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DESIGN GUIDELINES FOR FIRE RESISTANCE OF FRP-STRENGTHENED CONCRETE STRUCTURES

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1 INTRODUCTION

The performance in fire of FRP strengthened concrete structures is a key issue that needs to be addressed for the widespread application of FRP in buildings. This paper discusses approaches to developing design guidelines for such structures. To set the context of fire safe design of structures, this paper presents a general overview of North American and European approaches to the design of structures to address their performance in fire. The implication of these design approaches for FRP strengthened concrete structures is discussed. The paper then presents a summary of existing research on the performance of FRP strengthened concrete structures in fire. Based on the available work, recommendations for developing design guidelines for FRP strengthened concrete structures in fire are discussed.

2 OVERVIEW OF FIRE DESIGN PROCEDURES

The safety of buildings in fire depends on the appropriate combination of three basic inter-related elements: fire prevention, suppression, and extinction; evacuation of occupants; and structural integrity. The primary emphasis of fire safety is to reduce the incidence of fire in the first place, but building codes recognize that some fires will always occur. Thus, the next level of protection relates to suppressing or extinguishing any fire that might occur. This can be accomplished to some degree by limiting the availability of fuel or oxygen that the fire needs to grow, or by providing active systems such as sprinklers to suppress or extinguish the fire. One approach to limiting the available fuel for fires is to restrict the materials that may be used for certain types of construction. For example, combustible materials such as wood are not allowed in many building codes for certain classes of buildings. A related approach is to confine the fire as much as possible to a limited area or compartment of the building. Thus, building elements such as walls, floors, and doors should be able to resist fire sufficiently to prevent spread of the fire from one compartment to another. Separating compartments also reduces the availability of oxygen to the fire thus further reducing its ability to spread. Most of the fire compartmentation procedures are passive, but active measures such as sprinkler systems are widely employed. Unfortunately, such systems may not operate properly during a fire since they depend on the effectiveness of maintenance procedures and the availability of water under sufficient pressure. For example, sprinkler systems in the World Trade Center were not effective because the water system that supplied them was compromised. Although this is an extreme case, it does emphasize the limitations of sprinkler systems.

Another essential component to fire safety is the safe evacuation of the building occupants. To ensure this, buildings must have adequate avenues for escape and people must not be exposed to substantial smoke or heat while attempting to escape. Once again, compartmentation of the fire is central to safe evacuation because this helps to limit the spread of smoke and flame in the building. Further, stairwells and other exit routes should be isolated from other parts of the building to keep them relatively free of smoke from fires in other parts of the building. Limits are also placed upon building materials to minimize the impact of potentially toxic smoke. For example, building materials need to meet the requirements of ASTM E84 [1] in North America, and in Europe by ISO 5658/5660 and CEN/TC127. In addition to limiting smoke generation and flame spread, structural integrity of the building is essential during the evacuation phase.

Once building occupants have been successfully evacuated, the final element of fire safety is preventing collapse of the building while fire-fighting activities are on-going. Thus structural fire resistance is the last defence of structures in fire. Structural fire resistance is the main emphasis of this paper and will be discussed in more detail.

2.1 Structural fire resistance

Khoury [2] has defined fire resistance as "the ability of an element (not a material) of building construction to fulfil its designed function for a period of time in the event of a fire." In fires, building elements serve one of two basic functions. Some elements, such as non-load-bearing walls, simply need to separate the fire from adjoining compartments or rooms. Beams and columns, on the other hand, need to have sufficient strength during fire to resist the expected loads during the fire. Some other elements (especially floor slabs and load-bearing walls) need to satisfy requirements for both fire separation and strength. Since this paper is largely concerned with structural effects, the main elements concerned are beams, slabs, and columns. Thus, structural integrity or strength requirements typically govern although slabs also need to satisfy fire separation requirements. The required duration of the fire resistance depends on the type of buildings and the structural element. Typically, beams and slabs require 1 to 2 hour ratings while columns need longer fire endurance ratings (up to 4 hours). Another important point to note is that fire resistance depends on the strength of the *element* under consideration and not the specific materials. Thus, members with FRP may perform satisfactorily under fire scenarios even if the FRP is compromised in the fire.

North American and European requirements for fire resistance are similar in terms of basic goals but differ in terms of approaches. In general, North American requirements are more prescriptive whereas European regulations allow more opportunities for performance-based design. Khoury [2] describes the process well and defines three basic approaches: fire testing, prescriptive methods, and performance-based methods.

2.1.1 Fire testing

The first two approaches are fairly consistent between North American and European practices. Typically, prescriptive methods are described from experience with substantial fire testing on similar structural elements. Fire tests are conducted by exposing full-scale structural elements (beams, columns, walls, or slabs) to a standard fire while under load as described in ASTM E119 [3]. The load applied to the structural element is the load that is expected to be applied during a fire situation. Thus, the loading for a fire test is typically much closer to the service load level than to the ultimate load. In North America, the test load has commonly been taken as the full service load level [3]:

$$\text{Fire test load} = D + L \tag{1}$$

where D is the service dead load and L is the service live load. In European practice, the test load is typically taken as [4]:

$$\text{Fire test load} = 1.35D + (0.4 \text{ to } 0.6)\, L \tag{2}$$

More recently, ASCE [5] has recommended that the load during a fire scenario be taken as

$$\text{Fire test load} = 1.2D + 0.5L + A_k \tag{3}$$

where A_k is an additional load that may be specified by the local building authority. Such a change in loading in American practice is thus more consistent with European practice.

Another important variable in fire testing is the severity of the standard fire. For building fires, the standard fires in North American (ASTM E119 [3]) and European (ISO 834) practice are very similar as shown in Figure 1. These time-temperatures curves represent a severe fire that would be expected in a typical building environment, but do not represent all potential fire scenarios. For example, hydrocarbon fires are more severe; tunnels and other infrastructure where such fires are likely to occur should be designed for these types of fires. Also, real fires typically follow the pattern shown in Figure 1 with three distinct phases after ignition: growth, burning, and decay. The temperatures and the rate of change of temperatures depend upon the nature of the burning surfaces in the growth stage, the amount of oxygen (ventilation) available in the burning stage, and the total amount of fuel in the decay stage. The flashover point divides the growth stage from the burning stage. The ASTM and ISO standard fires do not have a decay stage and thus may be more severe than necessary for situations where the available fuel is limited.

2.1.2 Prescriptive methods

As mentioned earlier, prescriptive methods for fire design are derived from experience with fire testing. For reinforced concrete, prescriptive designs simply provide for minimum dimensions and thicknesses of concrete cover to satisfy given fire endurance ratings for specific classes of structures. For most conventional structures, this approach generally provides for safe designs but may not always be economical [2]. Furthermore, the approach is impossible to apply to new materials such as FRP because insufficient research has been conducted.

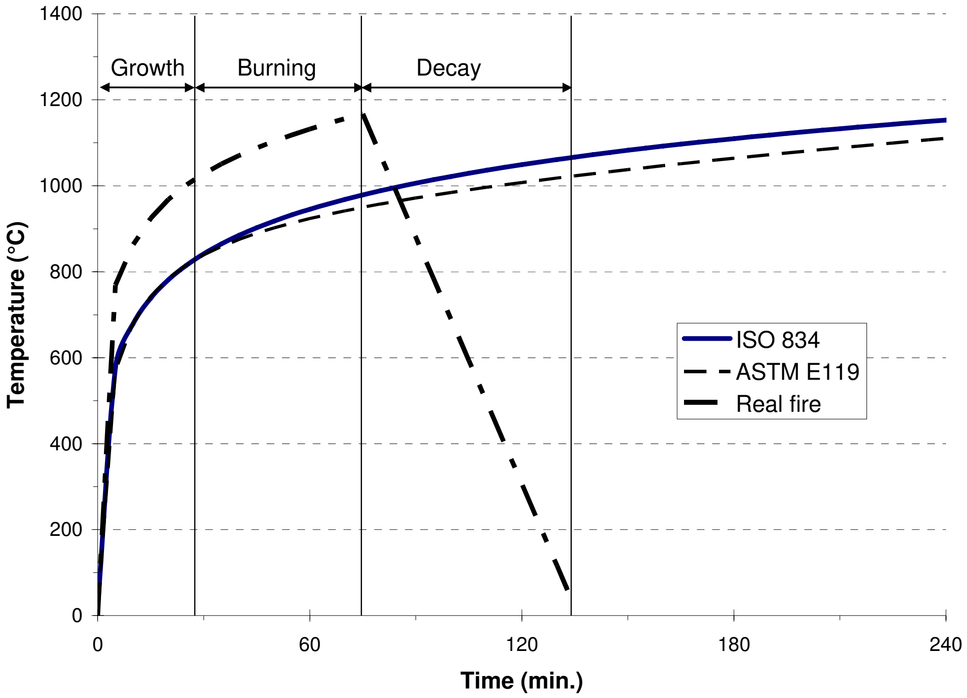


Fig. 1 Comparison of ISO 834, ASTM E119, and a potential real fire time-temperature curve [6,7].

2.1.3 Performance-based methods

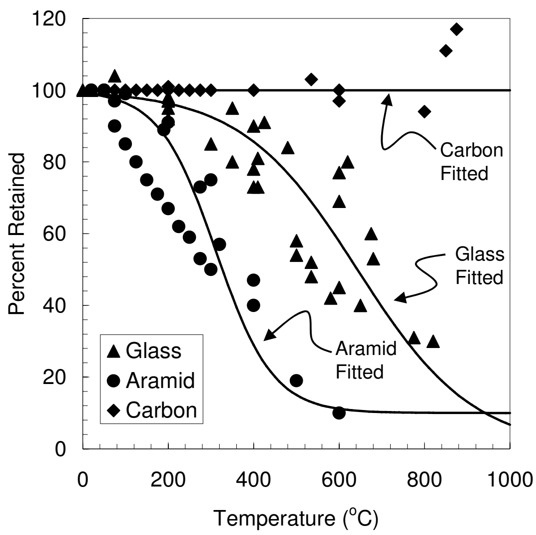
Performance-based methods are more rational procedures for evaluating the fire resistance of structures, but their acceptance with national building codes is more limited. Such methods typically involve engineering calculations of the strength of structural members during a fire event. Such procedures can be relatively simple, such as those suggested by ACI 216 [8] where the strength of the member is calculated based on the expected temperatures in concrete and reinforcing steel at the time of the required fire endurance. The calculated strength must be greater than the effect of the full service load (D+L). Tabulated values of material strength (concrete and reinforcing steel) at different temperatures are provided. The expected temperatures are also obtained from tables or figures that are based on results from fire tests; the temperatures are given for different depths into the concrete. Conducting a thermal analysis of the concrete member in a fire scenario is not part of the current ACI 216 [8] approach. For structures strengthened with FRP, ACI 440 [9] currently recommends ignoring the FRP completely in the fire and then applying the approach of ACI 216 [8]. In this case, the calculated strength of the member must be greater than the effect of the full *strengthened* service load.

In European codes, performance-based methods are more accepted [2,4]. Most of these performance-based methods involve both a thermal analysis and a structural analysis. In the simplest case, a thermal analysis is combined with a structural analysis similar to the approach of ACI 216 [8]. The next step of complexity involves thermo-mechanical numerical analysis. Often, the finite element method is employed and the thermal analysis is usually conducted first and the temperature results used to predict the strength of the member or structure at a given time during the fire. Using such an approach, different potential 'real fire' scenarios can also be investigated [2,4]. More recently, integrated thermal, hydral, and mechanical programs have been developed [2] and are useful for simulating complex phenomena such as concrete spalling.

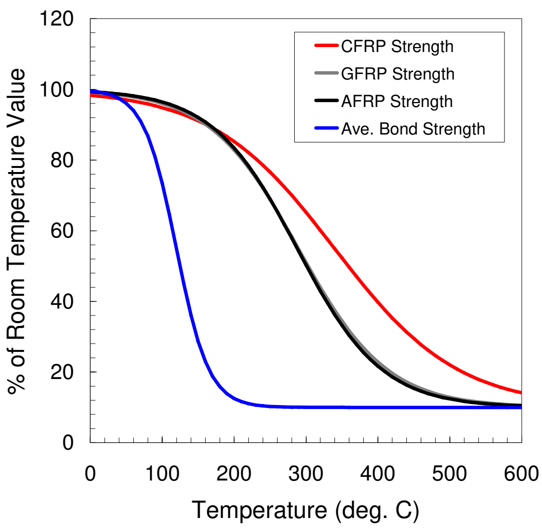
For FRP applications in Europe, fib Bulletin 14 [10] provides some guidance and separates strengthened structures into two classes: with fire protection and without. For FRP-strengthened structures without fire protection, fib recommends ignoring the FRP and calculating the strength of the member using performance-based calculations as described in the previous paragraph. The strength of the member in fire must be greater than the accidental load level (Eq. (2)) for the required fire endurance. For members with fire protection, performance-based methods can still be used to calculate the strength and the thickness of the fire protection should be chosen to limit the increase in temperature in the adhesive between the FRP and the concrete. The guide recommends that this temperature increase be limited to between 50°C and 100°C depending on the adhesive. The guide provides little specific information, but presumably the full strength of the FRP can be used until the specified limit is reached. After this point, the strength of the FRP is ignored.

3 FRP IN FIRE

The performance of FRP-strengthened structures in fire is of great concern for applications of FRP in buildings. In many situations, concerns about fire resistance can severely restrict potential applications of FRP. The fundamental problem is that FRPs are inherently combustible. Furthermore, typical polymer resins for FRPs for civil engineering applications typically have glass transition temperatures between 60°C to 80°C [11, 12]. At temperatures above the glass transition temperature, the polymer softens and degrades. Thus, the load-sharing function of the polymer resin suffers and individual fibres may become overstressed and break [10]. This will lead to eventual failure of the composite at a reduced load capacity. In principle, some fibres themselves are inherently resistant to high temperature. For example, Figure 2(a) shows the strength of fibres at elevated temperatures. Carbon fibres are virtually unaffected by high temperature up to 1000°C whereas glass fibres retain most of their strength to over 600°C. Unfortunately, when combined with the resin as a composite, the high temperature resistance drops considerably as shown in Figure 2 (b). At temperatures between 250°C and 400°C, most composites lose half of their original tensile strength. It should be noted that the results presented in Figure 2 are general results for FRP that are available in the literature, but are, in general, not results for specific materials used for strengthening concrete structures. Tests by Wang et al. [13], however, were conducted on FRP reinforcing bars; they found strength reductions of 50% at temperatures of 250°C for CFRP and 325°C for GFRP bars. Additionally, the tests on bond strength reported in Figure 2(b) were conducted with FRP reinforcing bars in concrete [14]. For FRP strengthened concrete structures, the bond will be affected the most by high temperature because much of the bond strength will likely be lost at temperatures just above 100°C.



(a)



(b)

Fig. 2 FRP properties at high temperature (a) Bare fibre strength[11, 12](b) FRP strength [11, 12] and bond strength to concrete [14].

4 FRP-STRENGTHENED CONCRETE MEMBERS

Relatively little work has been conducted on FRP strengthened concrete members in fire. Some of the first tests were conducted at EMPA in Switzerland on beams strengthened with CFRP laminates [15]. One objective of these tests was to compare CFRP strengthened beams to similar beams strengthened with steel plates. The CFRP strengthened beam performed better in the fire than the beam strengthened with steel plates because of the lower thermal conductivity and lower self-weight of the CFRP. This research also showed that insulating the CFRP could increase the fire endurance of the strengthened beams.

Further work on beams strengthened with CFRP plates was conducted in Belgium [16] and also demonstrated the benefit of insulating with calcium silicate boards. The insulation was most effective if it was provided on both the sides and the bottom of the beams. The work also showed that insulating the ends of the CFRP strips was almost as effective as insulating the whole length of the CFRP. Nevertheless, the reported fire endurances were typically less than 1 hour for the best systems.

4.1 NRC-ISIS program

Over the last few years, a collaborative research effort between ISIS Canada, the National Research Council of Canada, and industry partners has been conducted to study FRP strengthened columns and beams in fire [11, 12, 17-21]. This was the first known research on the performance of FRP confined columns in fire. The research program involves both full-scale fire testing and numerical modelling. The ultimate goal of the research is to develop design guidelines to achieve fire safety for FRP strengthened members in buildings.

The experimental portion of the research consisted of full-scale tests on reinforced concrete T-beams and circular columns, and intermediate-scale tests on slabs (Table 1). The main purpose of the slab tests was to investigate parameters in preparation for the full-scale tests. To date, four T-beams and four circular columns strengthened with CFRP have been tested. In addition, one fire test was conducted on a full-scale square column wrapped with GFRP. All the beams were insulated on the outside of the FRP with spray-applied fire protection. Two different insulation systems were applied: one was gypsum-based and the other was cementitious. The full-scale tests were all conducted with the members subjected to load whereas the slabs were unloaded.

The slab tests were preliminary investigations to estimate the required thickness of insulation for applying to the full-scale beams and columns [17]. Since these slabs were not loaded, the only fire endurance requirements related to the temperature in the internal steel and the transmission of heat through the slab. For the first slab with only 20 mm of insulation, debonding occurred before the end of the fire test and thus the temperature limit for the internal steel was reached relatively early in the test. For slabs 1, 3, and 4, the glass transition temperature, T_g of the FRP was exceeded after approximately 40 minutes into the test whereas for slab 2 with 40 mm of insulation, the T_g value was not reached until more than 100 minutes of fire exposure. Based on these tests, the potential insulation systems were deemed satisfactory and a minimum thickness of 25 mm of insulation was recommended to avoid debonding.

Based on this recommendation, the insulation systems were applied to columns that were strengthened with FRP wraps (Table 1) [11, 12, 18]. All of the insulated columns were able to resist the full service load for over 4 hours even though the T_g of the FRP was exceeded much earlier in the test. Figure 3 shows photographs of a typical column both before and after the fire test and Figure 4 shows the temperature in the FRP for all of the columns. To provide a reference test, one strengthened column (#3) was tested in fire without any insulation. This column achieved a fire endurance rating of 210 minutes. Furthermore, the uninsulated column failed under the applied service load whereas the other circular columns only failed when the load was increased substantially after more than 5 hours of fire exposure. Thus, the insulation systems were effective in increasing the fire endurance and enhancing the strength of the original column in fire by keeping the temperatures in the internal steel and concrete below critical levels.

Similar full-scale tests were conducted on T-beams (Figure 5) strengthened with CFRP sheets (Table 1) [19, 20]. In this case, all four beams were insulated. All of these beams achieved fire endurance ratings of over 4 hours. Figure 6 shows the temperature at the level of the FRP for all four beams. After this fire exposure, the load applied to the beams was increased to the maximum capacity of the test frame. Since the beams did not fail under these conditions, they were tested to failure afterwards at room temperature. The residual strength of the beams was found to be close to the strength of the beams without the FRP. Once again, the insulation systems were demonstrated to be effective in protecting the original strength of the reinforced concrete member.

In addition to the testing program, numerical models have been developed to predict the behaviour in fire of FRP-strengthened beam and circular columns [12, 19-21]. For each type of member, two computer programs were developed; one program conducts a thermal analysis of the member and the other performs the structural analysis using temperature-dependent material properties. The computer programs have also been validated against the test results. As an example, Figure 7 shows predicted and measured temperatures in T-beam 2. The predicted temperatures compare reasonably well with the measurements given the error inherent in the measurements and the sensitivity of the thermal properties of the concrete to the amount of moisture and the migration of moisture in the concrete.

Table 1 Summary of results from fire tests [11, 12, 17-20].

Member	Dimensions (mm)	FRP	Insulation	Test load ratio	Fire endurance (min)	Failure load or moment	Predicted room temperature strength
Circular column 1	ϕ 400 × 3810	1 layer CFRP-A	VG 30 mm	0.50	> 300	4437 kN	5094 kN
Circular column 2	ϕ 400 × 3810	1 layer CFRP-A	VG 60 mm	0.50	> 300	4680 kN	5094 kN
Circular column 3	ϕ 400 × 3810	2 layers CFRP-B	None	0.56	210	2635 kN	4720 kN
Circular column 4	ϕ 400 × 3810	2 layers CFRP-B	Cem	0.56	> 300	4583 kN	4720 kN
Square column	400 × 400 × 3810	1 layer GFRP	VG 40 mm	0.69	> 240	3093 kN	4483 kN
T-beam 1	Length 3900 h = 400 h_s = 150 b_s = 1220 b_w = 300 Figure 5	1 layer CFRP-A	VG 25 mm	0.53	> 240	142 kN.m	130 kN.m
T-beam 2		1 layer CFRP-A	VG 40 mm	0.53	> 240	142 kN.m	130 kN.m
T-beam 3		1 layer CFRP-B	Cem 30 mm	0.50	> 240	146 kN.m	145 kN.m
T-beam 4		1 layer CFRP-B	Cem 30 mm	0.50	> 240	120 kN.m	145 kN.m
Slab 1	150 × 950 × 1330	1 layer CFRP-A	VG 20 mm	No load	147	NA	NA
Slab 2	150 × 950 × 1330	1 layer CFRP-A	VG 40 mm	No load	> 240	NA	NA
Slab 3	150 × 950 × 1330	1 layer CFRP-B	Cem 40 mm	No load	> 240	NA	NA
Slab 4	150 × 950 × 1330	1 layer CFRP-B	Cem 40 mm	No load	> 240	NA	NA

Notes: CFRP-A - t_f = 1.0 mm per layer, f_f = 745 MPa, ε_f = 0.012, E_f = 62 GPa, T_g = 93°C
CFRP-B - t_f = 0.165 mm per layer, f_f = 3800 MPa, ε_f = 0.0167, E_f = 227 GPa, T_g = 71°C
VG - gypsum-based insulation, thermal conductivity 0.082 W/m-ºC at room temperature
Cem - cementitious insulation, thermal conductivity 0.37 W/m-ºC at room temperature
The test load ratio is the test load divided by the design strength of the strengthened member.
NA – not applicable
h = overall height of T-beam, h_s = height of slab, b_s = breadth of slab, b_w = breadth of web
t_f = thickness of one layer of FRP, f_f = strength of FRP, ε_f = maximum strain at failure for FRP
E_f = modulus of elasticity of FRP, T_g = glass transition temperature of FRP

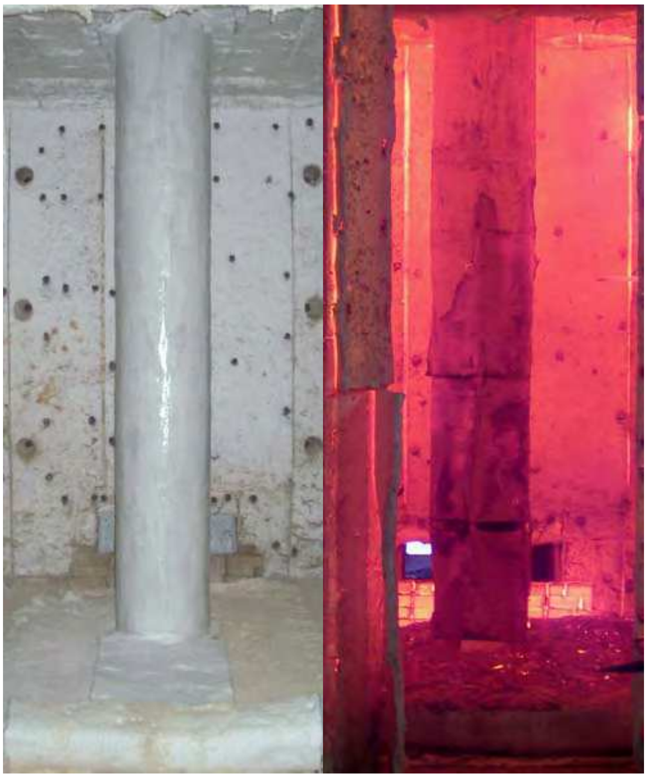


Fig. 3 Typical circular column before and after fire testing [12].

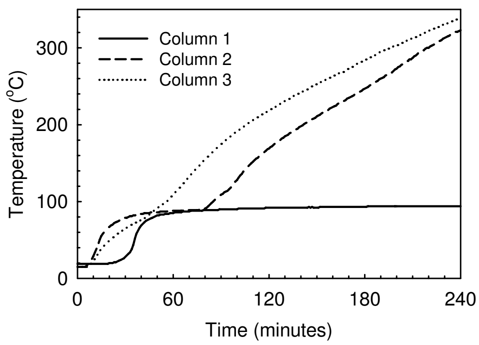


Fig. 4 Temperature in the FRP for three of the columns [11].

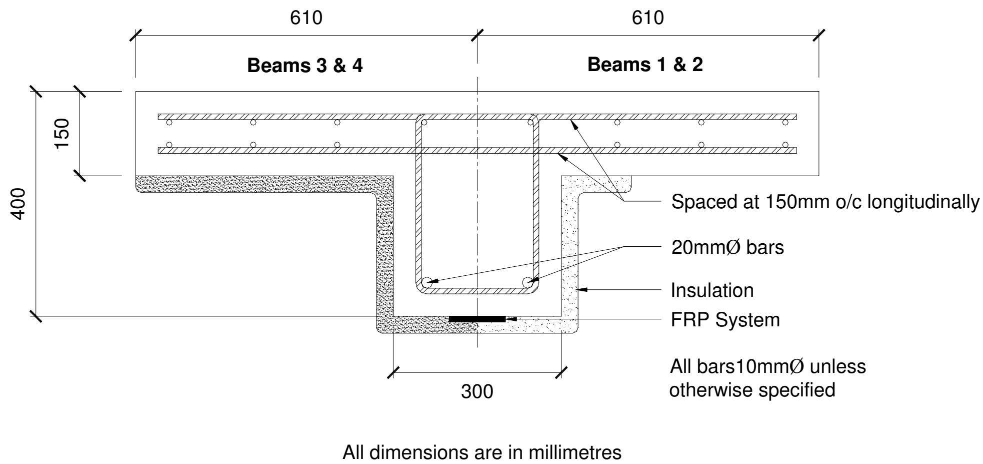


Fig. 5 Cross-sectional dimensions of the T-beams [11].

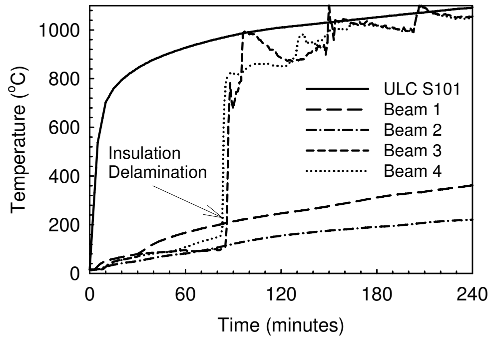


Fig. 6 Temperatures at the level of the FRP for all beams [11].

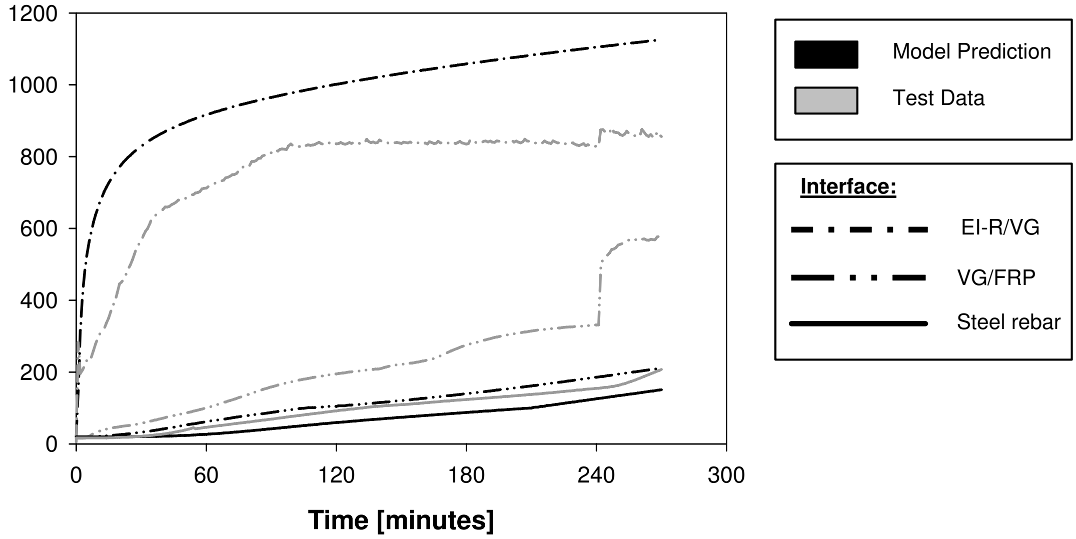


Fig. 7 Comparison of measured and predicted temperatures in T-beam 2 [19].

5 DESIGN GUIDELINES

Current information about design requirements for applying FRP strengthening in buildings is either not available or misunderstood. For example, some jurisdictions will not accept the application of FRP unless the full strength of the FRP can be maintained for the full duration of the fire. Such an approach is contrary to the concept of fire resistance as defined in section 2.1 which states that the fire resistance depends on the performance of the element and not just one specific material. More knowledge and understanding of the behaviour of FRP strengthened concrete structures is still required before detailed design requirements can be established but this section will make some preliminary proposals for design procedures.

As a starting point, FRPs need to be designed with some type of fire protection to restrict the evolution of smoke and prevent excessive flame spread. The specific requirements will depend on the classification of the building. It should be noted that most commercial FRP systems have formulations or coatings that meet the requirements of most building codes.

Designing for structural integrity during a fire scenario is a more difficult proposition. One approach is to conduct fire tests as described in section 2.1.1. Although this is an acceptable approach from a safety perspective, the cost of conducting such tests for each type of structural application is prohibitive. Another approach would be to develop prescriptive procedures as described in section

2.1.2. Such a prescriptive approach is not in step with current thinking about fire design and exhaustive tests would be required to develop such procedures.

Thus, the best approach for design is to apply performance-based design procedures. Such procedures are already recommended by fib Bulletin 14 [10]. In North America, the ACI 216 [8] approach provides the best starting point for conducting such an analysis as recommended by ACI 440 [9]. However, these existing guidelines are really only appropriate for situations where the existing concrete structure would still be adequate under fire conditions with the increased expected load consistent with the strengthened structure. In many cases, FRP strengthening would not be viable given this restriction. Another failing of this approach is that the FRP will combust and add some fuel to the fire that can increase the intensity of the fire on the surface of the concrete. Therefore, some extra conservatism should be incorporated into such an approach to allow for this effect unless tests have demonstrated otherwise. Thus, performance-based procedures that take into account fire protection schemes such as insulation are required.

Such procedures could readily be adopted into European practice and fib Bulletin 14 [10] mentions limited application of such an approach. Any of the approaches described in section 2.1.3 would be acceptable, but any models should be validated against existing and additional test data on full-scale FRP strengthened concrete members. Additional testing would be required for insulation systems to demonstrate their effectiveness for the full duration of the expected fire. Furthermore, for such models to be effectively used in design, more information is required on the thermal properties of potential insulating materials (especially at high temperatures). Another vital piece of information is better data on the strength and bond degradation of FRP. Both properties degrade at temperatures above T_g, but the amount of degradation and the rate of degradation with increasing temperature needs to be better quantified. For example, a critical temperature could be defined as the temperature above which the FRP no longer contributes to the strength of the structure. Given current knowledge, this critical temperature is conservatively recommended to be taken as T_g but better information could increase this limit. Also, strengthening schemes that do not rely solely on the bond of the FRP to concrete may be shown to have higher critical temperatures. Finally, the models should include an evaluation of the strength of the whole structural element. Thus, failure would only be deemed to occur when the strength of the element was no longer sufficient to carry the expected loads during fire and not just at the point when the FRP itself reached its critical temperature.

6 CONCLUSIONS

The following conclusions are deduced from the results and discussion in this paper:

- FRP-strengthened concrete structures can have adequate performance in fire, and can achieve fire endurance ratings of more than 4 hours if suitable insulating fire protection is provided.
- Performance-based fire safety design methods are recommended as the best potential design approach for FRP-strengthened concrete structures. Such methods would include modelling of thermal and mechanical behaviour of structural elements.
- To effectively develop such performance-based methods, more research is required to develop and validate simulation methods, to better characterize the effects of high temperatures on the mechanical behaviour of FRP, and to fully evaluate the thermal properties of potential external insulation systems for FRP.

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