CHARACTERIZATION OF "CREEP-RUPTURE" FAILURE IN PRESTRESSED CONCRETE MEMBERS

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ABSTRACT

It is well known that concrete experiences time-dependent creep displacement when subjected to sustained compressive loads. This paper describes a so-called "creep-rupture" failure mode in which concrete creep behavior leads to yielding of non-prestressed steel reinforcement in heavily-prestressed slender members. This type of failure has been observed in DT members in which the prestressed force caused sustained loads and creep displacement of the DT stems. Non-prestressed steel bars in the stems acted to restrain the creep displacement; this restraining action led to yielding of the bars in compression. The concrete was unable to confine the yielded reinforcement and the stems ruptured. The first part of this paper qualitatively explains the mechanics associated with creep-rupture failure. The second part of this paper presents experimental strain data which validate the qualitative explanation.

Keywords: Long-term behavior, concrete creep, failure, double-tee, creep rupture

INTRODUCTION

When a reinforced concrete section is subjected to sustained compression loads, the concrete undergoes creep displacement causing a redistribution of concrete stress to the steel reinforcement. Concrete has been observed to fail when this redistribution results in steel yielding and the concrete is unable to confine the reinforcement. This failure mechanism has been referred to as “creep rupture” in tests of reinforced concrete columns[1]. The same phrase will be used in this paper to describe stress redistribution due to time-dependent concrete behavior in precast-prestressed members. Creep-rupture behavior has been described in columns with low steel reinforcement to concrete ratios[2] and is the primary reason for the minimum reinforcement ratio stipulated in ACI-318[1,3].

Reinforcement stresses resulting from time-dependent behavior in prestressed members have also been referred to as “decompression stress” (Mast 1998). However, this phrase will not be used in the current paper to avoid confusion with the concept of “decompression load” as it is commonly used in prestressed concrete analysis.

Creep-rupture behavior and failure has been observed in field-topped, lightweight concrete double tees containing supplemental non-prestressed reinforcement in the stems (Figures 1 and 2). This failure occurred several months after casting while the DT was in storage awaiting transport and erection. The bottom of the DT shortened due to the application of prestressing and due to time dependent concrete creep and shrinkage, resulting in compressive yielding of non-prestressed reinforcement in the stem. The concrete was unable to confine the yielded reinforcement and the concrete ruptured. As can be seen in Figure 1, the bar buckled outward during the failure due to the compressive stress and lack of confinement.

The failure shown in Figures 1 and 2 was the impetus of an experimental study on creep-rupture behavior. This paper is focused on the experimental study and has two objectives. First, the mechanics associated with creep-rupture failure will be qualitatively described. Factors contributing to creep-rupture will be identified and discussed. Second, experiential strain data from a DT member that was intentionally designed to demonstrate creep-rupture behavior will be presented and discussed. While this paper focuses on the primary mechanics of the failure in Figure 1 and 2, specific analysis of that member is outside the scope of what the authors are presenting in this paper.

Figure 1– Failure of double-tee stem due to creep-rupture

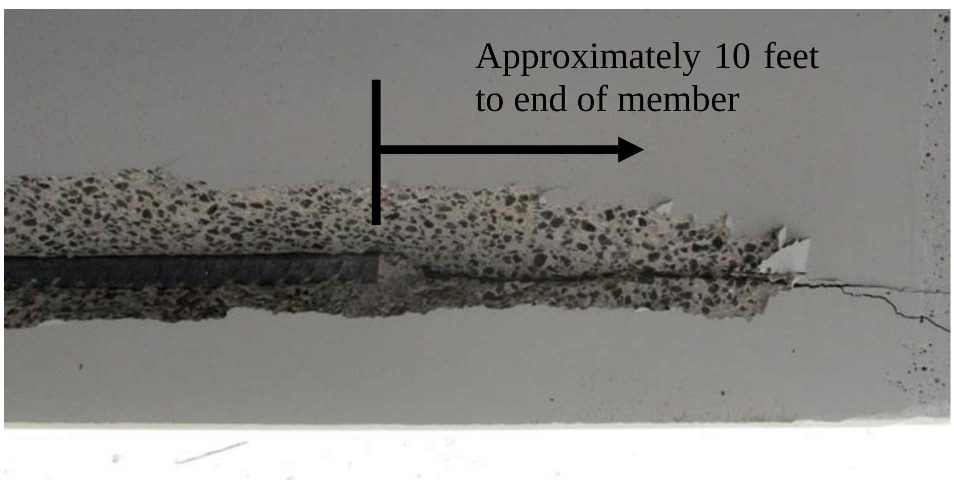

Figure 2– Termination of bar in failed double tee

DESCRIPTION OF CREEP-RUPTURE MECHANISM

Figure 3 diagrams the strains and stresses in a precast-prestressed member over time. The member in the figure is similar to the DT member shown in Figure 1. Specifically, an eccentric prestress force is applied near the bottom of the section and supplement non-prestressed reinforcement is placed below the prestressing strand.

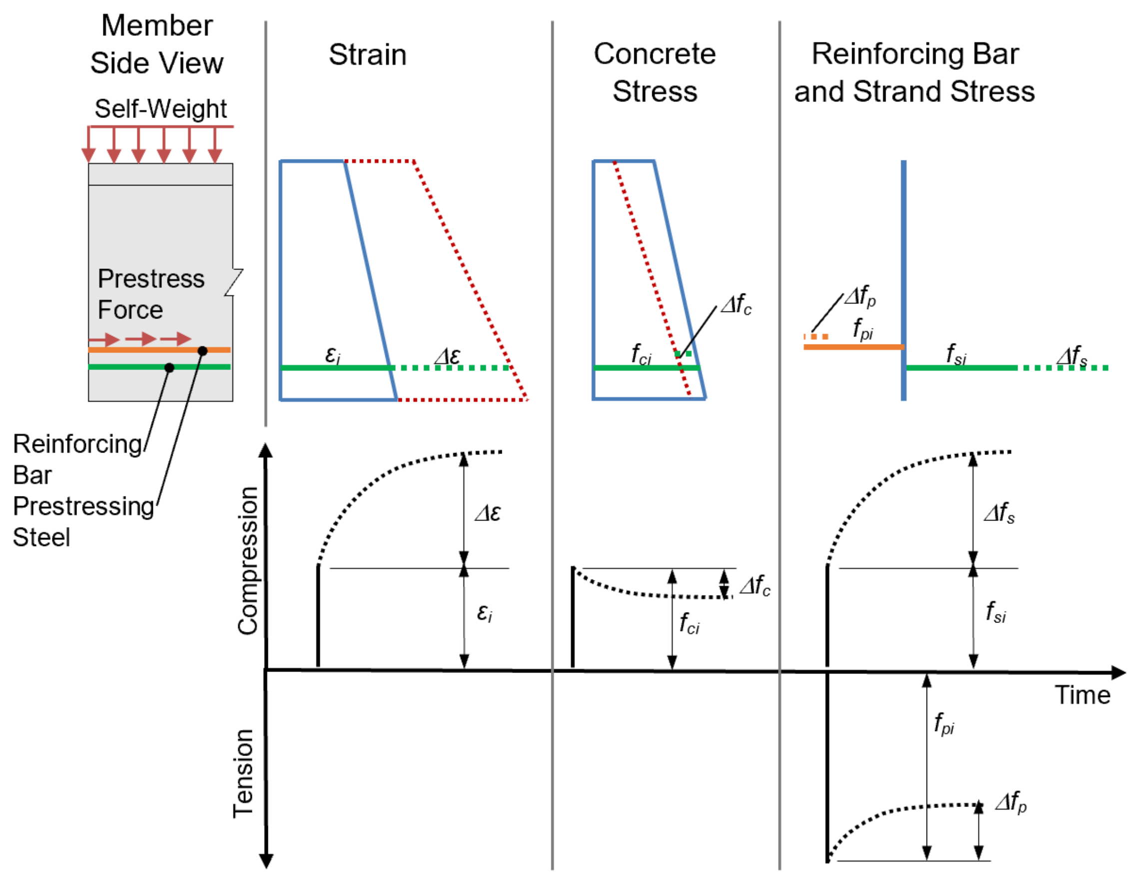

Figure 3– Time-dependent effects in prestressed member with non-prestressed reinforcement. ε = strain, f = stress, x_c = concrete, x_s = non-prestressing reinforcement, x_p = prestressing steel, x_i = initial condition, Δ= change over time

Stress and strain in the non-prestressed reinforcement near the bottom on the member are of particular interest. There is an initial compressive strain in the concrete and non-prestressed reinforcement (ε_i) as a result of the eccentric prestressing force. Over time the compressive strain increases ($\Delta\varepsilon$) due to time-dependent creep and shrinkage of the concrete. Creep and shrinkage (as well as relaxation and elastic shortening) lead to a loss of prestress (Δf_p), which in turn leads to a loss of concrete compressive stress (Δf_c). Because the non-prestressed reinforcement is bonded with the concrete it also undergoes the same change in strain ($\Delta\varepsilon$); the additional compressive strain results in additional compressive stress (Δf_s). Thus, even as stress in the concrete (Δf_c) and prestressing steel (Δf_p) decrease over time, stress in the non-prestressed reinforcement (Δf_s) increases.

In the case of the DT member shown in Figure 1, the summation of the initial and time-dependent stress ($f_{si} + \Delta f_s$) in the non-stressed reinforcement is likely to have reached yielding. Other factors such as lower than expected concrete strength, concrete elastic modulus, and reinforcement cover distance likely contributed to the failure. Due to this combination of circumstances the stem was unable to confine the non-prestressed reinforcement and the concrete ruptured. The next section will review these and other factors that can contribute to creep-rupture failure.

Mast (1998) states that time-dependent creep and shrinkage effects in non-prestressed flexural reinforcement can be calculated by the same approach used to evaluate time-dependent effects on prestressing strand. A detailed derivation and evaluation of stress and strain calculations for the creep-rupture mechanism will be made in a follow-up paper. A few of the equations are presented below to aid in description of the creep-rupture mechanism. These equations can be used to calculate stress in the non-prestressed reinforcement due to the combined effects of prestressing, self-weight, and time-dependent concrete behavior.

$$f_{si} = \left(\frac{P_i}{A} + \frac{P_i\, e\, y_s}{I} - \frac{M_{sw}\, y_s}{I}\right)\frac{E_s}{E_{ci}} \quad \text{Equation 1}$$

$$\Delta f_s = \Delta\varepsilon\, E_s \quad \text{Equation 2}$$

$$f_s = f_{si} + \Delta f_s \leq f_y \quad \text{Equation 3}$$

Where:

- f_{si}= Initial stress (after prestress release) in non-prestressed stem reinforcement
- P_i= Initial prestress force
- A= Cross-section area of member
- e= Eccentricity of prestressing
- y_s= Distance from cross-section centroid to non-prestressed reinforcement
- I= Cross-section moment of inertia
- M_{sw}= Moment due to self-weight of member
- E_s= Elastic modulus of non-prestressed reinforcement
- E_c= Initial elastic modulus of concrete
- Δf_s= Time-dependent change in stress in non-prestressed reinforcement
- $\Delta\varepsilon$= Time-dependent change in strain in non-prestressed reinforcement
- f_s= Stress in non-prestressed reinforcement after time-dependent effects
- f_y= Yield stress of non-prestressed reinforcement

FACTORS CONTRIBUTING TO CREEP-RUPTURE

Factors contributing to creep-rupture failure can be gleaned from observations of previous failures (Figure 1), the mechanics of creep-rupture behavior (Figure 3), and the mathematical description of non-prestressed reinforcement stresses (Equations 1-3). These factors are listed below:

- *Use of non-prestressed reinforcement*. The presence of non-prestressed reinforcement within the pre-compressed region of a member is essential to the creep-rupture failure mechanism described in this paper. Thus, one simple way of mitigating creep-rupture failure is to create designs that do not require supplement non-prestressed flexure reinforcement.
- *Placement of non-prestressed reinforcement*. If non-prestressed flexure reinforcement is required then it should be placed with sufficient cover. While the necessity of cover is obvious, the lack of sufficient cover was a contributing factor to the failure shown in Figure 1. Cut-off locations are another important issue. To prevent abrupt changes in stiffness and creep-resistance it is recommend that non-prestressed bars be extended to the ends of the member even if this isn't required for flexural capacity. Extending the bar to the end will not eliminate the formation of compressive stresses in the bar but it will alleviate stress risers due to an abrupt change in section properties.
- *Concrete elastic modulus*. Stiffness of the concrete is inversely proportional to the initial stress in the non-prestressed reinforcement (Equation 1). Stiffer concrete will affect lower stresses in non-prestressed reinforcement due to both elastic and inelastic strain in the section. Modulus of elasticity develops similarly to concrete strength over time. A low specified release strength will result in a lower modulus of elasticity and affect greater elastic shortening and higher initial strain in the non-prestressed reinforcement.
- *Concrete unit-weight*. The DT shown in Figure 1 utilized light-weight aggregate. Reducing the weight of a concrete member affects stress in the non-prestressed reinforcement through multiple mechanisms. First, the moment due to self-weight acts against the effects of the prestressed force; this directly reduces the stress in the non-prestressed reinforcement (Equation 1). Second, moment due to self-weight reduces the sustained compressive stress in the concrete, thus reducing the degree of time-dependent concrete creep. The beneficial effects of concrete self-weight are smallest at the ends where the self-weight moment is low and highest at mid-span where self-weight moment is maximum. Third, reduced concrete unit-weight tends to decrease elastic modulus (ACI 318 19.2.2.1a) the effects of which were mentioned in the previous point.
- *Concrete strength*. The final stage of creep-rupture failure is rupturing of the concrete around the non-prestressed reinforcement. It is reasoned that concrete with higher strength, particularly higher tensile strength, will have greater ability to confine non-prestressed reinforcement. In addition to directly increasing resistance to rupture, higher concrete strength also relates to higher elastic modulus thereby indirectly reducing stress in the non-prestressed reinforcement.

- *Concrete creep properties*. Mixes that are more prone to creep will result in larger time-dependent strain (Equation 2) in non-prestressed bars and increase the likelihood that the reinforcement will yield in compression.
- *Prestressing and cross-section*. Members with relatively large eccentric prestressing force and/or small cross-section properties will have larger stress in non-prestressed reinforcement (Equation 1). Additionally, because concrete creep is related to the level of compressive stress such members will also have greater changes in strain overtime.
- *Confinement of stem*. The presence of stirrups or weld-wire mesh the stem will provide confinement to the stem in the vertical direction. This confinement will add to the stem's ability to resist rupture. The DT shown in Figure 1 did not have vertical reinforcement at the sections where the stem ruptured.
- *Time between prestress release and placement of topping slab*. As mentioned previously, the creep-rupture failure shown in Figure 1 occurred months after casting while the member was in storage. The time in storage allowed the effects of creep and shrinkage to materialize and to cause the failure. Erection and placing of the field topping as soon as possible after prestress release will reduce the compressive stress in the stem thereby mitigating concrete creep and the formation of additional compressive stress in the non-prestressed reinforcement.

While light-weight concrete can contribute to creep-rupture failure, it is acknowledged that light-weight concrete has a long and successful history in the precast industry[4]. Research has shown that well-designed light-weight concrete mixes can have excellent performance in precast-prestressed applications[5]. The failure shown in Figures 1 and 2 is believed to have been a "perfect storm" in which concrete self-weight was one of many factors that led to failure. As will be shown in the following experimental study, the use of reduced-weight concrete does not necessarily lead to creep-rupture failure even when a member is intentionally designed to demonstrate creep-rupture behavior.

EXPERIMENTAL STUDY

Two 28 in. [710 mm] deep by 7 ft [2 m] wide double tees (7DT28) were cast to study creep-rupture behavior and to measure strain in the stem reinforcement. The members were 40 ft [12 m] and 60 ft [18 m] in length and yielded consistent results. This paper will focus on the results from the 60 ft [18 m] member for simplicity. While the members were intentionally designed to exhibit creep rupture issues, the design is not unrealistic for a narrow double tee that was cut back for spacing purposes and cast with members requiring a higher level of prestressing.

Fourteen 9/16th diameter 7-wire low relaxation strands with a total initial prestressing force of 508 kip [$2.26x10^3$ kN] were used (Figure 4). The concrete mix was designed with a compressive strength of 6000 psi [41 MPa] 28-day strength, 4800 psi [33 MPa] release strength, and 120 pcf [1922 kg/m^3] unit weight. Two #8 non-prestressed steel reinforcement bars were located 3 in. [76 mm] above the bottom of each stem with a total length of 40 ft [12 m] centered on the 60 ft [18 m] span. Mill certifications indicate that the reinforcing bar had a yield strength of 72 ksi [496 MPa] and a modulus of elasticity of 29000 ksi [$2x10^5$ MPa]. Prestress release was accomplished using flame cutting and stripping and transportation was done using standard lifting loops placed 3 ft [1 m] from the ends of each member. The members were released and then transferred to dunnage for outside storage. The members rested on dunnage 3 ft [1 m] from each end for approximately 12 months while strains were monitored.

Vibrating wire strain gages were imbedded in the concrete near the reinforcing bar as shown in Figure 4 & Figure 5. Gages in the 40 ft [12 m] member were similarly placed to those indicated in the 60 ft [18 m] member.

The specimens were cast on March 15, 2016 (Figure 6). Strain readings were taken prior to release and then monitored through the stripping and storage process. Readings were taken daily for the first week, monitored weekly until the 6th month and biweekly until the experiment ended at 12 months.

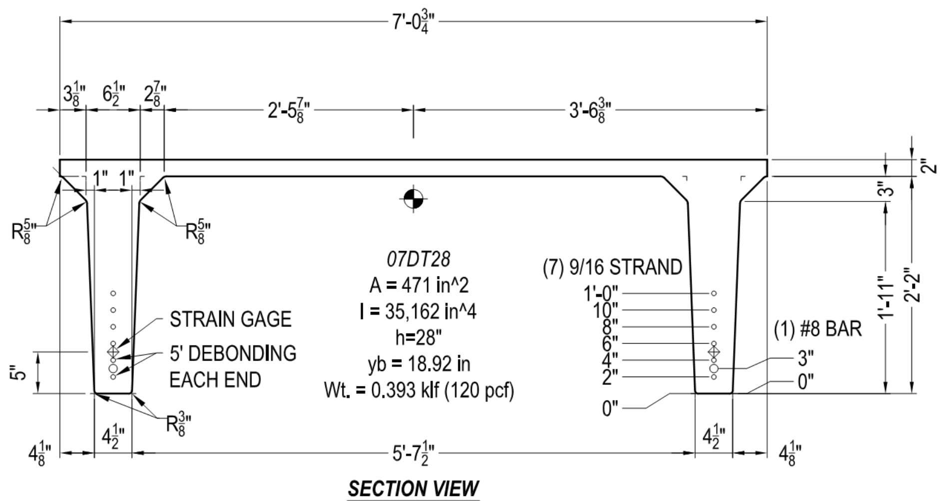

Figure 4– Test specimen cross-section

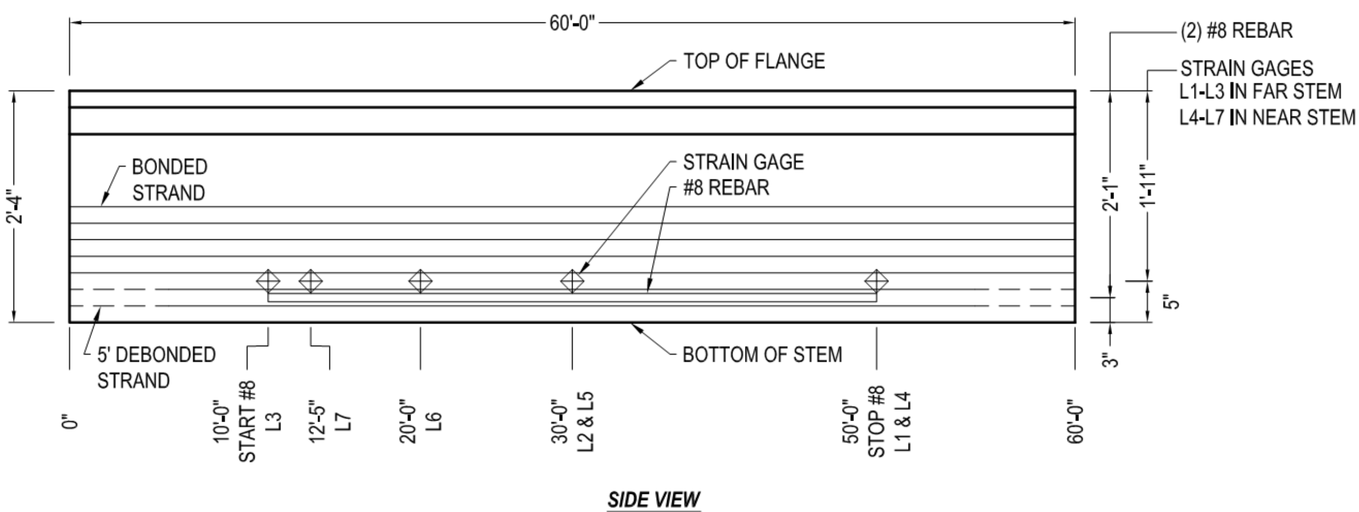

Figure 5– Test specimen side view

Figure 6– 40ft and 60ft test specimens on dunnage for storage

RESULTS AND DISCUSSION

Results from both members and redundant gages were consistent. A single set of gages from the 60 ft [18 m] member is presented in the following plots to simplify the discussion. Gages L4, L5, and L6 were chosen due to their positions along the length of the member and because they were in the same stem. Strain measurements were higher due to a reduced self-weight moment in the shorter 40 ft [12 m] member but did not include a gage at the 1/3 span location.

Figure 7 shows the measured strain over time in the test member at the $1/6^{th}$, $1/3^{rd}$, and 1/2 span locations. Strain readings were zeroed immediately prior to release. Strain increased in the member over time due to creep and shrinkage effects. The beam was simply supported resulting in a self-weight moment diagram with a peak positive moment at midspan and negative moment over the supports. Prestressing and self-weight resulted in peak bottom cord compression stress near the supports and lowest bottom cord compression stress at midspan. The change in sustained compression stress along the length resulted in creep strains varying along the length of the member. This can be observed from Figure 7, wherein the strain recorded at midspan (L5) was the lowest and strain recorded nearest the end (L4) was highest.

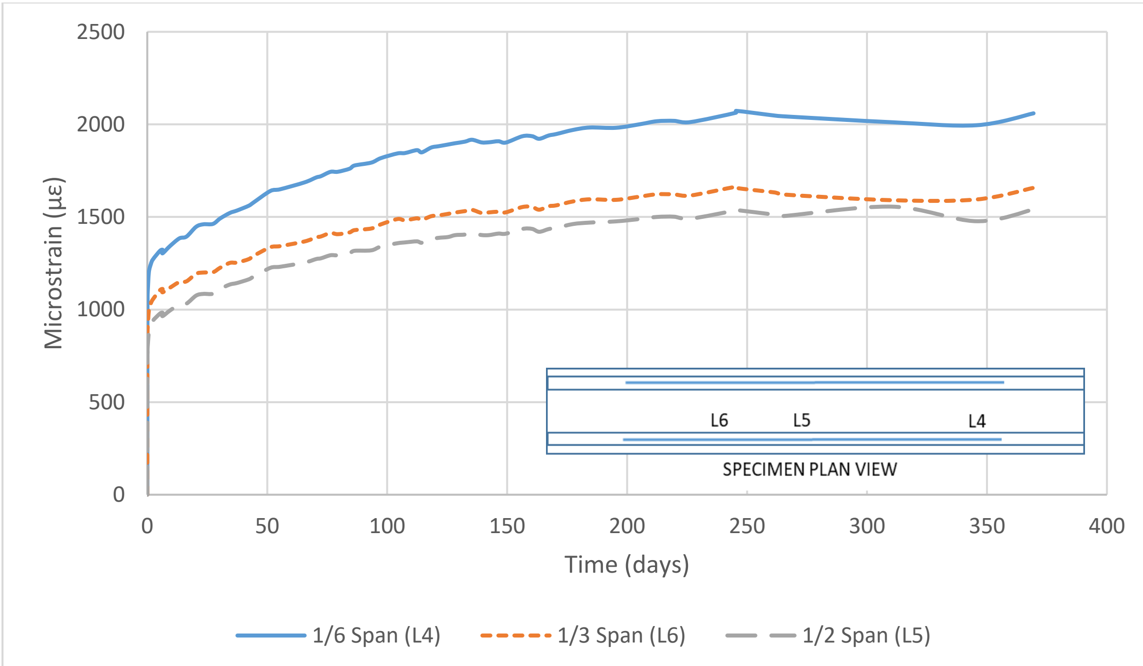

Figure 7– Strain in the beam at gage locations over time

Figure 8 shows the change in prestress in the member over time. Prestress was calculated from the measured strain data which were were adjusted from the gage location to the centroid of prestressing. PCI/ACI Simplified Method for calculating prestress losses was used to estimate the losses and determine the calculate effective prestressing shown in the figure. This method estimates higher losses than the AASHTO refined method (not shown in figure) and both methods are usually expected to overestimate total losses[6]. The losses predicted by the PCI/ACI Simplified Method were within 10% of the losses calculated from the strain data.

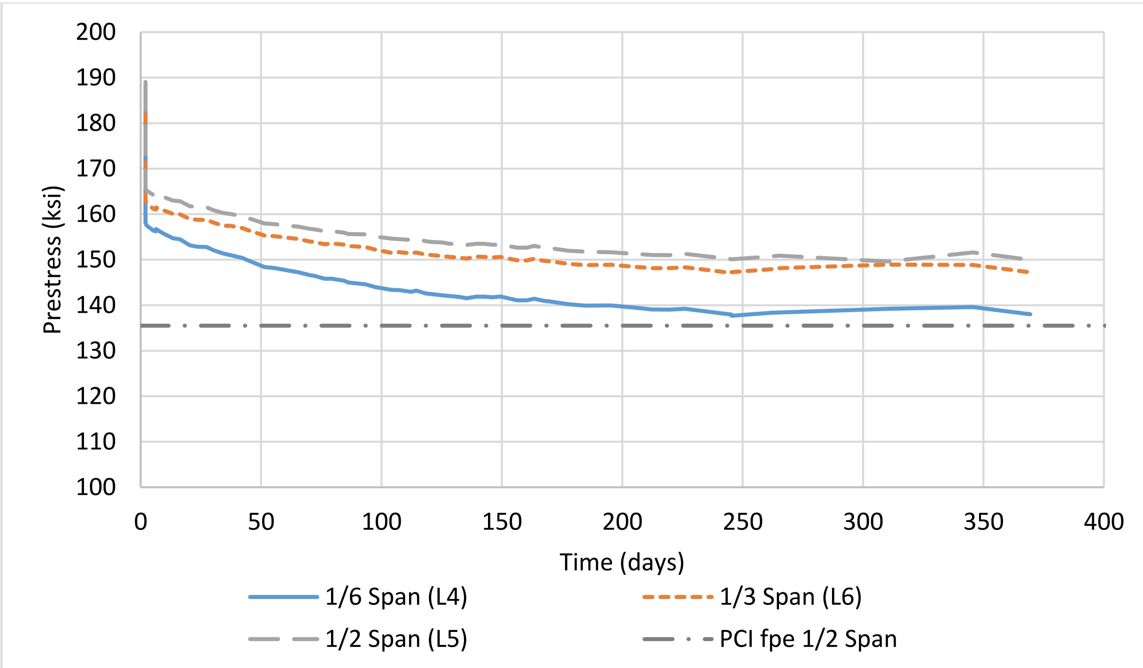

Figure 8– Average prestressing stress in the member over time. PCI effective prestress is calculated at mid-span.

Strains increased during the duration of the experiment with peak strains exceeding the yield strain of non-prestressed reinforcement in both members (Figure 9). The maximum strain 0.00253 (2525 με) was recorded in the 40 ft [12 m] member at 10ft from the end of the member. This corresponds to an estimated 0.00275 (2748 με) at the bar. Strains were seen to exceed the realistic yield strain of the reinforcement, 0.00248 (2483 με), corresponding to the yield stress of 72 ksi [496 MPa] based on the mill certifications provided with the ASTM A615 Gr 60 reinforcing bar. The maximum estimated strain at the bar in the 60 ft [18 m] member was 0.0024 (2401 με).

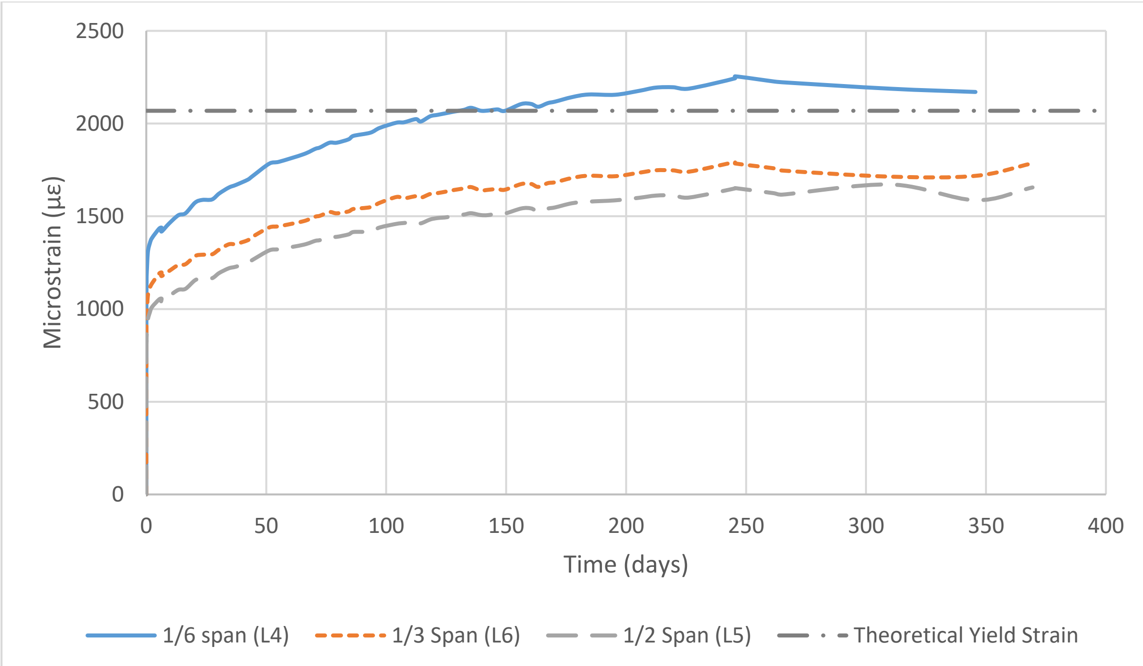

Figure 9– Strain at the non-prestressed reinforcement over time. Theoretical yield strain is calculated for a yield strength of 60 ksi in mild steel.

Figure 10 illustrates the strain changes along the length of the member. Strains were higher at the end of the member due to a reduction in section stiffness and lower bending moment stress. The prestressing was fully transferred to the section at location of gage L4, however, the non-prestressed reinforcement was not developed and some prestressing strand were not fully developed due to debonding. This coupled with a reduction in bending moment stress and the corresponding increase in creep strains causes the peak strain to be at the termination of the rebar in the test specimens.

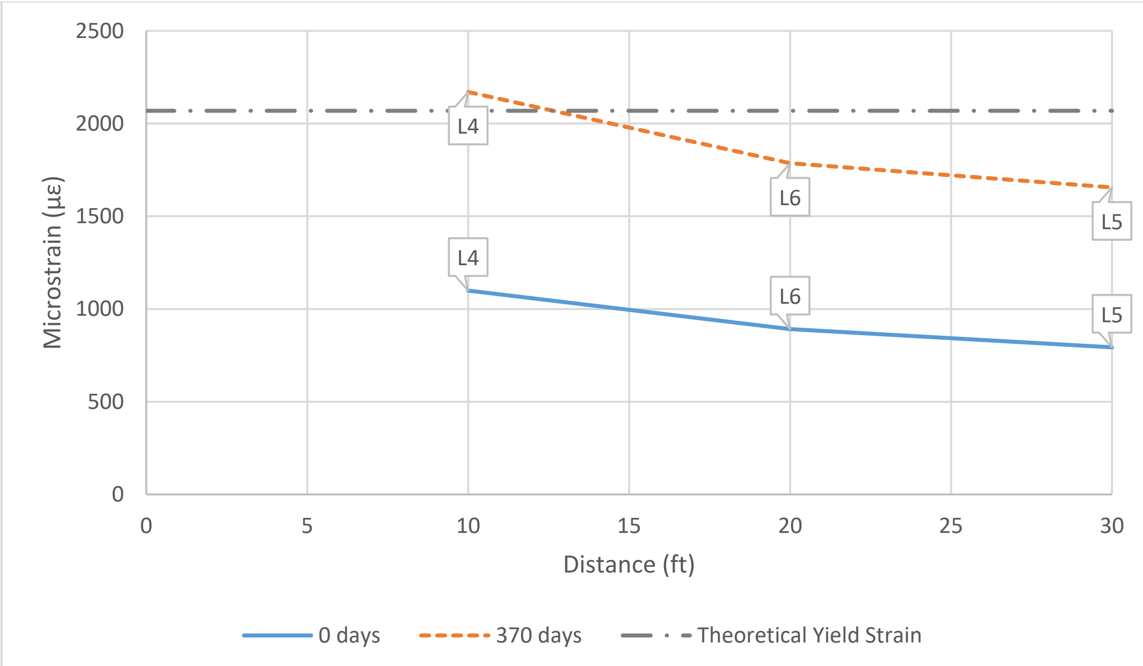

Figure 10– Strain at the non-prestressed reinforcement over the length of the member. Strain at Day 0 was taken immediately after release. Readings from gages L4, L5, and L6 are shown above.

While strains were seen to exceed the expected yield strain of the reinforcement, no creep-rupture failure was observed in either specimen. The strain data confirms the behavior described earlier in this paper, specifically that compressive strain increases in non-prestressed reinforcement due to time-dependent concrete behavior. The lack of failure is an encouraging result. In these specimens, the strain was near to or higher than the specified yield strain. Thus the occurrence of yielding of non-prestressed steel bars does not automatically lead to creep-rupture failure. The difference in result between the experimental specimen and the DT shown is Figure 1 is attributed to a combination of other factors such as lower cover, lower member self-weight, lower concrete tensile strength and lower concrete release strength.

SUMMARY AND CONCLUSIONS

This paper describes "creep-rupture" failure which has been observed in precast double-tee members. A qualitative description of creep-rupture mechanisms was provided and contributing factors were listed and briefly discussed. The behavior was experimentally demonstrated using a DT specimen which was intentionally designed to produce large compressive strains in non-prestressed stem reinforcement. Highlights and conclusions are as follows:

- Long-term creep of concrete in compressed regions of precast members can lead to "creep-rupture" failure. For this type of failure to occur the member must have non-prestressed reinforcement in the compressed region. Such reinforcement resists concrete creep and in doing so develops compressive stress. If the reinforcement stress is sufficient and if the surrounding concrete cannot restrain the bar(s), then the concrete will rupture.

- Care should be taken in circumstances where creep-rupture is likely. The section "Factors Contributing to Creep-Rupture" will guide engineers in identifying potentially problematic situations.
- While a creep-rupture failure did not occur in the experiment, strain data confirm that that the mechanisms contributing to creep-rupture were acting in the specimen.
- Based on experimental strain data the compressive strain in the non-prestressed stem reinforcement was up to 0.00253 (2525 με). Thus creep-rupture failure will not necessarily occur due to yielding of the reinforcement.

A simple way to prevent creep-rupture failure is to avoid the use of non-prestressed reinforcement in cases that may be affected by the creep-rupture mechanism. In lieu of placing non-prestressed reinforcement in a design requiring greater section properties, an alternative design could utilize additional prestressing strand pretensioned to a lower stress. This could maintain the same level of prestressing while changing the section properties as desired.

A follow-up paper on this topic is in progress. The authors' intention for the subsequent paper is to develop a numerical model to estimate the strain in the non-prestressed reinforcement and validate the model with the test data. This model will be used to investigate the impact of some design decisions and provide designers with a method of estimating strains induced by the creep-rupture mechanism.

ACKNOWLEDGEMENTS

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