RELIABILITY ANALYSIS OF PRECAST, PRESTRESSED CONCRETE BEAMS EXPOSED TO FIRE

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ABSTRACT

A reliability analysis is conducted on precast, prestressed concrete beams subjected to a fire load. Load random variables are taken to be dead load, sustained live load, and fire temperature, while mean fire temperature is based on a standard fire. Resistance is in terms of moment capacity, with random variables taken as prestressing steel ultimate strength, concrete compressive strength, placement depth of strands, beam width, and thermal diffusivity. A semi-empirical, calibrated model is used to estimate beam moment capacity as a function of fire exposure time. Various beam parameters were considered in the analysis, including cover, aggregate type, concrete compressive strength, dead to live load ratio, prestressing steel reinforcement ratio, and other factors. Reliability was computed from zero to four hours of fire exposure using Monte Carlo simulation. It was found that reliability decreased nonlinearly as a function of time, while the most significant parameters were concrete cover, proportion of end strands to total strands, and load ratio.

Keywords: fire, reliability, beam, precast, prestressed, concrete

INTRODUCTION

Every year building fires cause significant loss of human life and tremendous damage to property. In 2005 alone, fires caused 3,762 deaths, 17,925 civilian injuries, and $10.7 billion in property damage in the United States[1]. In addition to fire prevention techniques, various means of fire damage mitigation are used. Some of these include providing the proper architectural planning of exits and escape routes; the use of active fire protection techniques such as sprinklers to reduce the number of severe fires; and, providing structural fire protection to achieve a minimum fire resistance rating, with the intent to allow structural members to maintain their integrity throughout the escape and firefighting phases. A fire rating is frequently expressed in terms of time; i.e. the time which a member is expected to maintain its structural integrity when subjected to a standard test fire.

Traditionally, a structural member's fire resistance rating is determined by either conducting a fire endurance test such as specified in ASTM E119[2], or by calculation, which can be used for limited cases when previous fire endurance test results exist for similar structures[3-5]. For prestressed concrete (PC) slabs and beams, the Prestress Concrete Institute provides a fire rating procedure as well in MNL-124-89[6]. A fire rating, however, provides no quantitative measure of safety in terms of failure probability, and the reliability of PC structures exposed to fire loads is largely unknown. This is not consistent with prevalent Load and Resistance Factor Design (LRFD) philosophy, where load and resistance factors in various load combinations were specifically developed using probabilistic principles to insure a consistent and adequate level of safety for structural members of the same importance level. In the case of fire resistance, there is no guarantee that members have a consistent level of safety, and in fact it is well-known that significant performance variation results in traditional prescriptive fire load design methods[7-11].

Over the last several decades, there has been limited research on the probabilistic analysis of structures exposed to fire, though diverse types of analyses have been considered[12-15]. Based on an analysis of load frequency, Ellingwood[16] summarized practical load combinations that need to be considered for fire design. Only a few studies were identified in the technical literature that considered the failure probabilities of reinforced concrete structural elements exposed to fire[17-23], and currently, there exists no systematic assessment of the reliability of PC beams exposed to fire that have been designed to current ACI 318[24] standards considering load and resistance uncertainties, nor an examination of the changes in reliability as various important beam parameters change. As a step in this direction, this study estimates the reliability of a selection of prestressed concrete beams designed according to ACI 318 Code[24] exposed to a standard fire. The intent is to estimate a baseline of current safety levels, as well as to examine how various parameters affect beam reliability when exposed to fire.

LOAD MODELS

For design as well as reliability analysis, the various loads that a structure may be subjected to over its design lifetime must be considered, such as dead load, occupancy and roof live loads, snow load, wind, and earthquake, among others. However, based on an analysis of the frequency of occurrence of these loads relative to that of a structurally significant fire, it can be determined that some load combinations with fire can be practically neglected for calculation of reliability indices β that are at or below typical code targets (approximately when $\beta \leq 3.5 - 4$). By examining the coincidence rates of various extreme loads (in the United States) with a structurally significant fire, Ellingwood[16] determined that the only combination that need be considered includes the sustained loads: dead load and sustained (occupancy) live load.

Statistical parameters for dead load, the permanent gravity loads on the structure, are well known and available in the literature. Dead load is typically assigned a bias factor λ (ratio of mean value to nominal value) of 1.05 and coefficient of variation (COV) of 0.10. It is normally distributed[25].

Occupancy live load has two components: transient live load and sustained live load. Transient live load represents extreme loads for rare, special events such as emergencies, crowding, or remodeling. This load component becomes important in typical reliability analysis of structural members used for code calibration, such as that conducted for the 2002 ACI 318 Code[24, 26]. As noted above, this live load component is generally not important when considering fire due to its low coincidence probability, and is not considered further here. Sustained, or 'arbitrary-point-in-time' load, L_s, represents the typical load on the structure at any particular time, primarily representing movable items such as furniture, partitions, and other contents. Bias factor for sustained live load has been reported to range from approximately 0.24 – 0.50, depending on tributary area and occupancy type, with COV from 60-0.65. L_s is typically modeled with a gamma distribution[16, 26]. In this study, sustained live load is taken with bias factor of 0.24 and COV of 0.65[25].

Depending on fuel load, ventilation, convective and radiative properties of the compartment, as well as other factors, fires will produce various temperature-time profiles. It is this resulting temperature profile which causes a temperature rise in the structural member and causes a loss of capacity as a function of time. To conduct consistent reliability analysis, it is useful to consider a standard fire profile. Therefore, the mean value of fire temperature $\overline{T}$ is taken to be that given by the standard fire temperature (T)-time (t) profile used for fire rating in ASTM E119[2], which is given in Figure 1.

As fire temperature is considered a random variable (RV) in this study, statistical information regarding its variability (i.e. COV) is also needed. The variation in temperature experienced by a structural element in a fire depends on various parameters including fuel load, ventilation, room geometry, and other compartment characteristics. In this study, fire temperature COV was estimated for a typical range of compartment characteristics by determining how a variation in fuel load, essentially the sustained live load, in the compartment affects component heat load.

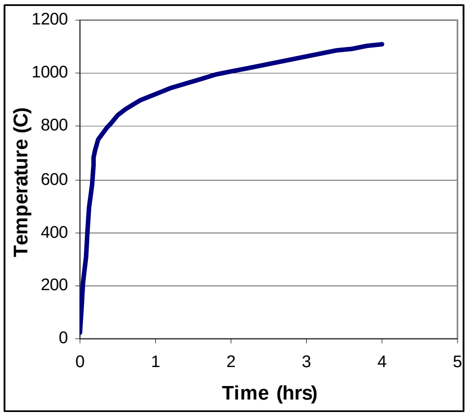

Fig. 1. Standard Fire Time-Temperature Curve

Based on a series of compartment burn tests, this relationship was developed by Hamarthy and Mehaffey[27]. Using this relationship in conjunction with Monte Carlo simulation (MCS), the COV of the resulting heat loads varied depending on the compartment characteristics considered, with the most representative case having a COV of 0.45. Note that there is a strong relationship between fuel load and fire heat load, and thus the live load L_s and temperature T random variables are not independent. In this study, both the independent as well as fully-dependent cases were considered. However, little difference was found in the reliability results between the two cases, and thus the relationship between these RVs was taken as fully dependent.

RESISTANCE MODEL

Various failure modes may be considered for members subjected to fires, including stability-related criteria such as strength and deflection; integrity criteria to prevent fire and gasses from penetrating through the member, and insulation criteria that limit the temperature on the cold side of the member[28, 11]. Integrity and insulation criteria are generally more useful for partitions and walls, while limits on serviceability become difficult to quantify and are not typically used for reliability analysis. Thus in this study, resistance is based on moment capacity. To calculate moment capacity as a function of temperature, $M_n(T)$, the effect that temperature has on several variables entering assessment of $M_n(T)$, including prestress level, f_{ps}, depth of compressive stress block, a, and yield stress of nonprestressing steel, f_y, if present, must be determined.

RVs important for reliability analysis for PC beams in flexure are prestressing steel ultimate strength f_{pu}, nonprestressing steel yield strength f_y, depth of prestressing and nonprestressing steel placement in the section, d_p and d, concrete compressive strength f_c', beam width b, and professional factor P, the latter of which accounts for uncertainties in the analysis model used for design. The statistical parameters for these RVs are taken from Nowak and Szerszen[25] for pre-tensioned, plant-cast PC beams, where distributions are reported as normal. There is insufficient statistical data to accurately determine the variation of steel and concrete strengths as a function of temperature[12, 17]. Therefore, the

COVs of f_{pu}, f_y and f_c' at elevated temperatures is taken as that at ambient temperature. For high temperature analysis, thermal diffusivity α is also considered as an RV, with COV taken from Shin et al.[29]. Mean value for α is highly variable and dependent on the type of section and material properties considered. As mean α significantly impacts results, in this study it is determined with a special calibration procedure detailed below. A summary of the statistical parameters taken for resistance RVs are given in Table 1.

Table 1. Resistance Random Variable Parameters

RV	bias factor	COV
f_{pu}	1.04	0.025
f_y	1.145	0.05
d_p	1.0	0.025
d	0.99	0.04
f_c'	1.38*	0.12*
b	1.01	0.04
α	1.0	0.06
P	1.02	0.06

*for 35 MPa (5 ksi) plant-cast concrete

In this study, fire acts on the bottom and sides of the beam section, while the top is assumed be in a different compartment or protected by a floor slab. When exposed to fire, loss of capacity occurs because of reduced strength of the steel and the concrete, although the former is much more significant when beams are under-reinforced. To describe the loss of strength of a PC beam, a fire-based resistance model must account for two major effects: the change of temperature in the material at various points of importance, such as the steel reinforcement and the concrete compressive block; and how the change in temperature affects strength. The latter effect is generally modeled by fitting curves to experimental results, though there is much scatter in the data.

Various temperature-yield stress curves have been proposed for steels[30,31,6]. For this study, prestressing steel tensile strength reduction factor is based on the strength-temperature relationship given in MNL-124-89, The Design for Fire Resistance of Precast Prestressed Concrete[6], as shown in Figure 2.

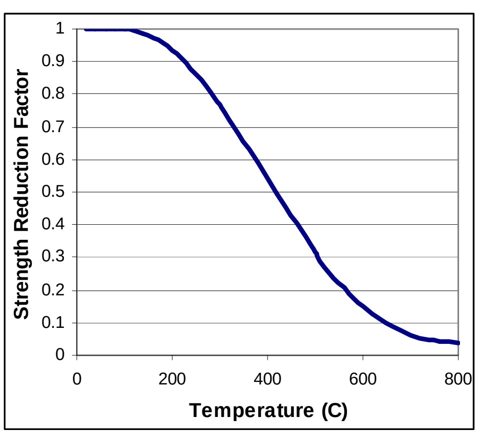

Fig. 2. Prestressing Steel High-Temperature Strength Reduction Factor

For concrete, researchers have found differing compressive strength-temperature relationships[27,32-37]. However, in tension-controlled beams, concrete properties have minimal impact on moment capacity, which is governed by the tension steel, and the choice of concrete model used has little influence on the final results. In this study, the reduced section (500 °C isotherm) method[38] is used, where $f_c'(T)$ is held constant in the analysis ($f_c'(T)=f_c'$) but the size of the effective compression block is reduced. This reduction is approximated by eliminating the compressive capacity of concrete at locations where internal temperatures are greater than 500 ºC ($f_c'=0$), and concrete is given full compressive strength ($f_c'(T)= f_c'$) at locations in the section where temperature is less than 500 ºC. Although more refined models are available[39], minimal difference results in $M_n(T)$ when under-reinforced beams are considered.

A more difficult effect to model is the change in temperature throughout the section as external temperature and time change. This is a function of section geometry, material density, specific heat, and other factors. Various models have been proposed to approximate this behavior, including finite element approaches[40,41,22] as well as semi-empirical approaches[42, 39].

For the reliability analysis used in this paper, a large number of simulations is needed. This practically precludes use of involved FEA approaches, as the required computational effort becomes too great. However, FEA is generally only needed for complex, non-standard cases, while the empirical approaches available can often provide good results for regularly-shaped sections subjected to standard fires, which are of interest to this study. To determine how internal temperature changes in the section as a function of time (t; hours) and external temperature (T), a specially calibrated version of Wickstrom's model[42] is used. In this approach, the temperature of the steel reinforcement T_r is given by:

$$T_r = (n_w(n_x + n_y - 2n_xn_y) + (n_xn_y))T \qquad (1)$$

where

$n_w = 1 - 0.0616t^{-0.88}$

$n_{s\,(s= x,y)} = 0.18\ln(\alpha_r t/s^2) - 0.81$

s is the distance of the center of the reinforcement bar considered to the outer edge of the concrete section, measured in the x or y coordinate direction, as appropriate (m), with a limit imposed of: $s \geq 2h - 3.6(0.0015t)^{0.5}$

α_r is the ratio of thermal diffusivity considered to a reference value of $0.417\text{x}10^6$ m^2/s.

To account for reductions of concrete compressive strength, the position of the 500 ºC isotherm in the section is needed. For compressive blocks exposed to fire from the sides of the section, using the Wickstrom model, it is given by the following, measured from the outer edge of the beam[43]:

$$x_{500} = \sqrt{\frac{\alpha_r t}{\exp\left(4.5 + \dfrac{480}{0.18 n_w T}\right)}} \qquad (2)$$

Once this is located, the effective width of the compression block as a function of concrete temperature (T_c) becomes: $b(T_c) = b - 2x_{500}$. Here the '2' assumes that the fire is encroaching on both sides of the beam. As noted above, this reduced effective section width takes the place of reducing f_c' at higher temperatures.

An important factor in Wickstrom's model is the thermal diffusivity ratio α_r. It is known that α_r changes based on density, aggregate type, temperature, and other factors[29,43-46]. Increasing α_r results in faster heat transfer through the section, and a decrease in time to failure. A good estimation of α_r is important because capacity results are sensitive to this value, as shown in Figure 3. In this study, α_r is determined by calibration, which is performed for each specific beam considered in the analysis. In this procedure, an α_r is determined such that Wickstrom's model, when used to determine internal temperatures when moment capacity is computed, provides a fire rating, or predicted time of failure, consistent with generally accepted experimental results. This resulting α_r value may not only represent the effects of different material thermal properties, but also how other factors not directly included in the Wickstrom model would effectively modify heat flow as well.

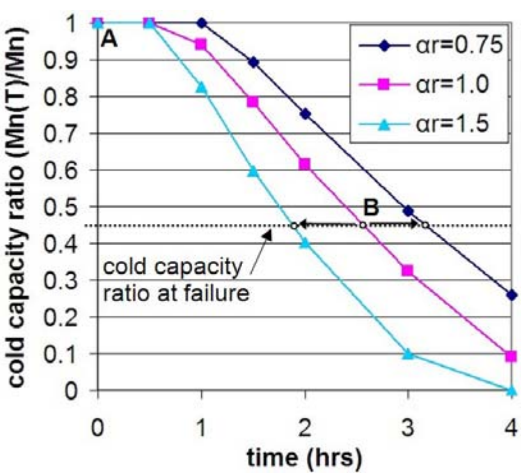

Fig. 3. Effect of Thermal Diffusivity in the Wickstrom Model

The α_r calibration is made to the fire rating method presented in MNL-124-89, which is based on a series of fire tests conducted on prestressed concrete members. In this study, the MNL-124-89 approach is first used to determine a nominal fire rating, or time to beam failure, to the nearest minute, for the specific beam being considered for analysis. Then, an effective α_r is determined in Wickstrom's model that would result in a matching prediction of time–to-failure, anchoring point 'B" on the horizontal line in Figure 3, for the particular beam considered. Depending on aggregate type and other section characteristics, typical values of effective α_r ranged from approximately 0.75-1.5, which appear reasonable, and are within spread of actual α values reported for different concrete materials[43,46]. Once the Wickstom model is calibrated to the specific beam case considered, it can then be used to estimate beam capacity as a function of time, between the fixed points A and B in Figure 3.

BEAMS CONSIDERED

By studying the load and resistance models used, it can be seen that the following parameters may effect capacity, and therefore reliability, of beams exposed to fire if designed to satisfy ACI 318: cover, aggregate type, f'c, D/(D+L) ratio, reinforcement ratio, and proportion of end strands (i.e. those nearest the sides of the beam) to total strands in a layer. Therefore, the reliability of various rectangular beams were studied, from t=0 to t=4 hours of fire exposure, by varying these parameters. Restraint moments caused by support conditions is also an important parameter. As the temperature of a PC member increases and it expands, if sufficiently constrained, significant and typically beneficial constraint moments are produced. However, due to a lack of experimental data regarding the behavior of PC beams exposed to fires with different support conditions, the beams considered in this study are assumed to be simply-supported with no additional axial or rotational constraints during a fire.

Two base beams are considered, with several variations as needed, to investigate the affect of the important parameters discussed above. The first base beam is taken as a rectangular section with b=305 mm (12 in), h=610 mm (24 in), f'c=35 MPa (5 ksi) with siliceous aggregate, seven 13 mm (0.5 in) nominal diameter low-relaxation 7-wire prestressing strands with nominal ultimate strength of f_{pu}=1860 MPa (270 ksi). The tendons are placed in one layer with minimum cover allowed by ACI 318 for precast (plant control manufacturing conditions, not exposed to weather), 20 mm (¾ in) from the bottom and sides of the beam. Note that this cover requirement would also allow the use of a #3 stirrup as well (miminum cover 10 mm (3/8 in)). Although allowed by code, this minimum cover is not typical, as it is difficult, if not impossible, to meet code-specified serviceability requirements at transfer with this extreme tendon placement in the beam for most realistic cases. Moreover, higher fire ratings are typically required than are achieved with this minimum cover section. Therefore, this first base beam should be considered to represent a lower bound, or 'worst case' for reliability with regard to fire exposure.

The second base beam represents a more typical case with regard to tendon placement, and thus cover is greater than the minimum allowed. As such, this beam does not provide the lowest reliability indices when exposed to fire. This beam is based on the rectangular precast section 12RB20 in the PCI Design Handbook[47], which has b=305 mm (12 in), h=510 mm (20 in), and eight prestressing strands of the type discussed above. Here strands are placed in two layers of four, with a centroid of 75 mm (3 in) above the bottom of the beam. Here, tendons in the lower layer have 45 mm (1.75 in) cover to the bottom and sides. Otherwise, this section is the same as the first base beam. Both base beams simply span 7.5 m (25 ft) and are uniformly loaded with a D/(D+L) ratio of 0.50. All beam variations are minimally designed according to ACI 318 in terms of moment capacity ($\phi M_n = M_u$), with the design load combination relevant to this study, as discussed above: 1.2D + 1.6L. The beams are under-reinforced, with ϕ taken as 0.90. For some cases, compression steel is added to insure this condition.

RELIABILITY ANALYSIS

Direct Monte Carlo simulation is used in this analysis to evaluate the limit state:

$$g = M_n(T) - D_M - L_{sM}, \quad (3)$$

Where D_M is the dead load moment and L_{sM} the live load moment. For each sample in the analysis, a α_r value is determined that results in Wickstrom's model matching the fire rating predicted from the MNL-124-89 result for the beam. Generalized reliability index is then reported in the results as $\beta = -\Phi^{-1}(p_f)$, where Φ is the standard normal cumulative distribution function. The number of simulations *n* varies in the analysis to maintain sufficient accuracy and precision, depending on the expected failure probability. The number of simulations ranged from $1\text{x}10^5$- $1\text{x}10^{10}$, depending on the time and beam considered.

RESULTS

Reliability indices (β) as a function of time are given in Figures 4-7. For all beams, the base cold strength reliability index (t=0) is approximately 5.2. Note that this is much higher than the cold-strength values reported by Nowak and Szerszen[25] in the ACI 318 Code calibration, which ranged from approximately 3.5-4.4 for the D+L load combination for reinforced concrete beams and 4.2 on average for PC beams (for designs with $\phi = 0.90$). The reason for the discrepancy is the live load model used. Recall sustained live load is considered in combination with fire, whereas for the ACI Code calibration (neglecting fire load), transient live load (i.e. 50 year maximum) is considered, which is accompanied by a significantly higher bias factor. Therefore, it should be kept in mind that values on the graphs represent reliabilities of beams exposed to fire (T) in combination with arbitrary-point-in-time dead and live load values: $D+L_s+T$. For reliability indices beyond about 3.5, results will be governed by load combinations other than fire, with values shown in Nowak and Szerszen[25].

Figure 4 shows the effect of D/(D+L) ratio for base beam 1 (B1) and base beam 2 (B2). As can be seen, increasing this load ratio generally decreases reliability across all times. A similar effect was observed by Nowak and Szerszen[25] for cold strength reinforced concrete beam reliability. For beam 2, the significant effect of load ratio is obvious, where large differences are present near mean failure time, which is at the point on the graph where β=0 (i.e. p_f = 0.50). For beam 1, which has a much lower fire rating (due to less cover), differences are less obvious, but still significant. Thus D/(D+L) load ratio has a substantial influence on reliability.

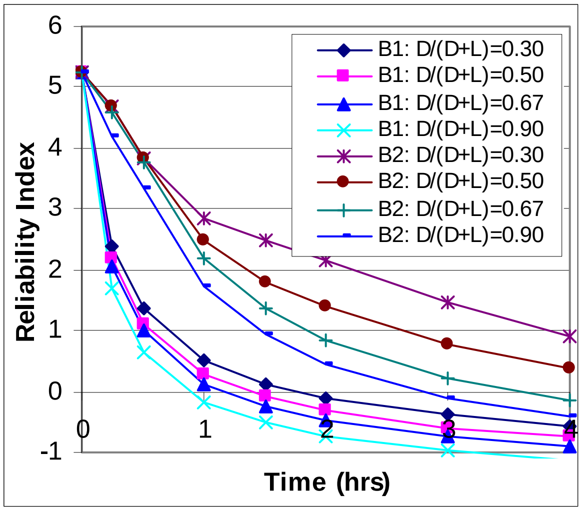

Fig. 4. Effect of D/(D+L) Ratio

Concrete cover is recognized as a critical measure of fire endurance, and this is borne out in the reliability indices presented in Figures 5 and 6, which present results for variations of cover for beam 1 (Fig. 5) and beam 2 (Fig. 6), where the distance from the bottom of the beam to the prestressing strand centroid (y_b) is varied. Here, a 25 mm (1 in) increase in cover provides large differences in mean failure times, close to 2 hours in beam 1 (Fig. 5) and what appears to be over 1 hour in beam 2 (Fig. 6).

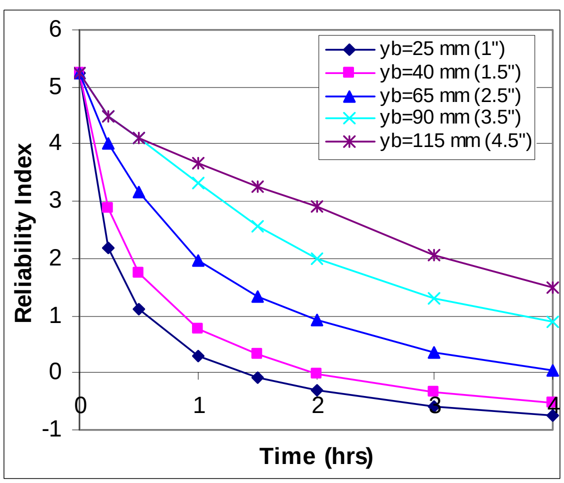

Fig. 5. Effect of Concrete Cover, Base Beam 1

The number of end strands to total tension strands in a layer (end strands: total strands) is studied in Figure 7, which shows results for beams 1 and 2 by varying this proportion. For these beams, strand area was adjusted such that prestressing reinforcement ratio is approximately equal in all cases. Increasing the end strand : total strand ratio decreases reliability since the effective cover decreases. As shown in the figure, minimal differences in reliability resulted in these two cases for beam 1. However, for beam 2, a large impact on reliability was observed.

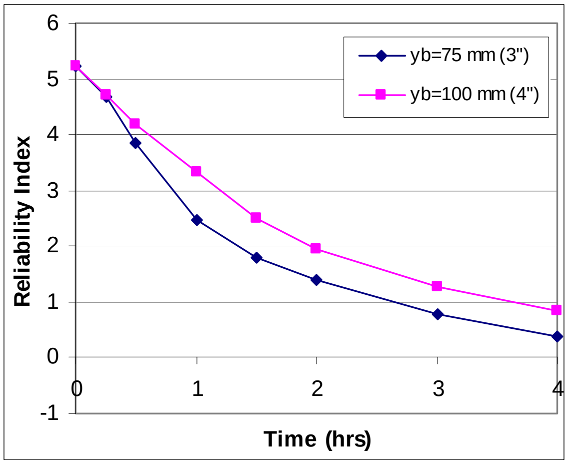

Fig. 6. Effect of Concrete Cover, Base Beam 2

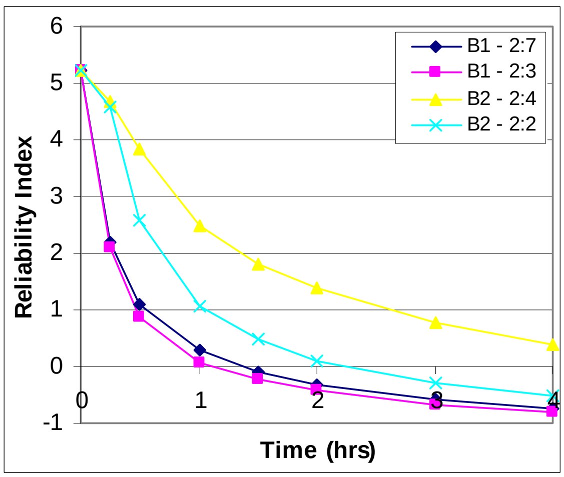

Fig. 7. Effect of Proportion of End Strands to Total Strands Per Layer

Various other factors were explored and were found to have a minor impact on beam reliability. The first of these was the effect of siliceous or carbonate aggregate type, as the use of siliceous aggregates generally increases thermal diffusivity over carbonate aggregates, resulting in a faster rate of temperature rise in the concrete and reinforcement. The effect, however, was found to have an insignificant effect on reliability for the beams studied. A higher strength concrete of 70 MPa (10 ksi) was also considered. However, as expected for under-reinforced beams, it was found to have little effect on reliability.

Similarly, altering prestressing reinforcement ratios within reasonable ranges had minimal impact on beam reliability, with small increases realized for larger reinforcement ratios. The addition of nonprestressing steel as well as compressive steel were considered as well, but were found to have an insignificant effect on reliability (keeping in mind that loads are similarly increased to match the new design capacities).

Although the results are primarily of interest from a research perspective, some results may be of use to design engineers as well. In particular, the figures can be used to estimate beam characteristics necessary to maintain a desired safety level considering a particular fire exposure duration. The target reliability index for concrete beams designed to ACI 318 in cold conditions is 3.5. By consulting the figures, one can quickly estimate the beam characteristics needed to maintain this reliability level for a considered time of fire exposure. For example, if the desire is to maintain the base cold-strength reliability for up to a one hour of fire duration, by consulting Figure 5, it is apparent that beams with y_b below 90 mm (3.5 in) are unlikely to have the ability to satisfy this criteria.

CONCLUSION

A reliability analysis was conducted for various prestressed concrete beams designed according to ACI 318 that are exposed to fire. Based on the load and resistance models used, it was found that most beams had a cold-strength reliability index of approximately 5.2 while exposed to dead load and sustained live load, with reliability quickly decreasing as fire exposure time increases. The most significant parameters on reliability were concrete cover; proportion of end strands to total strands, and D/(D+L) ratio. Aggregate type, concrete compressive strength, reinforcement ratio, and presence of nonprestressing steel or compression steel generally had minor effects on reliability.

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