PREDICTION OF THE FAILURE TIME OF GFRP-RC BEAMS UNDER FIRE CONDITIONS

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Abstract: A model is proposed to predict the time to failure of reinforced concrete beams in a fire. The model is developed specifically to predict the lifetime of beams reinforced with glass fibre reinforced plastic rebar, GFRP, but is applicable to beams with any form of reinforcement. The model is based on the calculations for flexural capacity and shear capacity of beams embedded within ACI design codes where time and temperature dependent values for rebar modulus and strength and concrete strength replace the static design values. The base equations are modified to remove safety factors and where necessary the temperature induced reductions in strength for concrete and steel are derived using the equations presented by Eurocode 2.

In order to validate the model it was used to predict the failure times of steel rebar reinforced beams that had been documented in the literature. There was excellent agreement between the model and the reported lifetimes for these conventional beams. The model was applied to predict the lifetimes of two beams that had been manufactured and tested to destruction in a fire by the research group. The model predicted that the failure mode of the beams would be via rebar rupture as opposed to the design condition of concrete crushing and this was confirmed by the experimental test results. The model provided reasonable agreement with experimental results with a lifetime of 103 minutes predicted based on flexural failure and 94 and 128 minutes observed in the experiments. The model was also used to predict failure in shear and this was not successful when the shear stirrups were produced from GFRP. This is linked to a low value of GFRP strength used in the code that forms the basis for the model calculations.

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INTRODUCTION

A model has been developed to predict the properties of a concrete beam, reinforced with a glass fibre rebar. The basic philosophy of the model is shown schematically in Figure 1. It relies on a continual comparison of applied service loads and the ability of the beam to carry those loads. The properties of the beam in terms of strength and stiffness are progressively reduced by the increasing temperature which softens and weakens the glass fibre rebar and the concrete matrix. At some point the properties of the beam will fall below those required to sustain the service loadings and the beam will fail. This generic approach to predicting failure is of course not specific to beams reinforced with composite rebar and could be applied to any reinforced concrete situation if the appropriate properties are used for the rebar in the model.

The model uses the design codes for composite rebar to supply the basic equations that determine flexural strength, the ability to sustain a flexural moment and shear strengths. These equations are however modified in order to eliminate the various safety factors that are introduced into the design process as the purpose of a design code is to ensure that failure does not occur rather than to predict the each time of failure.

Reference to the schematic diagram, Figure 1, shows that the first stage in the modelling process is the identification of the service requirements for a beam. This allows the standard codes to be used to design a beam in an appropriate fashion. It is at this time that the design equations are modified to allow a calculation of the actual strengths and stiffness of the beams. If the beam has been designed correctly then these properties will exceed the service requirements by a comfortable margin. If they do not then the design has not been performed correctly and a re-design is needed.

The next stage in the modelling process is to increment the time of the model such that the temperature experienced by the beam is increased. The increase in the ambient temperature changes the properties of the rebar. The thermal properties of the rebar are required in order to undertake a recalculation of the strength and stiffness properties of the beam. The change in properties of composite rebar with temperature has been assessed in a separate series of experiments which were also reported in a previous paper[3]. This data also takes into account the influence of continuous exposure to alkaline environments. In order to relate the ambient temperature change to the temperature of the rebar, a thermal model is required which is appropriate to the beam under evaluation. A simple thermal model, which is generally applicable to all beams with a rectangular cross section, was presented in an earlier paper by the authors[2]. This thermal model can also be used to calculate the reduction in compression strength of the concrete matrix.

FLEXURAL CAPACITY OF REINFORCED CONCRETE BEAMS IN A FIRE TEST

The flexural capacity of a beam is the ultimate bending moment that can be sustained by the beam in flexure before failure occurs. The value of the flexural capacity will be determined by the strength of the concrete, σ^{*}_{cT}, the strength of reinforcing bars, σ^{*}_{fT} and by the reinforcement ratio, ρ_f, which is the volume fraction of reinforcing bars, and beam dimensions b_T, d. Note that the residual width b_T is obtained by ignoring the concrete layer with temperature in excess of $700°C$ in the temperature profile of the beam cross section described in Ref. 2.

The reinforcement ratio, ρ_f, is a design input which is usually selected to ensure that failure in the beam, if it occurred, would do so by concrete crushing in the compression face of the beam, rather than by rebar failure in the tension face. The minimum reinforcing ratio that is acceptable is determined by identifying the reinforcement ratio of a balanced condition, ρ_{fb} where failure would occur simultaneously in the concrete and the rebar [1].

The code provides a general expression for this balanced state which links the failure stress of the reinforcing bars, the failure stress of the concrete, their respective moduli and the reinforcing ratio. An increase in the reinforcement ratio from this balanced condition ensures that failure occurs in the concrete.

If the reinforcement ratio is set such that failure occurs in the concrete, a modified expression can then be developed which calculates the stress acting on the reinforcing bars, $\sigma^{/}_{fT}$, at the point of compression failure in the concrete, Equation 1a.

$$\sigma'_{fT} = \left(\sqrt{\frac{(E_{fT}\varepsilon_{cu})^2}{4} + \frac{\sigma^*_{cT}}{\rho_f} E_{fT}\varepsilon_{cu}} - 0.5E_{fT}\varepsilon_{cu} \right) \tag{1a}$$

If the reinforcing ratio is set too low, then failure occurs in the reinforcing bar. In these circumstances another expression can be derived from the equation describing the balanced condition which links the failure stress of the rebar with the stress acting on the concrete at the instant of rebar failure, σ'_{cT}, Equation 1b.

$$\sigma'_{cT} = \frac{1}{2}\rho_f \sigma^*_{fT} \left(1 + \sqrt{\frac{\rho_f E_{fT} + 4E_{cT}}{\rho_f E_{fT}}} \right) \tag{1b}$$

Where

σ'_{fT} = stress in the FRP reinforcement when concrete fails at temperature T°C,

σ'_{cT} = stress at concrete when rebar fails at temperature T°C,

ε_{cu} = concrete strain to failure 0.35% taken from code[1],

ρ_f = ratio of GFRP reinforcement = $\frac{A_f}{bd}$,

σ^*_{fT} = rebar strength at failure at temperature T°C,

E_{fT} = modulus of elasticity of GFRP at temperature T°C,

E_{cT} = modulus of elasticity of concrete at temperature T°C,

σ^*_{cT} = concrete strength at failure at temperature T°C.

In the fire tests the rise in temperature experienced by the beam will result in simultaneous changes in the strength and stiffness of the rebar and the concrete resulting in a change in flexural capacity of the beam. Moreover, the relative changes in the properties of each constituent will vary such that it is conceivable that the failure mode of the beam may change from that initiated by concrete crushing at the beginning of the test to that initiated by failure of the rebar at some time into the fire exposure. Accordingly, the two expressions for the stress on one constituent (rebar or concrete) when the other constituent fails are required if the flexural capacity is to be determined under each of these possible failure conditions. The relevant equations are given below, Equations 2a where rebar failure occurs first and 2b where concrete failure occurs first.

$$M_{nT} = \rho_f \sigma^*_{fT} \left(1 - \frac{\rho_f \sigma^*_{fT}}{\sigma'_{cT}} \right) b_T d^2 \qquad (2a)$$

$$M_{nT} = \rho_f \sigma'_{fT} \left(1 - \frac{\rho_f \sigma'_{fT}}{\sigma^*_{cT}} \right) b_T d^2 \qquad (2b)$$

This set of equations provides the basis for the prediction of failure for the beams. Relevant data are required for the changes in properties of the rebar and concrete with time that allow the change in flexural capacity to be calculated and compared to the applied flexural moment. When the flexural moment exceeds the flexural capacity the beam is deemed to have failed. It should be noted that the data required is presented as changing with time rather than temperature. This requires a relationship to have been predetermined between the time duration of the fire test and the local temperature throughout the beam and a further link between that local temperature and the material property.

In order to validate this model before it is applied to the unknown situation of failure of a FRP rebar reinforced concrete beam, it will be used to predict the failure of steel rebar reinforced beams. The failure of such beams has been well documented and data is available in the literature that allows the predicted change in flexural capacity to be calculated and compared to an applied moment of service.

VALIDATION OF MODEL FOR PREDICTION OF FAILURE TIME ON STEEL REINFORCED CONCRETE BEAMS

In this section the proposed model is validated using the results of fire tests performed by Lin et, al. [4] on two steel reinforced concrete beams. The dimensions of the test specimens were 533mm in height x 229mm wide. The beams measured approximately 8.2m in length with 6.1m span exposed to fire. The reinforcements ratio in both beams was 0.032 while the concrete cover was either 38mm or 58mm. The beams were fabricated with normal-weight concrete with compressive strengths of 28 MPa and 30 MPa respectively. The beams were exposed to a standard fire test with an applied moment on the beam of 117kN.m. Tests were terminated when the rate of increase in deflection reached the point where flexural failure became imminent.

In order to calculate the flexural capacity of these beams, both equations 2a and 2b were used to separately predict the beam strength when either concrete crushing or steel rebar rupture triggered failure. Data for the reduction in strength and stiffness of the rebar as a function of temperature was obtained using the relationships given in ENV EC2 part1.2[5].

i.e.:

$$\frac{\sigma_{yT}}{\sigma_{yv}} = k_s \tag{3}$$

Where σ_{yT} is yield strength of steel at T °C (MPa)

$$k_\sigma = 1.0 \qquad \text{for } T \le 350 \tag{4}$$

$$k_\sigma = (1.899 - 0.00257T) \qquad \text{for } 350 \le T \le 700 \tag{5}$$

$$k_\sigma = (0.24 - 0.0002T) \qquad \text{for } 700 \le T \le 1200 \tag{6}$$

$$k_\sigma = 0 \qquad \text{for } 1200 \le T \tag{7}$$

σ_{yv} is assumed to be 414 MPa.

The elastic modulus of steel reinforcement, E_s, is 200GPa at room temperature. The reduction factors k_E for steel rebar as a function of temperature as proposed by ENV EC2 Part 1.2[5] are given as follows:

$$k_E = \frac{E_{fT}}{E_{f\,20°C}} \tag{8}$$

$$k_E = 1 \quad \text{for } 0 \le T \le 100 \tag{9}$$

$$k_E = 1.10 - 0.001T \quad \text{for } 100 \le T \le 500 \tag{10}$$

$$k_E = 2.05 - 0.0029T \quad \text{for } 500 \le T \le 600 \tag{11}$$

$$k_E = 1.39 - 0.0018T \quad \text{for } 600 \le T \le 700 \tag{12}$$

$$k_E = 0.41 - 0.0004T \quad \text{for } 700 \le T \le 800 \tag{13}$$

$$k_E = 0.27 - 0.000225T \quad \text{for } 800 \le T \le 1200 \tag{14}$$

$$k_E = 0 \quad \text{for } 1200 \le T \tag{15}$$

Lin et al. [4] report the temperature profiles experienced in their test beams so it was possible to accurately assign values of rebar strength and stiffness at any time during the fire test using the ENV EC2 relationships and hence to calculate the flexural capacities of the two beams.

The results of this exercise are plotted in Figure 2. Two curves are plotted for each beam. One calculates the flexural capacity assuming concrete crushing failure, and the other assumes failure via rebar rupture. For both beams, the flexural capacities calculated for rebar failure are higher than those for concrete crushing for the entire duration of the test.

The applied moment was 117 kN.m and a line was drawn from the flexural capacity axis to meet the lowest property curve. The point of intersection identifies the time at which the flexural capacity is reduced to the same value as the applied moment and hence defines the failure time. This is 220 minutes for the beam with a 38 mm cover and 225 minutes for a beam with a 58 mm cover.

The empirical failure times reported by Lin et al. [4] were identified as the point whereby the rate of beam deflection increased dramatically and were reported as 220 minutes and 243 minutes respectively.

This compares remarkably well with the predictions of our model, particularly for the 38 mm cover and suggests that the model is physically sound when the rebar is steel.

APPLICATION OF MODEL TO THE FAILURE OF FRP-RC BEAMS

The proposed model is now applied to the case of two reinforced concrete beams where the rebar is a glass fibre composite system. The experimental results of these tests[6] and experimental programmes that generated temperature dependent data for the rebar and a thermal model for the beam have been reported in earlier paper[2].

The beams were rectangular in cross section with dimensions 350mm wide, 400 mm deep with a span of 4250 mm. They were designed to meet a service requirement of a sustained load of 40kN. The beams were reinforced with an FRP rebar, diameter, ϕ, of 12.7mm and with a reinforcement ratio, ρ_f, of 0.0089, which was designed to ensure failure by concrete crushing under destructive testing at normal temperatures.

The two beams were manufactured to this same specification but one beam was equipped with composite stirrups to carry the shear load and the other with steel stirrups.

Full details of beam construction, mechanical testing and fire tests are given elsewhere [6] see Figures 3a, 3b, 3c for the beam section, reinforcement arrangement, loading and experimental set up. Figure 4 and Figure 5 shows failure after fire in beam 1 and beam 2 respectively. The various data inputs used in the model calculations are listed in Table 1 along with the values for flexural capacity based on either concrete crushing or FRP rebar rupture.

The results of the model predictions for flexural capacity changes in the FRP-RC beams are plotted in Figure 6. The failure mechanism for the FRP-RC beams will change with time. Equation 2a results in the lowest values for flexural capacity in entire duration of test. It is apparent that when the design values for rebar modulus and strength are replaced with modified values that account for the effects of alkaline exposure and subsequently for the effects of temperature, that the predicted failure mode changes from concrete crushing to rebar facture.

It should be stressed that these calculations require data on the thermal profiles thorough the beams at different times during the test. The thermal model [2] that was used for this calculation was not accurate over the first 30 minutes of the fire test and hence the values calculated for flexural capacity in this time interval, by assuming either failure mode, are inaccurate and should be ignored.

In order to predict the failure time of the beams using the model it is necessary to calculate the service moment due to the beam self load and the sustained load. In this case the sustained load was 40 kN and the beam dimensions were in cross section (350 x 400) and 4250mm of the span length. The moment at service is calculated as follow

Self-weight of the beam =(width) x (height) x (beam span) x (concrete density)

=(0.350) x (0.400) x (4.25) x (23.5kN/m^3)

= 14/4.25 = 3.3kN/m

Sustained load = 40/4.25 = 9.4 kN/m

Total load = 9.4 + 3.3 =12.7 kN/m

Moment at service $M_s = \dfrac{(12.7)(4.25)^2}{8} = 28.7 kN.m$

This value is depicted in Figure 6.

The predicted failure time for the two beams is the time at which the flexural capacity falls below the applied flexural moment. The predicted flexural capacity of the beams falls below the imposed moment of service after 103 minutes.

When the real beams were subjected to the fire test, the fire rating or load bearing capacity based on BS 476 part 21[7] was 128 minutes for beam 1 and 94 minutes for beam 2. Failure in both cases was identified as the onset of a rapid increase in deflection causing the removal of the sustained load.

The failure prediction for the FRP reinforced beams is reasonable and conservative although not quite as accurate as the predictions for the steel reinforcements. A possible cause for the apparent delay in failure of beam 1 could be the poor thermal conductivity of the composite rebar itself. When the thermal model predicts that the rebar interface temperature reaches a certain value, it is probable that the temperature throughout the rebar itself is non-uniform. This in turn could mean that the value for residual rebar strength used in the calculations is slightly lower than the real strength.

However the model itself has relied on a number of assumptions and approximations built into the data. The thermal model used, while adequate, does not give a precise representation of the temperature changes in the beam. Furthermore, the data used to predict rebar strength varied according to the presumed extent of exposure to alkali [3]. Hence the prediction for beam 1, which was accurate to 20%, must be considered to be a good result.

The prediction for beam 2, of 94 minutes in the actual test compared to the predicted 103 minutes seems even better. However in this case the rebar used in construction differed from the rebar used to generate test data. In the case of beam 2 the rebar was a glass fibre composite with a high temperature thermoplastic matrix. This rebar would not have suffered such a pronounced reduction in strength due to alkali attack but would probably have seen a greater reduction in strength and stiffness at the higher temperatures. The main purpose of using beam 2 was to examine the importance of shear connections in the beam which is discussed later in the paper.

THE RELATIVE PERFORMANCE OF FRP AND STEEL REINFORCEMENT IN CONCRETE BEAMS DURING A FIRE TEST

While the development of a model to predict the fire resistance of a FRP-RC beam was the prime objective of this work, a secondary objective was to assess the relative performance of the composite and steel rebar reinforcements during a fire.

As the model used in this work has been validated as accurate for steel reinforced beams, it was used to predict the fire resistance of two model beams that are considered equivalent to the FRP-RC beams tested. The first model beam, steel 1, replicated the FRP-RC beam exactly. It has the same dimensions and reinforcing ratio, but the FRP rebar is directly replaced with steel rebar. This resulted in an initial flexural capacity of 242 kN.m. The second model beam, steel 2, is different. In this case the beam has been designed to satisfy the same design criterion as the FRP-RC beam. It has the same span but the reinforcing ratio and depth have been adjusted to provide the same initial flexural capacity, 104 kN.m, as the FRP-RC beam 1. This second model beam was reinforced with 5 x ϕ10 steel rebars, the section was designed as 200 x 326 mm with a concrete cover of 38mm. The concrete compressive strength for these beams was 28MPa.

The change in flexural capacity of these beams is calculated for a fire test using the same thermal model [2] used to predict the temperature distribution in the FRP-RC beam.

The change in flexural capacities with time for both steel 1 and steel 2 are compared in Figures 7 to the changes in beam 1. It can be seen that both steel reinforced beams outperform the composite reinforced beam. However the difference in predicted failure times for the same applied bending moment (28.7 kN.m) is less if the composite reinforcement is compared to the beam designed to exhibit the same initial flexural capacity. This beam, steel 2, would fail after 255 minutes compared to the 103 minutes predicted for the FRP-RC beam and the 128 minutes recorded in practice. The direct geometric equivalent beam with steel reinforcement, steel 1, would fail after 290 minutes.

Shear capacity

While the observed mode of failure for the reinforced concrete beams loaded under fire conditions in this programme was in flexure, it is also possible that beams may fail in a shear mode. Stresses due to the thermal strain frequently cause the beams to crack or spall and lead to a reduction in concrete available to resist shear force. This section accordingly considers the shear capacity (shear load before failure) of the beams during the fire test. The calculations for flexural capacity of beam 1 and beam 2 were identical, but differ for shear capacity due to the different stirrup materials used in each case. Beam 1 was equipped with composite stirrups and beam 2 with steel stirrups

The total shear capacity of a reinforced concrete beam, τ_{nT}, is provided by two elements, the shear resistance of the concrete, $\tau_{c,f}$, and that of the stirrups, τ_f .

$$\tau_{nT} = \tau_{c,f} + \tau_f \qquad (16)$$

The ACI code [1] includes a route for calculating the composite stirrup contribution to the shear capacity of a reinforced beam which was modified earlier to include time and temperature dependent variables for composite stirrups.

For beam 1, the nominal shear capacity , τ_{nT}, at temperature T°C is accordingly calculated using the expression below;

$$\tau_{nT} = \frac{\rho_f E_{fT}}{90\sigma^*_{cT}} \frac{\sqrt{\sigma^*_{cT}}}{6} b_T d + \frac{A_{f,v}\sigma_{f,vT} d}{s} \tag{17}$$

Where

A_{fv} =Area of GFRP shear reinforcement within spacing s,

b_T = width of the GFRP RC beam at temperature,

d = distance from extreme compression zone to centre of the tension reinforcement,

E_{fT} = modulus of elasticity of GFRP at temperature T°C,

σ^*_{cT} =concrete strength at failure at temperature T°C,

$\sigma_{f,vT}$ = FRP tensile strength for shear design at T°C = 0.002 E_{fT}

s = stirrups spacing.

A similar expression can be used to determine the shear capacity of beam 2. However the second part of the expression, for τ_f, is replaced by a term, τ_s, taken from the Eurocode equations[5] to calculate the reduction in strength of the steel stirrups used in this beam.

For steel stirrups in beam 2 the shear resistance based on ENV EC2 Part 1.2[5] is as follows

$$\tau_s = \frac{A_{sw}\sigma_{yv} d}{1000s} \tag{18}$$

where A_{sw} = area of cross section of stirrups, σ_{yv} = yield stress for steel reinforcement, s = 160mm stirrup spacing

The same thermal models and experimental property data are used to calculate the reduction in composite properties as those used to determine flexural capacity. The detailed calculations for the beams flexural and shear capacities over time exposure are presented in Table 1. Note that these calculations are based on a concrete compressive strength $\sigma^/_c$ = 28 MPa and elastic modulus of rebar

E_f = 31000 MPa, at normal temperature, which is itself selected from the tested rebar after 30 days exposure in 60°C in alkaline conditions[3]. Concrete cover on the tensile reinforcement was 70mm.

The total and component shear capacity values with time are predicted to change as shown in Figure 8 for beam 1 and Figure 9 for beam 2.

The shear load ,V_u , acting on the beam during the fire test can be calculated as follow

Shear load $V_u = S_d \dfrac{L}{2}$ (19)

Where S_d = applied load 12.7kN/m and L = span length 4.25m

In the case of the experimental test on the two beams, a shear load of 27kN was imposed and this value is indicated on both Figures 8 and 9. The results suggest that beam 2 is fully protected against shear failure by the presence of the steel stirrups, but that beam 1 should have failed in shear after only 50 minutes. As this did not happen, there is some error in the method of calculating the shear capacity of the beam.

The term in the equation used by the code that determines the contribution of the composite stirrups contains the value, $\sigma_{f,vT}$, which is the product of the tensile modulus of the rebar multiplied by 0.002. This suggests that the code has taken a failure strain of 0.2% for the rebar under these conditions. This is a very small value for the tensile strength of a unidirectional glass fibre composite, and is the probable source of error. It must be remembered that the purpose of the design code is to ensure that the beam does not fail in service, and the introduction of a low value for the failure strain would force a designer to over reinforce the beam to compensate. This is good practice to ensure safety when material property data is unreliable and where the experience of service performance is limited. However it is likely that this value could and should be raised to more realistic values, say 0.0035, to allow composite reinforcements to be competitive without compromising safety.

The relative shear capacities of the two beams are shown in Figure 10. It is clear that steel stirrups provide much greater shear resistance for a given stirrup spacing, which would be true even if more realistic values for rebar strain were used in the calculations. It is probable that more composite stirrups would need to be used in a given design to ensure comparable behaviour.

Conclusions

A model has been developed to predict the time to failure for a concrete beam reinforced with a glass fibre composite rebar, during a fire test.

The basic approach of the model has been validated by applying the method to predict the failure times of steel reinforced concrete beams which were fully documented in the literature. The agreement between the model and the published data is very good.

The model has also predicted the failure time of beams that were produced with composite rebar and tested experimentally in a fire by the authors. A prediction of the time to failure within 20% of the experimental figures was achieved. The likely source of error in the model was the input thermal model and the choice of reduced strength values for both composite rebars (which vary with time of exposure to alkaline environments) and concrete (which was calculated using an approximate method).

The predictions assumed failure to take place in flexure. It was demonstrated that, although the initial design had ensured flexural failure would occur via concrete crushing, the likely failure mode in a fire is one of rebar rupture if the reinforcement is FRP.

Additional calculations were undertaken to predict the likely time to failure if shear was the main failure mode. The results from this exercise indicated that the assumptions incorporated in the model, based on the ACI code were in error. The value of assumed failure strain for composite shear stirrups looks too low to provide an accurate prediction of lifetime.

The results of the modelling have also allowed a direct comparison of beams reinforced with either steel or composite rebar. This has shown that the lifetime of an equivalent beam with steel reinforcement is at least double that of a beam with composite reinforcement. It is however important to compare beams that have been designed to undertake the same duties rather than to compare geometrically identical beams with different reinforcements.

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APPENDIX I. REFERENCES

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APPENDIX II. NOTATION

The following symbols are used in this paper:

A_{fv} = Amount of GFRP shear reinforcement within spacing s, mm^2.

A_{sw} = Area of cross-section of steel stirrups, mm^2.

b = width of rectangular cross-section, mm.

b_T = width of concrete cross- section at temperature T°C, mm

d = distance from extreme compression zone to centre of the tension reinforcement, mm.

E_{cT} = modulus of elasticity of concrete at temperature T°C, GPa.

E_{fT} = modulus of elasticity of GFRP at temperature T°C, GPa.

ε_{cu} = concrete strain to failure 0.35% taken from code[1].

$\sigma^{/}_{cT}$ = stress at concrete when rebar fails at temperature T°C, MPa.

$\sigma^{/}_{fT}$ = stress in the FRP reinforcement when concrete fails at temperature T°C, MPa.

σ^{*}_{fT} = rebar strength at failure at temperature T°C, MPa.

σ^{*}_{cT} = concrete strength at failure at temperature T°C, MPa.

$\sigma_{f,vT}$ = FRP tensile strength for shear design at T°C = 0.002 E_{fT} [1], MPa.

σ_{yd} = yield strength of steel stirrups, MPa.

h = overall height of flexural member, mm.

M_{nT} = nominal flexural capacity of the beam at temperature T°C, kN-m.

t = time exposure to temperature, minutes.

τ_c = nominal shear strength provided by concrete with steel flexural reinforcement, kN.

$\tau_{c,f}$ = nominal shear strength provided by concrete with GFRP flexural reinforcement, kN.

τ_f = shear resistance provided by GFRP stirrups, kN.

τ_{nT} = nominal shear strength at section at temperature T°C.

ρ_{fb} = GFRP reinforcing ratio producing balanced strain conditions, MPa.

ρ_f = ratio of GFRP reinforcement = $\dfrac{A_f}{bd}$

S_d = applied load, kN.

s = stirrups spacing, mm.

V_u = shear load, kN.

Table 1 Calculated of the residual flexural and shear capacity due to exposure time in beam 1 and beam 2 using model

a) Residual flexural capacity

Time (min)	σ^{*}_{cT} (MPa) (k_c = 1- 0.0031t Eq. 16 of Ref. 2)	T rebar ºC (Eq. 3 of Ref. 2)	σ^{*}_{fT} (MPa) (k_{σ} = 1-0.0073t for 70mm cover Eq. 18 of Ref.2)	E_{fT} (MPa) ($k_E = 1 - 0.0046t$ for 70mm cover Eq.21 of Ref. 2)	$\sigma^{/}_{fT}$ (MPa) (Eq. 1a)	$\sigma^{/}_{cT}$ (MPa) (Eq. 1b)	M_{nT} (kN-m) (Eq. 2a)	M_{nT} (kN-m) (Eq. 2b)
0	28	20	349	31000	533	33	104	145
30	25.4	101	273	26722	472	28	82	130
60	22.8	231	196	22444	411	22	59	114
90	20.2	316	120	18166	349	15	37	97
120	17.6	383	43	13888	287	6	13	81
135	16.3	413	5	11749	254	0.8	1.6	72
143	15.6	440	-	10608	237	-	-	67

b) Residual shear capacity

Time t (min)	$\tau_{c,f}$ beam 1 and beam 2 (kN)	T stirrups ºC (Eq. 3 of Ref. 2)	τ_f beam 1 (kN)	τ_{nT} beam 1 (kN)	τ_s beam 2 (kN)	τ_{nT} beam 2 (kN) Eq. 16
0	11	20	21.2	32.2	132	143
30	9.9	103	18.3	28.2	132	142
60	8.8	233	15.4	24.3	132	141
90	7.6	320	12.4	20.0	132	140
120	6.2	387	9.5	15.7	117	123
135	5.5	417	8.0	13.5	107	113
143	5.1	445	7.3	12.4	98	103
165	3.8	470	5.2	9.0	89	93
180	2.8	493	3.7	6.5	80	83

Note: The rebar temperature θ for any concrete cover c and exposure time t can be calculated using below equation, which was given in Ref. 2.

$$\theta = (345.\log(8t+1)+20) - 767.\exp^{-(0.001.\exp^{\frac{7.602}{c-23.623}}).t}$$

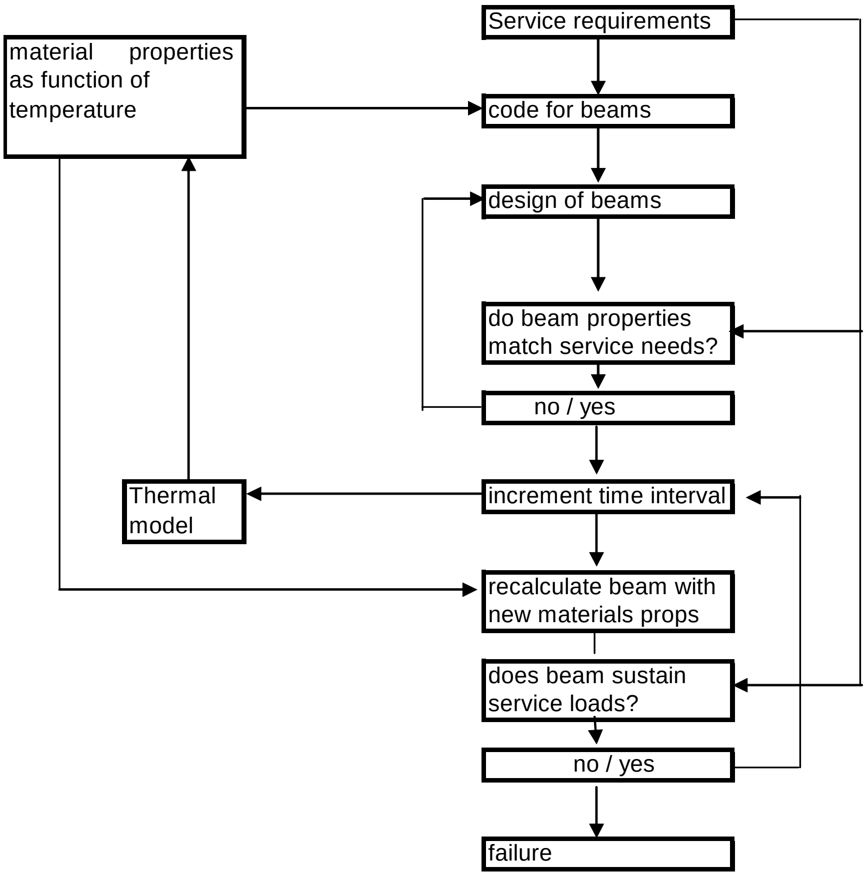

Fig 1 Outline/diagram of the model for prediction the failure time

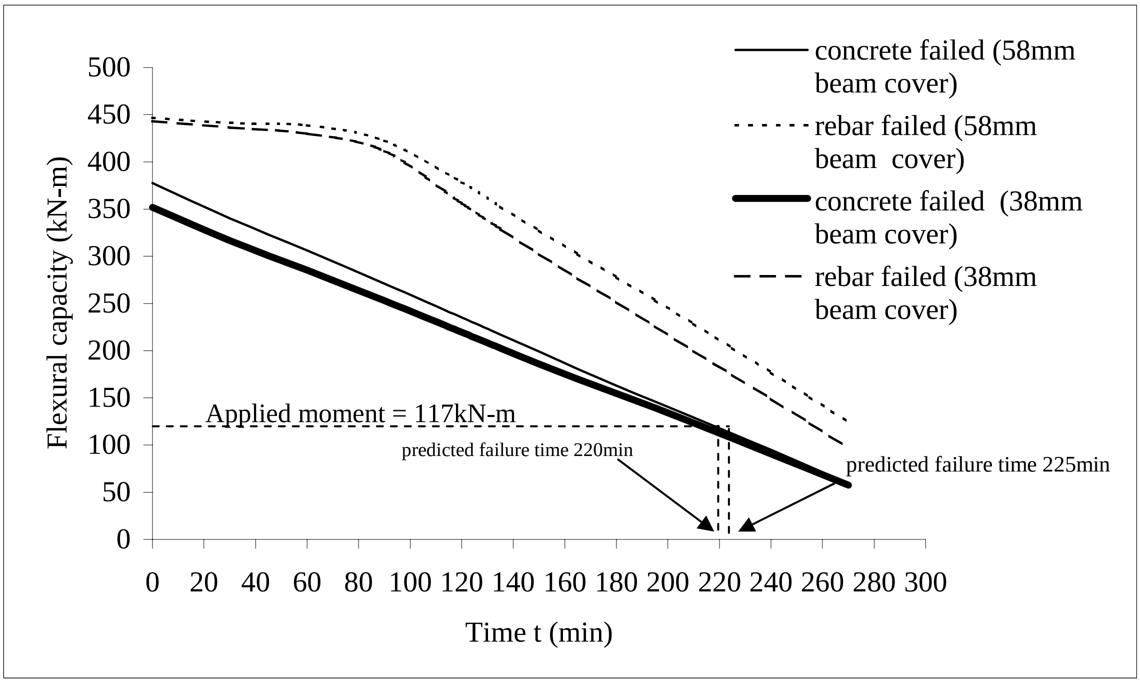

Fig 2 The change in flexural capacity with time in a fire test as predicted by Eq. 2a for concrete failure and Eq. 2b for rebar failure

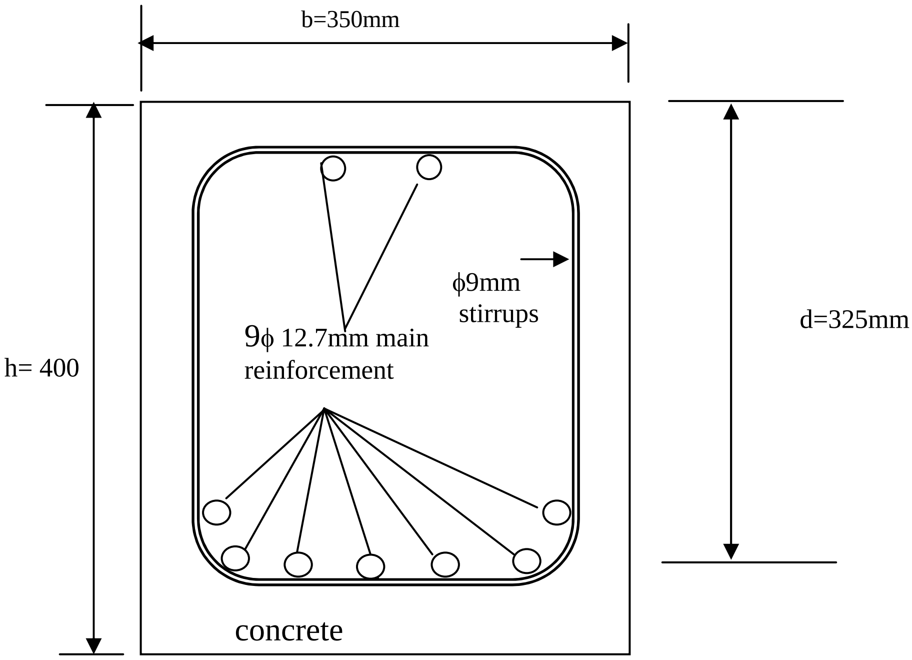

Fig 3a Beam cross section with reinforcement arrangement

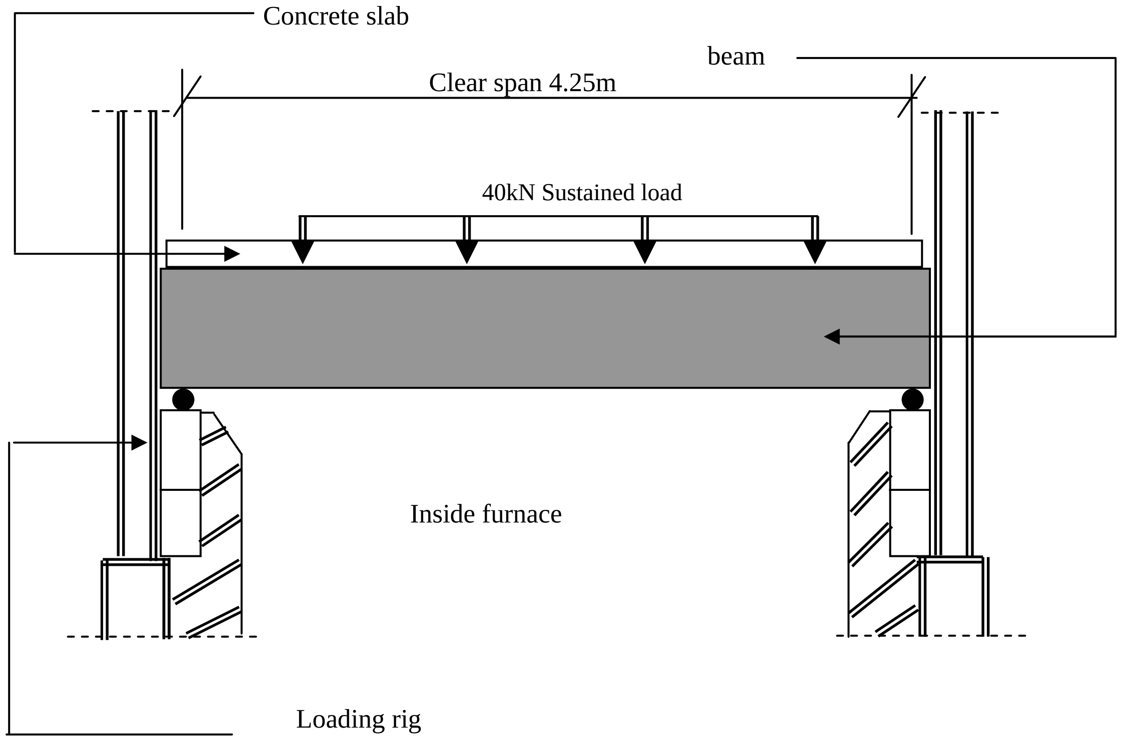

Fig 3b Fire test set up

Fig 3c Beam is set inside the insulated furnace for fire test

Fig 4 Rupture at the mid centre of the concrete in beam 1 after fire test

Fig 5 Failure of beam 2 after fire test inside the furnace shows some flexural cracks and rupture in the main reinforcement

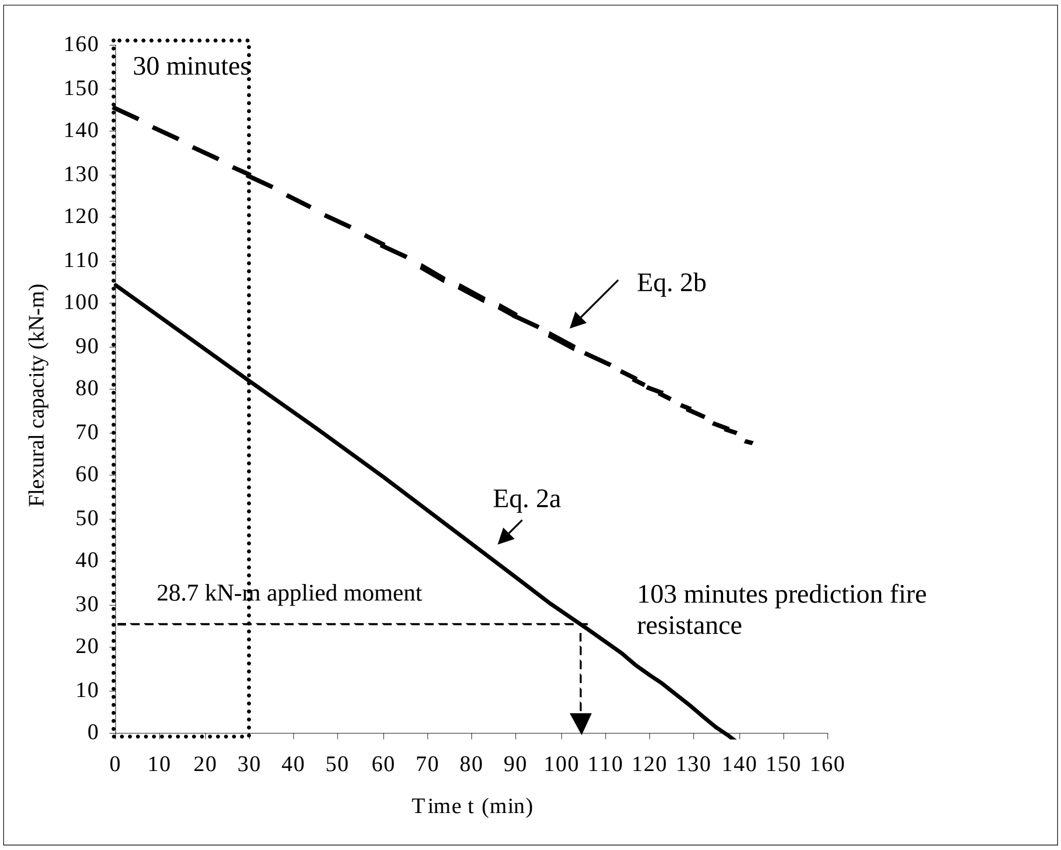

Fig 6 Flexural capacity/time curve of the beam using Eq. 2a and Eq. 2b

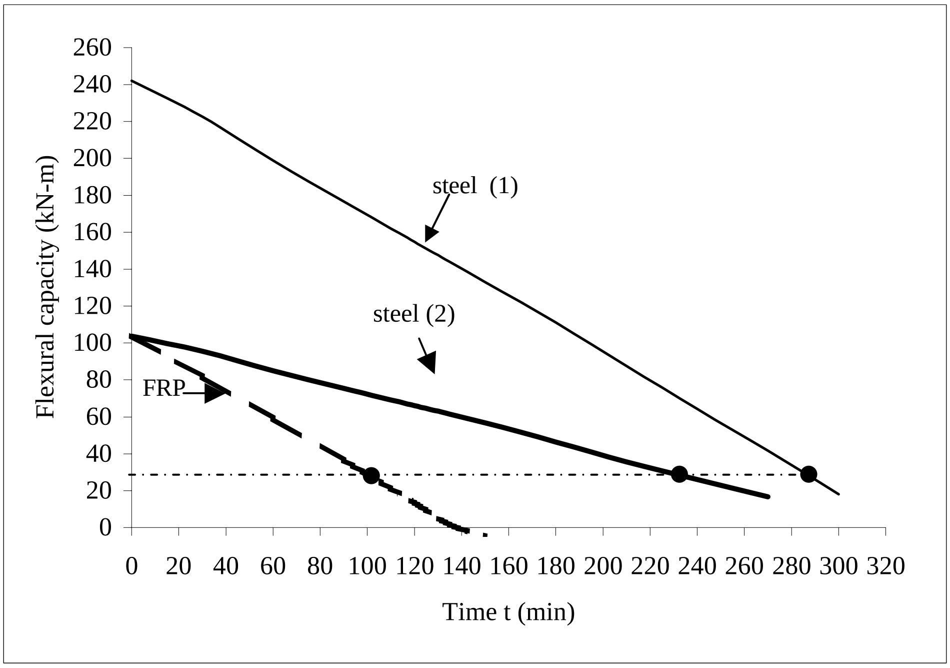

Fig 7 Flexural capacity and predicted failure time for a beam reinforced with steel and its comparison with FRP reinforcement

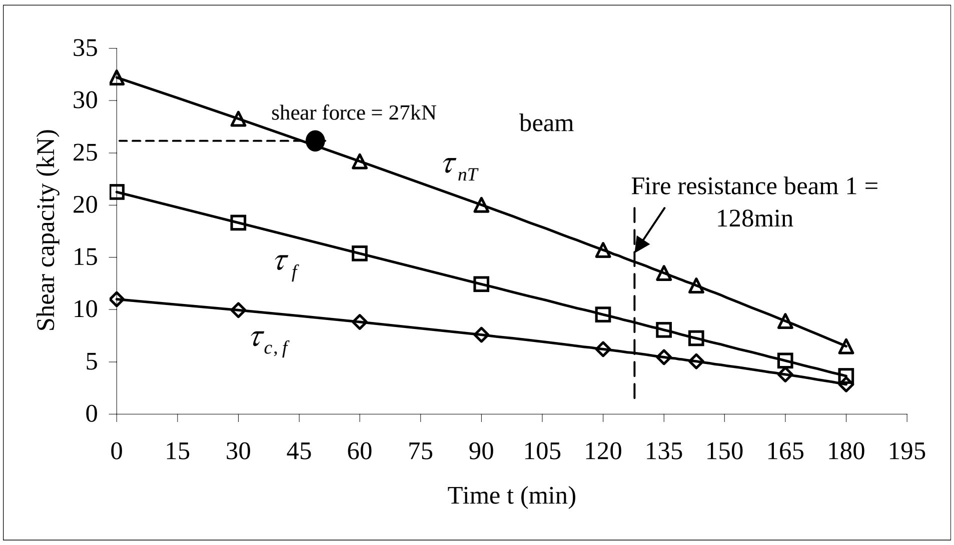

Fig 8 Effect of fire exposure time on the shear strength of concrete and GFRP reinforcement of beam 1 using model

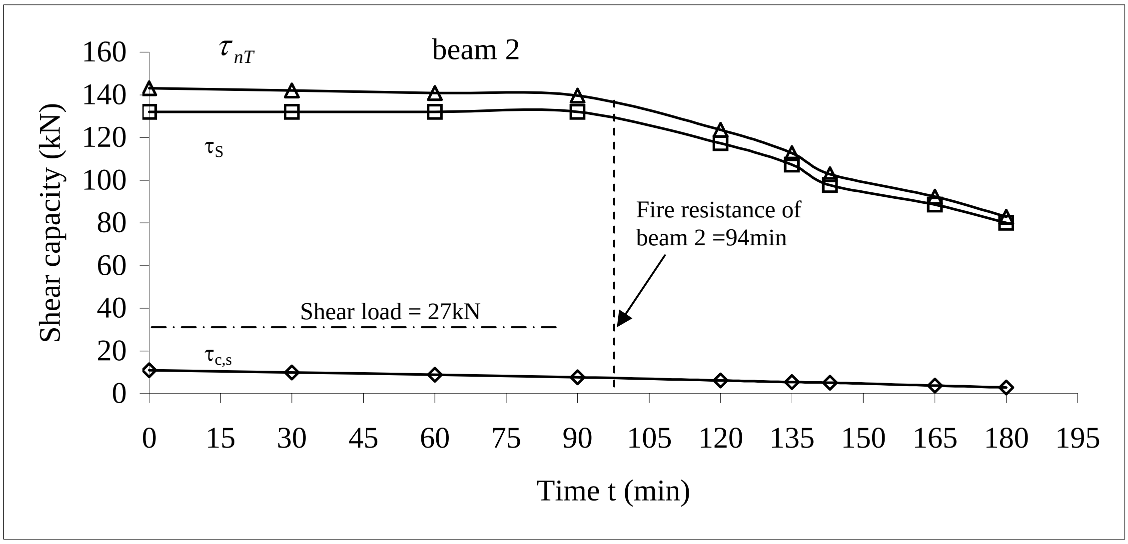

Fig 9 Effect of fire exposure time on the shear strength of the concrete and GFRP reinforcement of beam 2 predicted by model.

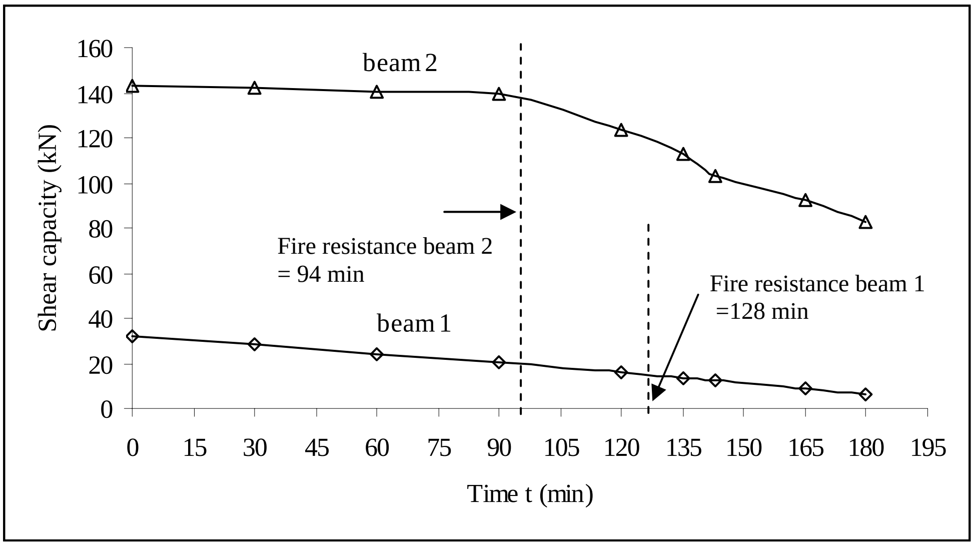

Fig 10 The predicted effect of fire exposure time on the total shear capacity of beam 1 in comparison with beam 2