $Q(t) := G \cdot \frac{L}{2} + F$ $Q(t) = 20\,kN$

Moment load $M(t) := \frac{1}{8} \cdot G \cdot L^2 + F \cdot 1.425 \cdot m$ $M(t) = 28\,kN \cdot m$

Total reinforcement area $A_s := n_s \cdot \pi \cdot \frac{D^2}{4} \cdot m^2$ $A_s = 628.319 \times 10^{-6}\,m^2$

Temperature of reinforcement $T_s(t) := T_1(c,t)$ $T_s(t) = 465$ C $\xi_s(T_s(t)) = 0.544$

Yield strength of reinforcement $F_s(t) := A_s \cdot \xi_s(T_s(t)) \cdot f_{s20}$ $F_s(t) = 188\,kN$

Temperature at top $T_t(t) := T_2(h, w, t)$ $T_t(t) = 20$ C $\xi_c(T_t(t)) = 1.000$

Depth of compression zone $y(t) := \frac{F_s(t)}{b \cdot \xi_c(T_t(t)) \cdot f_{cc20}}$ $y(t) = 0.00784\,m$

Moment capacity $M_u(t) := F_s(t) \cdot \left(h \cdot m - c \cdot m - \frac{y(t)}{2} \right)$ $M_u(t) = 33.1\,kN \cdot m$

Temperature in the weakest shear layer $T_{cs}(t) := T_1(cs, t)$ $T_{cs}(t) = 396$ C

Shear capacity $F_{shear}(t) := \frac{f_{ct20} \cdot \xi_c(T_{cs}(t)) \cdot b \cdot \left(h \cdot m - c \cdot m - \frac{y(t)}{2} \right)}{2}$ $F_{shear}(t) = 293.9\,kN$

Estimated tensile strength of 15 mm concrete cover on a reinforcing bar

$F_{cts}(t) := \left[0.005 \cdot \left(\xi_c(T_1(0.0025,t)) + \xi_c(T_1(0.0075,t)) \right) + 0.005 \cdot \xi_c(T_1(0.0125,t)) \right] \cdot m \cdot f_{ct20} \cdot l_a$

$F_{cts}(t) = 2.282\,kN$ Ultimate splitting strength $F_{as}(t) := F_{cts}(t) \cdot 2 \cdot \pi$ $F_{as}(t) = 14.3\,kN$

Ultimate bond strength $F_{ab}(t) := \xi_c(T_1(c,t)) \cdot 1.3 \cdot f_{cc20} \cdot \pi \cdot D \cdot m \cdot l_a$ $F_{ab}(t) = 47.1\,kN$

Ultimate anchorage capacity $F_a(t) := n_s \cdot \min(F_{ab}(t), F_{as}(t))$ $F_a(t) = 115\,(kN)$

Ultimate shear capacity $Q_u(t) := \min(F_{shear}(t), F_a(t))$

$Q(t) = 20.1\,kN$ $M(t) = 28.4\,kN\cdot m$ $t := 5, 10 .. 80$

$Q_u(69) = 112\,kN$ $M_u(69) = 28.7\,kN\cdot m$ Calculated fire resistance time 69 **minutes**

$t =$	$Q_u(t) =$ (kN)	$M_u(t) =$ (kN·m)
5	160.5	59.72
10	150.7	58.62
15	142.8	54.95
20	137.1	51.95
25	132.5	49.43
30	128.8	47.28
35	125.7	45.39
40	122.9	43.73
45	120.5	42.10
50	118.4	38.85
55	116.5	35.87
60	114.7	33.13
65	113.1	30.59
70	110.9	28.22
75	108.2	26.00
80	105.6	23.92

(An increase of cover thickness from 15 to 25 mm would give rise to an increase of fire resistance time to 114 minutes.)