PRECAST CONCRETE HOLLOW CORE SLABS EXPOSED TO ELEVATED TEMPERATURES IN TERMS OF SHEAR DETERIORATIONS – REVIEW ARTICLE


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Hollow core units were developed in the 1950s when long-line prestressing techniques evolved. Ever since then extensive studies followed concerning this particular field for concrete structures. Design methods of hollow core slabs have no requirements for transverse reinforcements which make it more prone to shear failure especially at elevated temperatures. It is understandable that the behaviour of hollow core slabs under elevated temperatures is more complex than that of solid slabs. The longitudinal wholes cause discontinuity in thermal transfer even though webs are effective parts transferring thermal loads to the unexposed parts of the structural element. Even if several influencing parameters have been investigated in order to evaluate their influence on different failure mechanisms of the hollow core slabs at elevated temperatures, still further efforts are needed. This paper presents a review for influencing parameters, failure modes and code provisions for hollow core slabs at elevated temperatures.

Keywords: hollow core slabs, precast, prestressed, shear failure, elevated temperatures

1. INTRODUCTION

1.1 Preface

Hollow core (HC) slabs are generally precast prestressed concrete slabs made primarily for floor structures in buildings. HC slabs also have applications as both vertical and horizontal wall panels, spandrel members, and bridge deck slabs (PCI, 2015). Since 1950`s, these flooring units have been widely used in Europe, USA and many countries all over the world. The width of the slabs are typically 1.2 m whereas depth of the slab depends on the desired span based on design restrictions. It generally ranges from 200 to 400 mm (*Fig. 1*).

HC slabs have no reinforcement other than the longitudinal prestressing wires or strands, anchored by bond. Subsequently it uses 30% less concrete and 50% less steel. As a result, HC slabs possess lower shear capacity as compared to traditional solid slabs (Fellinger, 2004). Therefore, HC slabs, unlike solid slabs in which failure is predominantly governed by flexural capacity, are susceptible to shear failure at both ambient and elevated temperatures (Rahman et al., 2012). Finally, the production of HC slabs causes almost no waste since all lost material is collected and reused as granulate in the same precast plant (Fellinger, 2004).

1.2 Manufacturing of hollow core slab elements

Seven major manufacturing systems of HC slab elements are available today. Because each production system is patented, producers are usually set up on a franchise or license basis using the background, knowledge, and expertise provided with the machine development. Each producer then has the technical support of a large network of associated producers (PCI, 2015).

However, two basic manufacturing methods are currently used for the production of hollow core slabs. The first is the dry-cast or extrusion system where a low-slump concrete is forced through the casting machine. The cores are formed with augers or tubes, and compaction and vibration are used to consolidate the concrete around the cores. The second is the wet-cast system which uses a normal slump concrete.

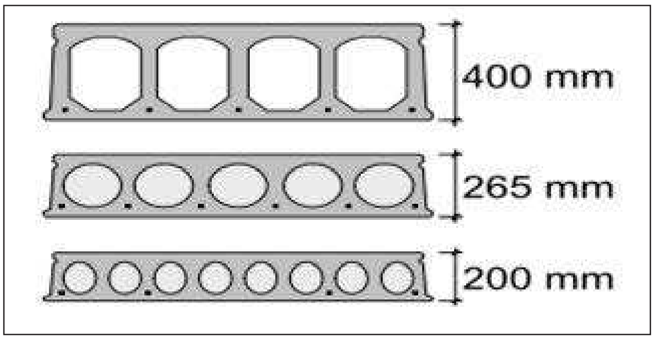


Fig. 1: Cross section of various HC elements (Fellinger, 2004)

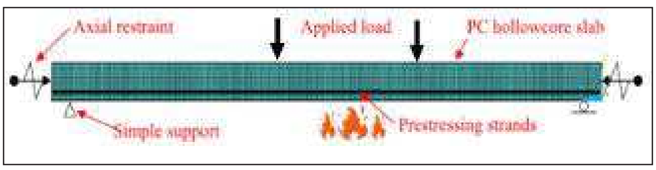


Fig. 2: Typical HC slab exposed to fire (Kodur and Shakya, 2017)

Fig. 3: Illustration for some of the steps of manufacturing HC slabs (Elematic, 2012)

In this system, the sides of the slabs are formed either with stationary, fixed forms, or with forms attached to the machine (when the sides are slip formed). The cores in the wet-cast systems are formed with either lightweight aggregate fed through tubes attached to the casting machine, pneumatic tubes anchored in a fixed form, or long tubes attached to the casting machine that slip form the cores (PCI, 2015).

In most cases, the hollow core slab elements are cast on long-line beds, normally 300 ft (~92 m) to 600 ft (~183 m) long (*Fig*. 3). After curing, the slab elements are sawcut to the appropriate length for the intended project (PCI, 2015).

2. INFLUENCE OF ELEVATED TEMPERATURES

2.1 Background

European and international building regulations require a minimum fire resistance for structures. Eurocode 1 (EN 1991-1-2:2002) defines fire resistance as: "The ability of a structure, a part of a structure or a member to fulfil its function (load bearing function and/or separating function) for a specified load level, for a specified fire exposure and for a specified period of time". The fire resistance is assessed either by testing (both large and small scales) or by calculation. Generally, an assessment of the entire structure under fire conditions is very complex and expensive experimentally although it is the likely method to obtain accurate data and results. Therefore, the fire resistance is mostly assessed on the basis of a member analysis, i.e. a single slab as shown in *Fig. 2* separated from the rest of the structure.

Remarkable fire resistance tests on HC slabs were performed by Abrams (1976), Borgogno (1997), Schepper and Anderson (2000), Acker (2003), Fellinger (2004), Jensen (2005), Bailey and Lennon (2008), Venanzi et al, (2014) and Kodur and Shakya (2014, 2017). These fire tests were performed under standard fire conditions and subjected also to service loads. Some of them present numerical studies and modeling as well.

Most of the aforementioned fire tests were performed under standard fire exposure solely to develop fire-resistance ratings for tested precast, prestressed concrete HC slabs. Based on these fire tests, some of possible factors for failure were identified such as spalling, bond slip of prestressing tendons and shear. However, the effect of factors on fire resistance is not fully quantified. Therefore, reasons due to failure mechanisms of HC slabs are not well established yet (Kodur and Shakya, 2014).

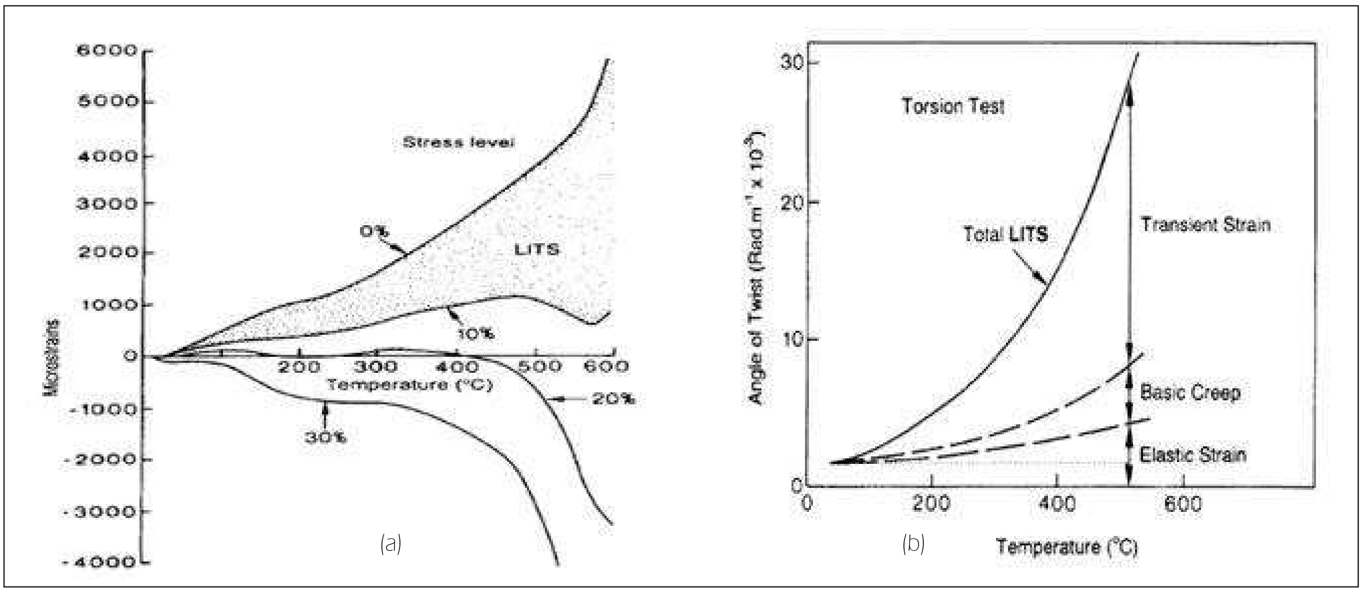
2.2 Theory of thermal stresses

As pointed out in the introduction, the shear behaviour of HC slabs at elevated temperatures can be dominant in structural design. Furthermore, the shear behaviour can only be assessed taking into account the effect of thermal strains.

The *Thermal Strains* is strictly the strain of non-drying concrete measured when concrete is heated without applied load. It does not contain drying shrinkage (*fib*, 2007). It is worth to mention that *Transient Creep,* called sometimes Transient Strain or Transient Thermal Strain, is often confused with *Load Induced Thermal Strain (LITS).* LITS is essentially measured by the difference between the strain measured during first heating without load and that measured during first heating under load. Whereas thermal creep is the largest component, in addition to basic creep and elastic strain, of the LITS. Fig. 4 (*fib*, 2007).

The thermal elongation of the exposed side of the slab will cause thermal stresses over the entire cross section of the HC slab. Thermal stresses result from mechanical strains that have to develop to counteract incompatible thermal strains in order

Fig. 4: Illustrations of thermal strain: (a) definition of the Load Induced Thermal Strain (LITS) and (b) LITS contains three main components (fib, 2007).



to meet the compatibility requirements. The thermal strains solely depend on the temperature rise and the coefficient of thermal expansion, which is a material's property. Due to the thermal expansion, axial forces and bending moments can develop by restraining boundaries. Even though HC slabs are in principle simply supported and the thermal expansion is not restrained, thermal stresses will develop within the cross section if the temperature distribution over the cross section is non-linearly distributed. The actual stress distribution has to satisfy the equilibrium conditions. Moreover, the actual constitutive behaviour of concrete and reinforcing steel shall be considered (*fib*, 2007).

The well-known theory of elasticity is no longer applicable due to the thermal strains and the highly non-linear stress-strain relationships for concrete and prestressing steel at elevated temperature (Fellinger, 2004). As a result, compressive thermal stresses will develop at the flange and tensile stresses at the web causes vertical cracks. Thus new state of equilibrium must be taken into consideration containing the developed normal force, the shear force and the bending moment.

3. CODE PROVISIONS

Eurocode 2 (EN 1991-2: 2004) presents requirements for fire design to determine the fire resistance of precast, prestressed concrete slabs. It provides three options i.e. tabular, simplified or advanced methods (*Table 1*). However, In case of hollow core slabs, additional rules are required (Jansze et al., 2012). These requirements were introduced in EN 1168 by the Eurocode which referred to the chapters 4.2, 4.3, and to Annex B in Eurocode (EN 1168, 2005). It is worth to mention that there are additional Annexes in Eurocode, D and E, for bending failure, and shear and anchorage failure (EN 1991-2: 2004).

Table 1: *Alternative methods of verifications of fire resistance Eurocode 2 [EN 1991-2: 2004]*

	Tabulated data	Simplified calculation methods	Advanced calculation methods
Member analysis The member is considered as isolated. Indirect fire actions are not considered	Yes	Yes	Yes
Analysis of part of the structure Indirect fire actions are considered, but no time-dependent interactions with other parts of the structure.	No	Yes	Yes
Global structural analysis Analysis of the entire structure is applied.	No	No	Yes

On the other hand, *Building Code Requirements for Structural Concrete* (ACI 318-11) have references to *Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies* (ACI 216.1-07). The later gives prescriptive specifications for evaluating fire resistance ratings of concrete and masonry structures based on ASTM E11924 standard fire tests. ACI provisions for determining the fire resistance of precast, prestressed concrete slabs are also similar to provisions in the PCI *Design Handbook and the International Building Code*. ACI also specifies minimum slab and concrete cover thicknesses to achieve a required fire-resistance. Further, both PCI *Design Handbook* and *Design for Fire Resistance of Precast/Prestressed Concrete* provide a rational design methodology for evaluating the fire resistance of precast, prestressed concrete slabs based on strength degradation of strand with temperature.

Other design codes, such as Australian code AS 3600, New Zealand concrete standard NZS 3101 *and the National Building Code of Canada* include provisions similar to those of *PCI Design Handbook* and ACI 216.1-07 (Kodur and Shakya, 2014).

4. FAILURE MODES

4.1 At ambient temperature

Unlike ordinary slabs which are usually predominant by flexural failure, shear failure mode has been observed in several HC slabs tests (Rahman et. al, 2012). In fact, many of researchers have found that nominal shear stress at failure start to develop even before shear strength calculated by some design codes (Angelakos et al, 2001; Lubell et al, 2003; Dwairi et al, 2005).

Therefore, a reduction in web-shear strength from the capacity predicted by (ACI 318-05) seems warranted for HC members deeper than 320 mm (Hawkins and Ghosh, 2006). With such a reduction, web shear will control the design of deep HC sections more often. Walraven and Mecx (1983) noticed four principal failure modes in a large project containing several tests:

a) **Pure flexural failure**
Ductility of a slab after flexural cracking is considerable since the cross section of the steel is relatively small, developing fork-shape and reducing the compression area, however, failure generally occurs as a consequence of rupture of the prestressing steel (*Fig. 5a*).

b) **Anchorage failure**
If the crack pattern extends towards the support the length of the anchored strand may be too small to develop sufficient capacity thus, the concrete element is susceptible to strand-slip through the concrete (*Fig. 5b*).

c) **Shear tension failure**
In case the tensile strength in the webs becomes too high in the uncracked region in bending, an inclined crack occurs resulting in immediate failure. This region is generally where the influence of the vertical support stresses is exhausted (*Fig. 5c*).

d) **Shear compression failure**
Flexural cracks can develop into shear cracks. By increasing the load, subsequent failure can occur in the compression zone, by crushing or splitting *(Fig. 5d*).

4.2 At elevated temperatures

The four failure modes presented by Walraven and Mecx are extensively studied by Fellinger (2004). He compared it

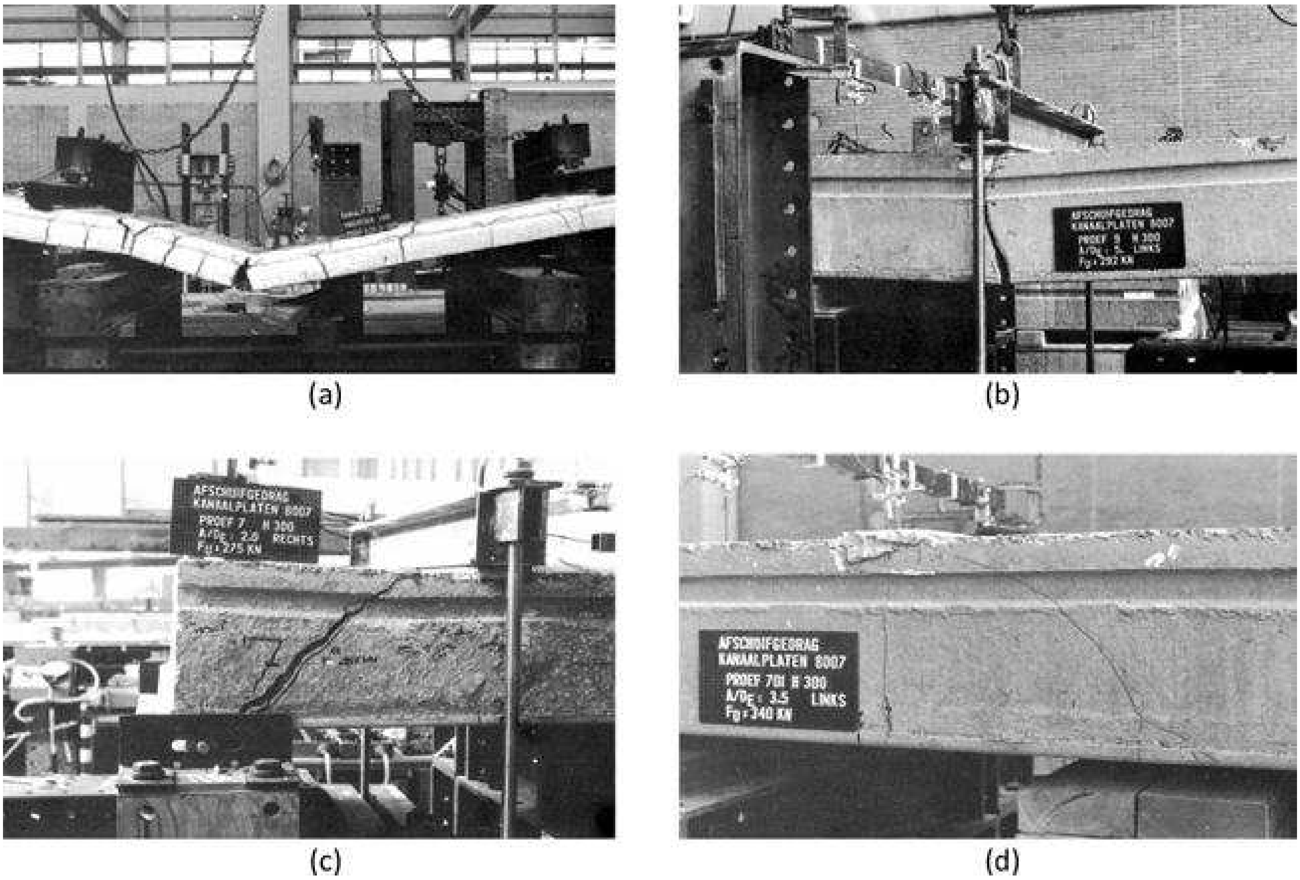


Fig. 5: *Different types of failure modes in HC slabs (Walraven and Mecx, 1983) (Room temperature)*

to his theoretical and experimental results. Concluding that the load bearing capacity of HC units on rigid supports can adequately be described by the theoretical formulations given before. He also proceeded to prove that all input parameters are considered in an appropriate manner for all four failure modes. Therefore, the theoretical formulations can be used in order to evaluate the load level in the fire tests found in literature. The results were as the following:

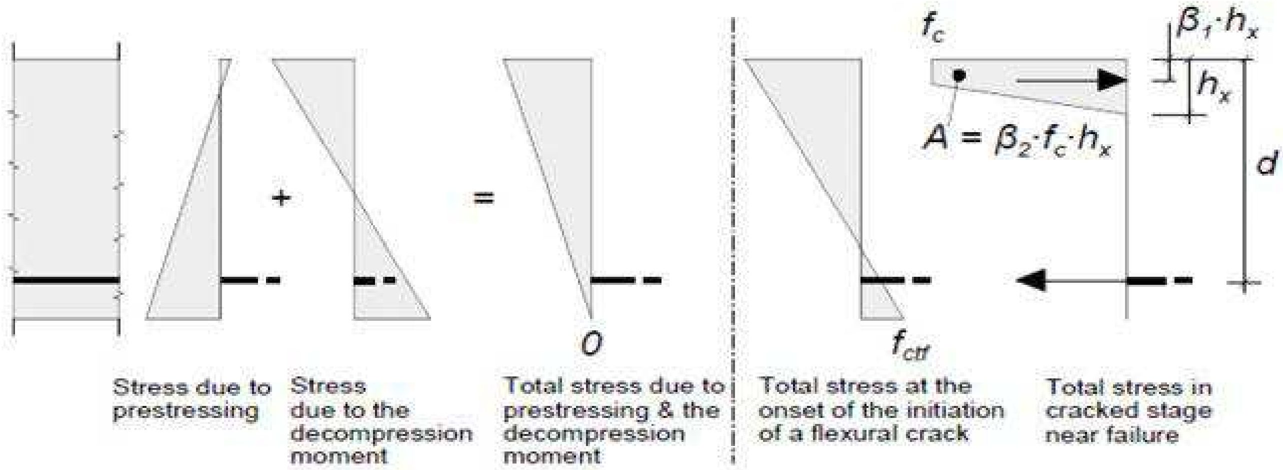
4.2.1 Flexure

Due to prestress, the strands are tensioned and the concrete is in compression. Bending tensile stresses reduce the compressive stresses at the bottom to reach the tensile strength of concrete. Then vertical cracks develop perpendicular to the tensile stress causing the so called flexural cracks from the bottom of the slab. The slabs have to be designed with sufficient strands such that the strands can undergo this stress increase without reaching the steel strength.

By increasing the load the tensile stress in the strand and the compressive stress in the concrete compression zone both increase. Since slabs are designed in such a way that the strand reaches the yield strength before the concrete compression zone crushes, rupture of the strands will occur after significant yielding and large deflection. (Fig. 6)

The bending moment capacity of a HC unit with *n* bottom strands is given on the basis of the theory of plasticity by

Fig. 6: *Linear elastic stress distribution over the height of the cross section due to prestressing and bending moments and non-linear stress distribution in the crack close to flexural failure (Fellinger, 2004)*



$$M_F = \sum_j^n z^j A_p^j f_p^j = \sum_j^n (h - \beta_1 h_x - c^j) A_p^j f_p^j \quad (1)$$

The height of the compressive zone h_x can be calculated on the basis of horizontal equilibrium as:

$$h_x = \frac{1}{\beta_2 b\, f_c} = \sum_j^n A_p^j f_p^j \quad (2)$$

In which f_p is the steel strength, A_p the cross sectional area of each bottom strand j, f_c the concrete compressive strength, b the width of the unit, h the slab depth, c the axis distance, i.e. the distance from the centroid of the strands to the bottom of the slab, and β_1 and β_2 shape factors for the concrete stress-strain relationship with ratio ranges from 1/2 to 2/3.

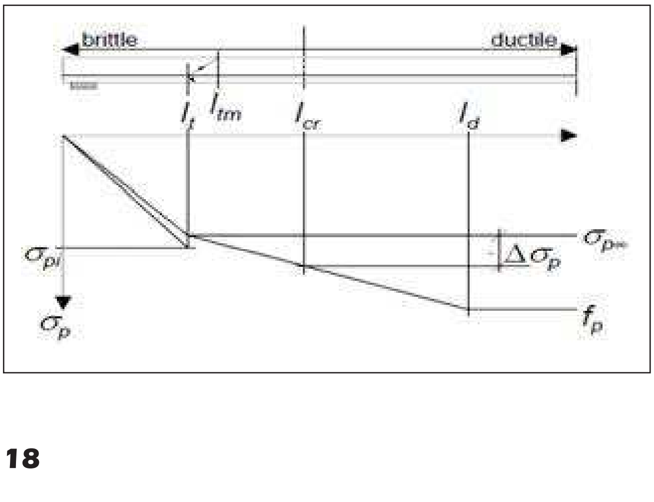
4.2.2 Anchorage

When a flexural crack appears, the tensile stresses in the concrete drop. Thus, the tensile force in the strand is locally increased to achieve equilibrium, see Fig. 5. The stress increment in the strand depends only on the concrete tensile strength and the ratio between the concrete area in tension that releases the stress and the cross section of the strands that takes over this stress. Bond stresses between the strand and the concrete will play significant role by building up the tensile force.

Anchorage failure can occur either in a brittle or in a ductile way. The transition between brittle and ductile anchorage failure is determined by the position of the flexural crack. If the flexural crack occurs within the so-called critical length from the slab end, the embedment length of the strand is insufficient to take over the stress released in the crack and cracking will cause brittle failure. If the flexural crack appears outside the critical length but within the so-called development length ductile anchorage failure occurs. Outside the development length, the embedment length is long enough to allow for full yielding of the strand. The maximum steel stress envelop is schematically presented in Fig. 7. The stress envelope is simplified to a tri-linear diagram defined by the transfer length and the development length thus, both should be calculated.

Brittle anchorage failure type occurs if the flexural crack is located close to the slab end. In that case, the steel stress increment due to crack cannot be developed in the strand. The strand is immediately pulled out. The anchorage capacity at this case equals the cracking moment resistance, which consists of the decompression moment (M_0) and a part causing tensile stresses in the bottom. The decompression moment is the moment that counteracts the prestress in which no axial stress remains in the bottom, see Fig. 5. It can be calculated from the linear elastic beam theory. So, anchorage capacity can be evaluated through the following equation:

Fig. 7: Stress envelope for the strand (Fellinger, 2004)



$$M_{cr}(x) = W_0 f_{ctf} + M_0\,(x) \quad (3)$$

In which W_0 is the section modulus of the lower half of the total cross section including the contribution of the steel strands. f_{ctf} is the flexural tensile strength of concrete which can be derived from the mean splitting tensile strength according to the Model Code [CEB-FIP: 1991].

On the other hand, ductile anchorage failure occurs when the initial stress increment due to cracking can be sustained by the strand, but further increase of the load causes pull out of the strand before it yields. The calculation of the ductile anchorage capacity is similar to that of the flexural capacity, refer to eq. 1, i.e. the bending moment equals the tensile force in the strands multiplied with the internal lever arm (z). By application of the theory of plasticity for anchorage failure based on bond-slip behaviour, the anchorage capacity then is:

$$M_A\,(x) = \sum_j^n z^j A_p^j \sigma_p^j\,(x) = \sum_j^n (h - \beta_1 h_x - c^j)\, A_p^j \sigma_p^j(x) \nless M_{cr}\,(x) \quad (4)$$

In order to calculate the anchorage capacity, the transfer length and the development length are given by the Model Code [CEB-FIP: 1991]

4.2.3 Shear compression

When a crack initiated in the area of bending moment and shear force it tend to be inclined crack. The shear force is then transmitted by aggregate interlock in the crack, the dowel action of the strand and by the uncracked compression zone. While the crack grows, the capacity of all contributions decreases. Since the deterioration mode starts with flexural cracks up to failure, it is sometimes also referred to as flexural shear failure.

Over the past 40 years, various models were developed for shear compression failure. Kani (1964) developed an analytical model that describes the shear compression behaviour of reinforced beams without shear reinforcement. A simply supported beam loaded by two equal point loads. Due to the flexural cracks, the beam transforms into a comb-teeth like structure. The compressive zone of the beam is the backbone of the comb (see Fig.8). The teeth of the structure are loaded as cantilever beams by the bond stresses. This loading causes the teeth to bend, leading to additional tensile stresses at the crack tip, which drives the crack to propagate.

Several of researches have been conducted based on the comb-teeth model introducing contribution parts i.e. aggregate interlock and dowel action. Hedman and Losberg (1978) derived a formula for shear compression based on statistical evaluation of shear tests. It was as:

$$V_{SC} = \gamma_{SC1} \sum b_w\, d\, k_s k_{ta} \left(1 + 50\, \frac{A_P}{\sum b_w d}\right) \sqrt{f_{cm}} + \gamma_{SC2} \frac{M_0}{a} \quad (5)$$

(f_{cm}) term is a measure for the tensile strength, therefore is expressed in MPa. The calibration factor γ_{SC1} is 0.104 and γ_{SC2} is 1.23 to predict the mean shear capacity and γ_{SC1} is 0.068 and γ_{SC2} is 1 to obtain a 95 % characteristic lower bound. M_0 is the decompression moment and a the distance between the point load and the support. k_s is the scale factor according to

$k_s = 1.6 - d \nless 1$

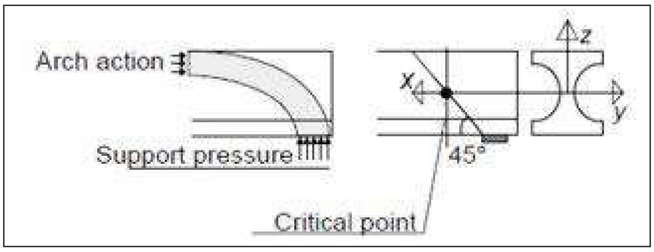


Fig. 9: Critical point, assumed just outside the zone strengthened by the introduction of the support reaction (Fellinger, 2004)

With d in meters and k_{ta} is the factor including increased shear resistance near the support.

$$k_{ta} = \frac{3d}{a} \not< 1$$

4.2.4 Shear tension

Shear tension failure starts with cracking near the support of the HC slab caused by shear stresses in the web. With no reinforcement to take over the tensile stress, shear tension capacity significantly decreases. The brittle behaviour propagates cracking immediately which cause brittle failure. The combination of shear stress and axial stress causes a principal tensile stress which has an inclination of approximately 45°. The maximum shear stress occurs at the support which has not fully developed of prestress at the slab end. Therefore the principal tensile stress reaches its maximum in the thinnest part of the web near the support. Nevertheless, the crack will not occur exactly at the support, because the stress distribution is disturbed near the support due to the vertical support pressure. So, the crack will be initiated in the web just outside the zone that is strengthened by the support pressure. In shear tension models, this starting point of cracking is called the critical point. (Fig. 9)
For very short shear spans, the slab can even act as a tied arch, which means that the support pressure is directly transferred to the point load through a concrete arch, tied by the strands.

Thus, shear tension capacity will be as:

$$V_{ST} = \frac{\sum b_w I_z}{S_z}\sqrt{\alpha_{cp}\frac{A_p}{A_c}\,\sigma_{p\infty} f_{ct} + f_{ct}^2} \qquad (6)$$

This formula is based on the assumption that shear capacity is to be reached if the principal stress equals the tensile strength ($\sigma_I = f_{ct}$). The principal stress can be calculated on basis of basic tensor algebra (I_1, I_2 and I_3). (α_{cp}) is the reduction factor for the prestress in the critical point, ranging between 0.15-0.25 for practical cases.(Σb_w) is the minimum web width.

From other side, ACI 318 (1995) uses another equation

$$V_{ST} = \gamma_{ST}\left(1.33\sqrt{f_c} + 0.3\,\alpha_{cp}\frac{A_P}{A_c}\,\sigma_{p\infty}\right)\sum b_w d \qquad (7)$$

In which f_c is the cylinder compressive strength in MPa and the safety factor for the model uncertainty of shear tension, equal to 0.85

5. FACTORS GOVERNING THE SHEAR RESPONSE OF HC UNDER FIRE

There are many of influencing parameters that affect the shear behavior of HC slabs, i.e. slab depth, load level, loading pattern, axial restraint, level of prestressing and fire scenario. At this review, these factors will be presented as follows: (based on Kodur and Shakya, 2017).

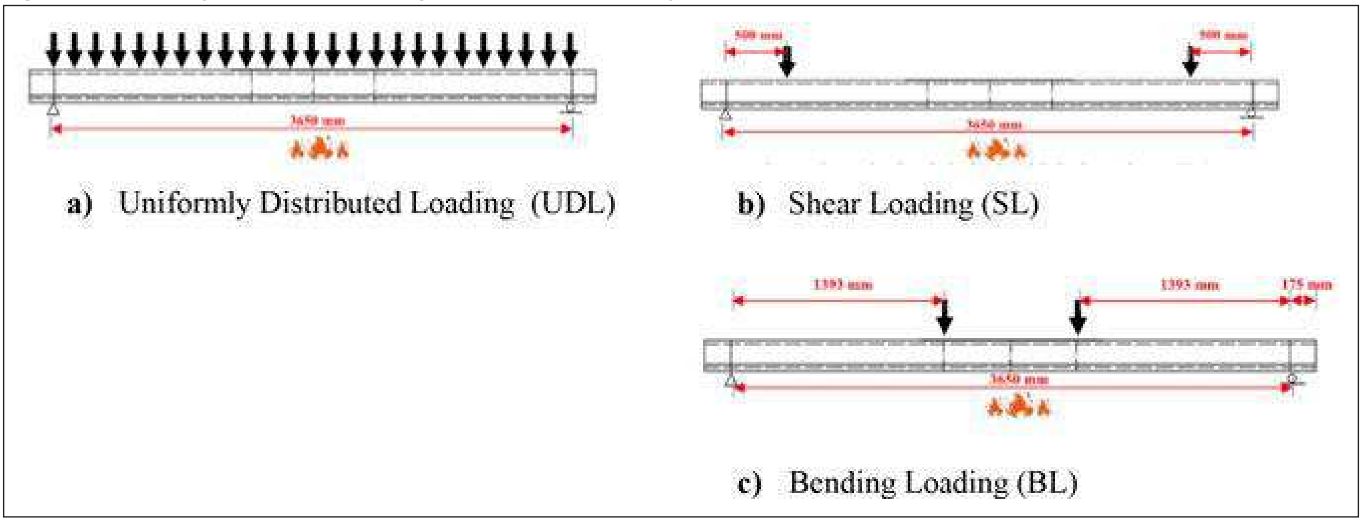
5.1 Effect of slab depth

Results showed that sectional temperatures typically decrease with increase in slab depth (higher thermal inertia). It was also noticed that lower sectional temperatures in thicker slabs lead to improve fire resistance in the slabs whereas thinner slabs undergo higher rate of deflection comparing to thicker slabs (higher sectional temperatures). The results infer that thicker slabs are more susceptible to shear failure as compared to thinner slabs, especially when the thickness exceeds 250 mm.

5.2 Effect of load level

To study the effect of load level on shear response, 200 mm and 400 mm slabs are analyzed under three different levels of loading, 50, 60 and 85% of room temperature for moment capacity (*see Fig. 2*). The results showed that higher load levels lead to higher deflections and lower fire resistance, indicating that load level has significant effect on the fire resistance of HC slabs. A review of crack patterns from ANSYS analysis results clearly show that 200 mm slab fail through flexural failure mode, while 400 mm slabs fail

Fig. 10: HC slabs subjected to various loading scenarios (Kodur and Shakya, 2017)



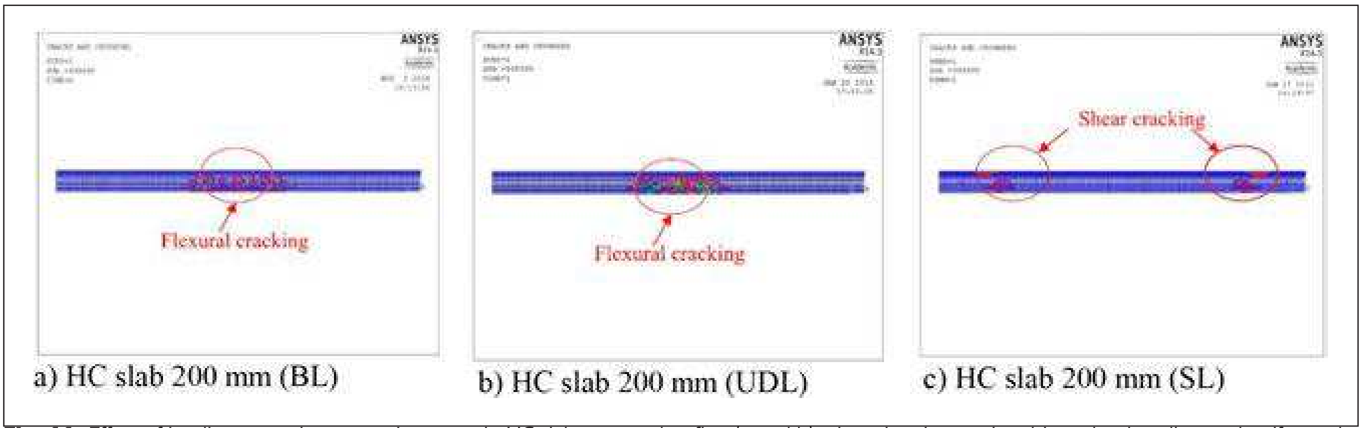


Fig. 11: Effect of loading scenarios on crack pattern in HC slabs exposed to fire: (a and b) when the element is subjected to bending and uniformed respectively whereas (c) when it is subjected to shear load (Kodur and Shakya, 2017)

through shear failure mode under all three cases of loading 50, 60 and 85% of the capacity. These results further infer that in thicker slabs, failure occurs through shear limit state under fire conditions.

5.3 Effect of loading scenarios

To study the effect of loading pattern, 200 mm and 400 mm thick HC slabs were analyzed under three different loading scenarios, concentrated bending loading (BL), distributed loading (UDL) and shear loading (SL) (*Fig. 10*).
Results infer that shear limit state can govern failure even in thinner HC slabs when subjected to high shear forces under fire conditions as shown in *Fig. 11*.

5.4 Effect of axial restraint

In order to study the effect of axial restraint on the behavior of HC slabs under fire conditions, 200 mm and 400 mm HC slabs were tested under 50 and 100% axial restraints.

The results show that presence of axial restraint enhances fire resistance of HC slabs through redistribution of stresses and in turn delaying the failure. Results also illustrate that slabs with 100% axial restraints undergo lower deflections and exhibit higher fire resistance, as compared to those with 50% axial restraint at supports. Similarly, slabs with 50% axial restraints undergo lower deflections and exhibit higher fire resistance than those without any axial restraints.

These results infer that presence of high axial restraint can enhance the fire resistance and also prevent shear failures in thicker slabs.

5.5 Effect of level of prestressing

To study the effect of level of prestressing on fire resistance, 200 mm and 400 mm HC slabs were analyzed under three different levels of prestressing, namely 50, 70 and 85%. The results infer that, although an increase in level of prestressing can lead to improved structural (moment and shear) performance at ambient conditions, the effectiveness of prestressing gets diminished at elevated temperature. This is attributed to the loss in strand strength which therefore lead to insignificant improvement in the overall fire resistance.

5.6 Effect of fire scenario

The results infer that fire intensity has significant effect on the fire response and failure modes of PC hollow core slabs, wherein a higher intensity fire results in lower fire resistance and can shift failure mode from shear to flexure in thicker HC slabs. This can be attributed to the fact that the flexural capacity of HC slab is mainly dependent on the strand temperature, and under higher fire intensity, temperature in strands increase at a much higher rate than at inner concrete layers, causing rapid degradation in moment capacity than shear capacity.

6. CONCLUSIONS

Fire accidents are often inevitable in buildings. This must be taken into account in design of structural members. Hollow core (HC) slabs are widely used in all Europe as well as around the world. Hollow core slabs have been studied in numerous experiments.

HC slabs have no reinforcement other than the longitudinal prestressing wires or strands, anchored by bond. Subsequently it uses 30% less concrete and 50% less steel. As a result, HC slabs possess lower shear capacity as compared to traditional solid slabs. Therefore, HC slabs, unlike solid slabs in which failure is predominantly governed by flexural capacity, are susceptible to shear failure at both ambient and elevated temperatures.

Four failure modes, at ambient temperatures were observed for hollow core slabs by Walraven and Mecx (1983) i.e. flexural failure, anchorage failure, shear tension and shear compression failures. At elevated temperatures, HC slabs are exposed to high levels of temperatures causing thermal influences. *Thermal Strain* is the largest component that essentially measured by the difference between the strain measured during first heating without load and that measured during first heating under load.

Fellinger (2004) studied the four failure mode equations presented by Walraven and Mecx (1983). He compared the equations to his theoretical and experimental results on HC slabs at elevated temperatures. Finalized formulas have been presented for all types of the failure.

On the other hand, Eurocode 2 presents requirements for fire design including requirements for mechanical resistance to maintain load bearing function during fire. It provides three options for determining the fire resistance prestressed precast structures, i.e. tabular, simplified or advanced methods. Additional rules have been provided in case of hollow core slabs. ACI, based on PCI, provides a rational design methodology for evaluating the fire resistance of precast, prestressed concrete slabs based on strength degradation of strand with temperature. ACI also specifies minimum slab and concrete cover thicknesses to achieve a required fire-resistance.

Finally, several parameters have been investigated in the literature. Some of critical parameters that affect the shear behavior of HC slabs are presented at this paper as; slab depth, load level, loading pattern, axial restraint, level of prestressing and fire scenario.

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