EFFECTS OF INCREASING THE ALLOWABLE COMPRESSIVE RELEASE STRESS OF PRETENSIONED GIRDERS

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ABSTRACT

Over the last decade, an increase of the current allowable concrete stress in compression at prestress transfer has gained considerable support within the precast/prestressed concrete industry. To investigate the implications of increasing this compressive stress on the live-load performance of pretensioned members, a research study, conducted at the University of Texas and funded by the Texas Department of Transportation, was initiated. Static testing was performed on 24 laboratory-scale specimens that were subjected to compressive stresses at release ranging from $0.46f'_{ci}$ *to* $0.91f'_{ci}$*. The results of this research indicated that exposing concrete at early ages to high levels of stress damages the microstructure of the concrete. In addition, the use of typical design procedures based on linear-elastic principles did not produce accurate predictions for the live-load behavior of overstressed beams. The predicted cracking loads for beams subjected to a maximum stress at release in excess of* $0.70f'_{ci}$ *exceeded the corresponding measured cracking loads by up to 20-percent. As a result, the findings of this experimental study suggest that an increase in the allowable stress to a value higher than* $0.70f'_{ci}$ *is not acceptable. However, an increase to a value between* $0.60f'_{ci}$ *and* $0.70f'_{ci}$ *could potentially be justified with future testing.*

Keywords: Prestressed concrete, prestress transfer, allowable release stresses

INTRODUCTION

When design provisions for prestressed concrete were formally adopted in the AASHTO Standard Specifications and in the ACI Building code in 1961 and 1963, respectively, the allowable concrete stress in compression at prestress transfer was set at $0.60f'_{ci}$.[1,2] This same stress limit exists in the current editions of these codes. Over the last decade, an increasing amount of attention, in practice and in research, has been given to increasing this allowable stress. The primary motivation for this interest is the economic benefits of a higher allowable release stress. Some of these benefits include:

- the increase in span capabilities due to an increased number of prestressing strands in a given section
- the reduction in production time of precast facilities
- the reduction of external curing costs
- the reduction of the overall cement content
- the reduction of debonded or harped strands
- the negation of increased production time from using low-alkali cement or substituting cement for other cementitious materials

With this motivation, several research studies have been initiated since 1996. In each of these projects, an effect of increasing the allowable release stress in compression was investigated. By and large, the results of each of these studies supported the increase of the allowable release stress. However, the live-load performance of overstressed beams was not evaluated in any of these studies. As a result, a research project was initiated at the University of Texas at Austin funded by the Texas Department of Transportation. In this study, TxDOT Project 5197, 24 laboratory-scaled beams were tested to determine the impact of increasing the allowable release stress in compression on the performance of the member in flexure. This study and its results are discussed in this paper.

In the following discussion, the literature survey, the experimental program, and the results of TxDOT Project 5197 are presented. The literature review covers the history of the allowable release stress in compression and the recent research studies associated with this limit. In addition, the effect of loading early-age concrete to high levels of compressive stress on the internal microstructure of concrete was addressed. In the subsequent sections of this paper, the static load tests of 24 scaled beams are discussed. The beams consisted of rectangular, tee, and inverted-tee girders subjected to compressive stresses at release ranging from $0.46f'_{ci}$ to $0.91f'_{ci}$. For each static load test, the measured cracking load was compared to the cracking loads predicted according to the PCI Design Handbook[3] and the AASHTO LRFD Specifications.[4] The accuracy of these estimates evaluated the impact of increasing the allowable stress limit on the live-load performance of pretensioned members.

LITERATURE REVIEW

HISTORY OF ALLOWABLE RELEASE STRESS IN COMPRESSION

In the early 1960s, $0.60f'_{ci}$ was formally adopted as the compressive stress limit at release into the respective building and bridge specifications. This stress limit originated from the recommendations of the following two documents: "Criteria for Prestressed Concrete Bridges"[5] published by the Bureau of Public Roads and "Tentative Recommendations for Prestressed Concrete"[6] released by ACI-ASCE Committee 323 (later Committee 423). In both of these documents, the origin of $0.60f'_{ci}$ is alluded to. In the closing remarks of the published discussion of the "Tentative Recommendations for Prestressed Concrete," the following excerpt exists:

> *"Here, production had preceded design recommendations, and the stress of $0.60f'_{ci}$ had already been widely established in the pretensioning industry. No ill effect had been reported in regard to strength and performance. Only camber proved difficult to control for certain building members."*[7]

In a paper published in the Proceedings of the World Conference on Prestressed Concrete in 1957, Erickson referred to the disagreement of several researchers in regards to the allowable stress limit.[8] A summary of the release stress recommended by several researches is provided in Table 1. While the original limit was most clearly adopted based on its successful use in practice, it is clear that the limit is not some arbitrary value. The success of this allowable stress limit is based on the proper response of pretensioned members.

Table 1: Summary of release stresses from several sources[8]

Suggested By	Initial Stress	Condition
Hajnal-Konyi (England)	$0.45f'_{ci}$	---
Dobell (Preload Co.)	$0.50f'_{ci}$	---
Holley (MIT)	$0.60f'_{ci} – 0.40f'_{ci}$	$0.60f'_{ci}$ only if reduced to $0.40f'_{ci}$
Simpson (MIT)	$0.60f'_{ci} – 0.50f'_{ci}$	$0.60f'_{ci}$ only if reduced to $0.50f'_{ci}$
Siess (U. of Illinois)	$< 0.60f'_{ci}$	---
Bureau of Public Roads Criteria	$0.60f'_{ci}$	pretensioning
	$0.55f'_{ci}$	post-tensioning

RECENT RESEARCH AND DISCUSSION REGARDING $0.60f'_{ci}$

In 1997, Pang et al. investigated the feasibility of increasing the allowable release stress in compression by conducting several series of cylinder tests.[9] In the study, cylinders were loaded in compression to 60-, 70-, and 80-percent of their strength at the time of loading for a specified amount of time. After which, the cylinders were unloaded and tested in compression. Cylinders that were not loaded, or "control" cylinders, were tested with the loaded cylinders. According to the authors, the difference in compressive strength between the "control" cylinders and the loaded cylinders evaluated the impact of increasing the allowable stress at prestress transfer.[9]

For the cylinders loaded after 1-day of curing and to 60- and 70-percent of f'_{ci}, a reduction in compressive strength was not observed. The cylinders loaded to these stress levels failed in compression at the same load as the "control" cylinders. For the cylinders loaded to 80-percent of their strength, two specimens failed prematurely due to the sustained stress. It is interesting to note that for the cylinders loaded to $0.80f'_{ci}$ that did not fail prematurely, a reduction in compressive strength was not detected. The results of these tests seem to suggest that compressive strength is not a proper indicator of damage. At the conclusion of this study, the authors stated that an increase to $0.70f'_{ci}$ for the allowable release stress is a possibility.[9]

In 1997, Huo and Tadros performed an analytical study to illustrate the behavior of prestressed concrete members subjected to release stresses greater than $0.60f'_{ci}$.[10] The study consisted of performing a linear and a nonlinear analysis of an 18-inch by 18-inch, concentrically prestressed concrete section. Essentially, the number of strands was progressively increased for each approach until each analysis suggested that the section had failed.

The results illustrated the "self-relieving mechanism" of prestressed concrete.[10] According to the linear analysis, only 45 strands were required to fail the section. On the contrary, the nonlinear analysis suggested that 62 strands would be required to crush the concrete. The difference between these two approaches lied solely in the assumed stress-strain relationship for the concrete. In the linear case, the concrete was elastic until failure. In the nonlinear case, the concrete reached higher strain levels as the load increased beyond the elastic range. As a result, more strands were required to achieve the strain that corresponded to the ultimate stress in the nonlinear analysis. The authors compared this procedure to a displacement-controlled cylinder test in which the ultimate strain of the concrete had to be reached for the cylinder to fail. The nonlinear behavior of prestressed concrete subjected to extreme compressive stresses at release was referred to as its "self-relieving mechanism."[10] As the level of stress on the section increased, the member deformed more, thereby, 'relieving' the stress.

In conclusion, the authors did not make any recommendations as a result of this study citing several factors that influence the relaxation of this limit. These factors included creep, shrinkage, concrete strength gain, bond capacity, confinement, and accidental eccentricity of the prestressing force.[10]

In 2001, Noppakunwijai et al. conducted a research study in which two pretensioned, inverted-tee beams were fabricated and monitored.[11] The compressive stresses at release according to linear analysis for the two members were $0.79f'_{ci}$ and $0.84f'_{ci}$ at the ends, respectively. Primarily, the creep, shrinkage, and the camber of the two specimens were measured for approximately 100-days after the specimens were released. The creep and shrinkage were predicted with available methods fairly well over time. In addition, the short- and long-term camber at midspan were estimated within adequate margins of error. As a result, the authors concluded that these factors did not inhibit the increase of the allowable stress in compression at prestress transfer.

Furthermore, they supported the removal of the current allowable stress limit. In its place, they proposed a strength design approach.[11] Essentially, the proposed method treated a prestressed member like a reinforced concrete column subjected to an axial load and a bending moment. Load factors were applied to the axial force and bending moment and resistance factors were applied to the nominal axial and bending moment capacities. The end result was a required compressive strength of the concrete at release that was a function of the area of top tension reinforcement. As compared to the current allowable stress limit, the proposed approach suggested a significantly lower required concrete strength at release. The results of the approach for a PCI Standard rectangular section, 16RB40, are provided in Figure 1. The entire premise of the design procedure relied on the claim that the allowable stress in compression was not a serviceability limit, but a strength limit. In the words of the authors, the allowable stress at release "appears to be an indirect way of checking that concrete will not 'crush' due to prestress transfer."[11]

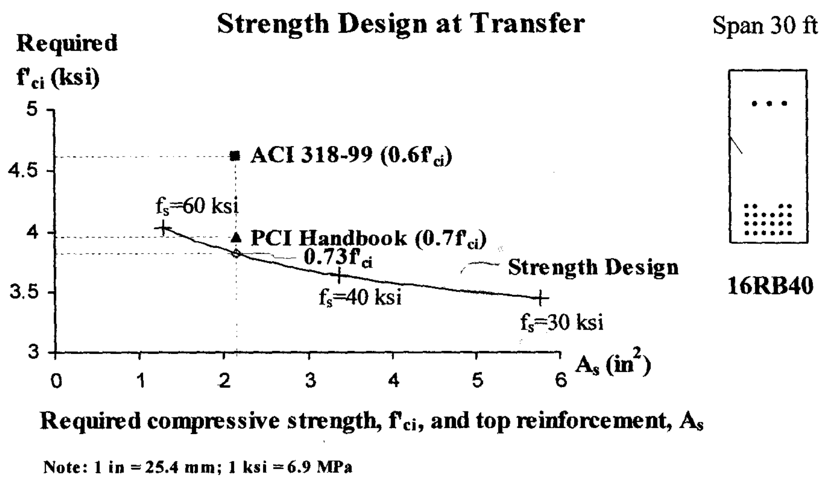

Figure 1: Results of the strength design approach for the PCI standard 16RB40 [11]

In 2003, a research study investigating the allowable release stresses was conducted at the University of Texas (Project 4086).[12] In this study, Castro, Kreger, and Bayrak fabricated 30 scaled, pretensioned beams with the maximum compressive stress at release ranging from $0.46f'_{ci}$ to $0.91f'_{ci}$. At release, the beams did not exhibit any visual indications of damage. The camber was monitored for all of the specimens at release and for approximately 90 days afterwards. The measured short- and long-term camber for each beam were compared to those estimated with several available methods. For the most part, the short-term or 10-day camber was better predicted for the beams subjected to allowable release stresses than for those subjected to elevated stresses. In fact, the short-term camber was significantly underestimated for the overstressed beams. The long-term or 90-day camber was more accurately predicted than the 10-day camber and was fairly consistent regardless of the stress at release. In conclusion, the authors suggested that increasing the allowable stress at release was a possibility if the short- and long-term camber was more accurately predicted and within reasonable limits. In addition, in the final TxDOT report for this project, it was strongly suggested that the live-load performance of overstressed beams be evaluated before the allowable stress is increased.[13] As a result, Project 5197, the research study described herein, was initiated.

In 2006, a research project was conducted by Hale and Russell to evaluate the effect of increasing the allowable release stress in compression on the short- and long-term losses of the prestressing force.[14] The purpose of the study was to compare the measured prestress losses to the predicted prestress losses for pretensioned beams subjected to conventional and elevated stresses at release. As such, four I-girders were fabricated with the maximum release stress ranging from $0.57f'_{ci}$ to $0.82f'_{ci}$. The loss of prestress was monitored in each of the girders at release and for 1-year afterwards.

The measured elastic shortening losses and the measured long-term losses (1-year) were compared to those estimated by three design procedures. The three loss procedures included the PCI Design Handbook method, the AASHTO LRFD Bridge Design Specifications estimate (2004), and the NCHRP Report 496 procedure. For the three specimens that exceeded $0.60f'_{ci}$ at release, an effective modulus was used to compute the transformed section properties and the elastic shortening losses. According to the authors, this effective modulus accounted for the inelastic behavior of the pretensioned members at release.[14]

The results of the prestress loss comparisons indicated that the loss of prestressing force was adequately predicted regardless of the compressive stress at release. In fact, for every beam, the ratio of total losses to the stress at release was essentially equal. For this reason, the authors recommended an increase to $0.70f'_{ci}$ even though a beam subjected to a stress of $0.82f'_{ci}$ performed adequately according to their criteria.[14]

In the aforementioned research projects, several effects of increasing the allowable release stress in compression were studied. The parameters investigated in each project are summarized in Table 2. While all of the studies supported the increase of the compressive stress at release, none of them investigated the impact of this stress limit on the live-load performance of the pretensioned member. The next two sections of this paper address this impact by investigating the early age properties of high-strength concrete and the behavior of concrete under uniaxial compression.

Table 2: Summary of variables studied by several researchers

Researchers	**Studied Variables**	**Scope of Experimental Work**
Pang et al.[9]	Compressive strength	432 – cylinders
Huo and Tadros[10]	Nonlinear behavior	None
Noppakunwijai et al.[11]	Creep, shrinkage, and camber	2 – IT girders
Castro, Kreger, and Bayrak[12]	Camber	30 – Rect., IT, T girders
Hale and Russell[14]	Effective prestressing force	4 – I girders

HIGH-STRENGTH CONCRETE PROPERTIES AT EARLY AGES

In 1995, Khan, Cook, and Mitchell investigated the stress-strain properties and the modulus of elasticity of several concrete mixes at early ages.[15] The three mixes represented low-, medium-, and high-strength concrete with corresponding 28-day strengths of 4,000-psi, 10,000-psi, and 14,500-psi. Within the first 24-hours of mixing, the cylinders were loaded in compression to obtain their stress-strain properties. The results of this portion of the study suggested that at 16½-hours, a 10,000-psi mix behaved more like normal-strength concrete than high-strength concrete. Essentially, the stress-strain curve was more nonlinear at these typical release times than at a mature age. The stress-strain curve for the same 10,000-psi mix at various times is illustrated in Figure 2. In addition, the modulus of elasticity of several mixes at early ages was measured according to ASTM C 469. The results of this portion of the study revealed that empirical design equations for the modulus of elasticity typically overestimate the modulus at early ages.[15] Primarily, this inconsistency is related to the extreme variability of high-strength concrete material properties at early ages.

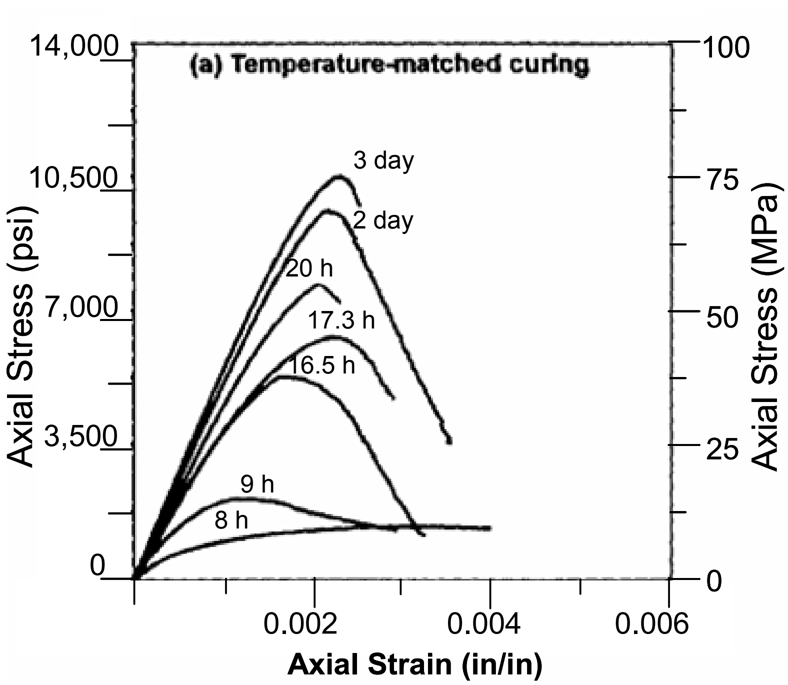

Figure 2: Stress-strain curves for a 10,000-psi concrete mix at various ages[15]

The variability of high-strength concrete properties at early ages was also investigated in a research study conducted by Tuchscherer and Bayrak in 2006.[16] In this study, the tensile strength gain of four mixes used at four Texas precast manufacturing plants was monitored. To evaluate the strength gain, the tensile strength factor was plotted over time. This factor is represented as x in the following empirical relationship for the tensile strength of concrete: $f_t = x\sqrt{f'_c}$. In the study, x was calculated as the measured split cylinder strength divided by the square root of the corresponding compressive strength. The results showed that the tensile strength factor did not reach '6,' the factor used in the empirical equation for spit-cylinder tests, until approximately 16-hours after initial mixing. This study verified the unpredictability of concrete material properties at early ages.

These two studies related to the current discussion in two ways. First, they illustrated that current design equations for modulus of elasticity and tensile strength are not conservative at early ages. If the allowable release stress in compression is increased, it is possible that beams will be released sooner. As a result, the variability in these properties will worsen. Secondly, the study by Khan, Cook, and Mitchell demonstrated that at typical release times, high-strength concrete does not behave in compression like mature high-strength concrete, but rather more like normal-strength concrete.[15] This stress-strain behavior is discussed in greater detail in the next section.

CONCRETE IN UNIAXIAL COMPRESSION

In 1963, a research investigation conducted by Hsu et al. examined the stress-strain characteristics of normal-strength concrete with a focus on internal microcracking.[17] In this study, 4-inch by 8-inch cylinders were loaded, un-loaded, sliced, and then examined with a microscope and x-ray photography. The results of the procedure indicated that the following three types of internal cracks exist in concrete elements: bond, mortar, and aggregate cracks. Before a specimen was loaded, bond cracks existed along the aggregate-paste interface. According to the authors, at approximately 30-percent of the ultimate load, these cracks increased in size and in number. At this point, the stress-strain curve departed from the linear-elastic portion of the response. As the load increased, these cracks propagated to uncracked portions of the microstructure and additional mortar cracks developed. These mortar cracks attempted to connect adjacent bond cracks. The slope of the stress-strain curve decreased more significantly during this second stage. At approximately 70- to 90-percent of the ultimate load, a continuous pattern of microcracking existed and the microstructure began its final breakdown. This stage signified the “critical load.”[17] These three stages as defined by Hsu et al. are depicted graphically in Figure 3.

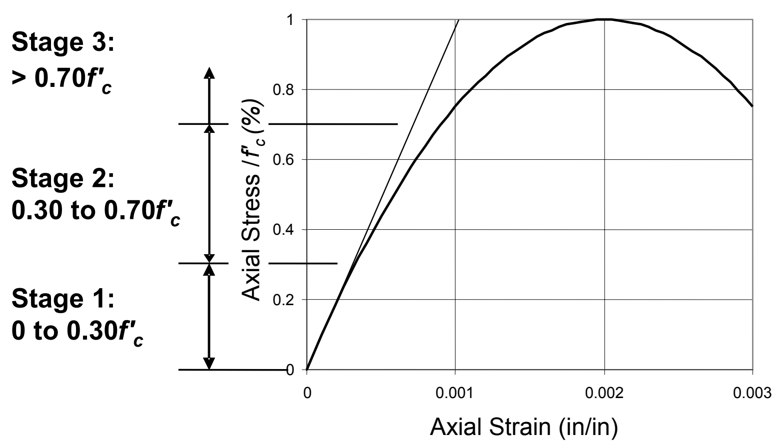

Figure 3: Illustration of findings of Hsu et al.[17] using concrete model of Hognestad[18]

In regards to prestressed concrete, this study provided some insight into the impact of increasing the allowable compressive stress at release. As concrete is loaded in compression, the departure of the stress-strain curve from a linear response is the sign of internal damage. The level of damage increases as the slope of the stress-strain curve decreases. This damage was quantified in the research study described in the next paragraph.

In 1987, Delibes Liniers conducted a research investigation to quantify the internal damage of concrete when it is loaded in compression.[19] To accomplish this task, the tensile strength of concrete was used as the 'damage indicator.' It is important to note that Pang et al.[9] in 1997 used the compressive strength of concrete as the 'damage indicator.' In this study by Delibes Liniers, mature concrete cylinders were loaded to a specified percentage of their strength. The load was maintained for either 1-minute or 15-minutes depending on the series being tested. After the specified duration, the cylinders were split according to ASTM C 496-71. The results of the study indicated that the tensile capacity of the cylinders decreased with an increasing stress-to-strength ratio and duration of the sustained load. A tensile strength reduction of approximately 30-percent was detected for cylinders loaded to 90-percent of their strength for 1-minute. In fact, a slight reduction in tensile capacity was even detected for cylinders loaded with a stress-to-strength ratio of 0.50. This reduction of less than 10-percent was stable at and below a stress-to-strength ratio of 0.65.[19] A summary of the results of the study is provided in Figure 4. Notice that the 'damage' is stable until the concrete is loaded to approximately 65- or 70-percent of its strength at the time of loading. This research quantified the damage that was present in the microstructure of the concrete when it was loaded in compression beyond the linear-elastic range.

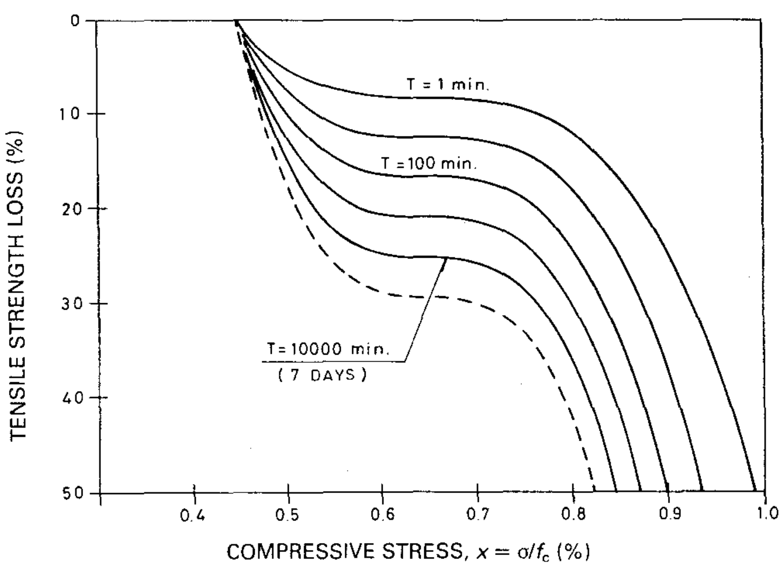

***Figure 4: Summary of tensile strength loss for typical concrete and curing conditions*[19]**

In the aforementioned research studies, the presence of internal damage was discussed when concrete cylinders are loaded to high levels of compressive stress. It is important to note that the bottom fibers of prestressed concrete girders are subjected to a stress gradient at prestress transfer. The compressive stress is not uniform as in the case of these concrete cylinder studies. Regardless, the behavior of cylinders provides some insight into the behavior of the bottom fibers of a pretensioned girder.

EXPERIMENTAL PROGRAM

The experimental program discussed herein consisted of the static load testing of 24-laboratory-scaled prestressed concrete girders. The rectangular, inverted-tee, and tee-girders were designed in a previous research study to represent TxDOT standard I-, U-, and tee-girders, respectively.[12] Each beam was tested in four-point loading with a constant moment region in the middle third of the span. During each test, deflection and strain instrumentation were monitored with the primary purpose of measuring the cracking load. The beam specifications, the test setup, and the data acquisition devices are described in the following sections.

BEAM DESIGN AND FABRICATION

The three most important parameters in the design of the scaled beams was the compressive stress at release, the cross-sectional shape, and the concrete mix design. As such, five series of beams were designed and fabricated to cover these three parameters.[12] The maximum compressive stress at release ranged from $0.46f'_{ci}$ to $0.91f'_{ci}$. Three cross-sectional shapes were utilized. And three concrete mix designs were incorporated.

The maximum compressive stresses at prestress transfer listed above were calculated from the perspective of a typical designer. The PCI Design Handbook approach for elastic shortening losses was used with the conventional allowable stress formula (Equation 1) to compute the bottom fiber stress at release. The location of maximum stress for all of the beams was 25-inches from the end of the girder, at the location of prestress transfer. However, due to the small size of these beams, the release stress at midspan was only a few percent of f'_{ci} smaller than the end stress.

$$f_{bot} = \frac{P_o}{A_g} + \frac{P_o e_p y_b}{I_g} - \frac{M_g y_b}{I_g} \qquad \textbf{\textit{Equation 1}}$$

where,

- P_o = prestressing force immediately after transfer (kips)
- e_p = eccentricity of prestressing strands of gross section (in.)
- y_b = distance from geometric centroid to extreme bottom fiber (in.)
- A_g = area of gross section (in.^{2})
- I_g = moment of inertia of gross section (in.^{4})
- M_g = moment due to dead load (in.-kips)

Since some of these beams were loaded well into the inelastic range, a nonlinear approach was also used to calculate the maximum compressive stress at release. Each girder was analyzed with a program called RESPONSE.[20] In this program, a layered analysis was used with the nonlinear, high-strength concrete model developed by Thorenfeldt, Tomaszewicz, and Jensen.[20] The stresses ranged from $0.47f'_{ci}$ to $0.84f'_{ci}$ according to this procedure.

The design of these specimens was based on TxDOT standard shapes.[12] The average ratio of the position of the geometric centroid to the total height of the section was computed for standard TxDOT I-, U-, and double-tee girders. This ratio (y_b/h) was approximately matched in the design of the scaled specimens. To simplify their fabrication, the I-, U-, and double-tee shapes were reduced to rectangular, inverted-tee, and tee-shapes, respectively, with the appropriately matched y_b/h ratio for each section. In addition, the beams were designed to an approximate scale of 3:1. All of the shapes fabricated in Project 4086 are provided in Figure 5. Each beam was 15-feet in length.

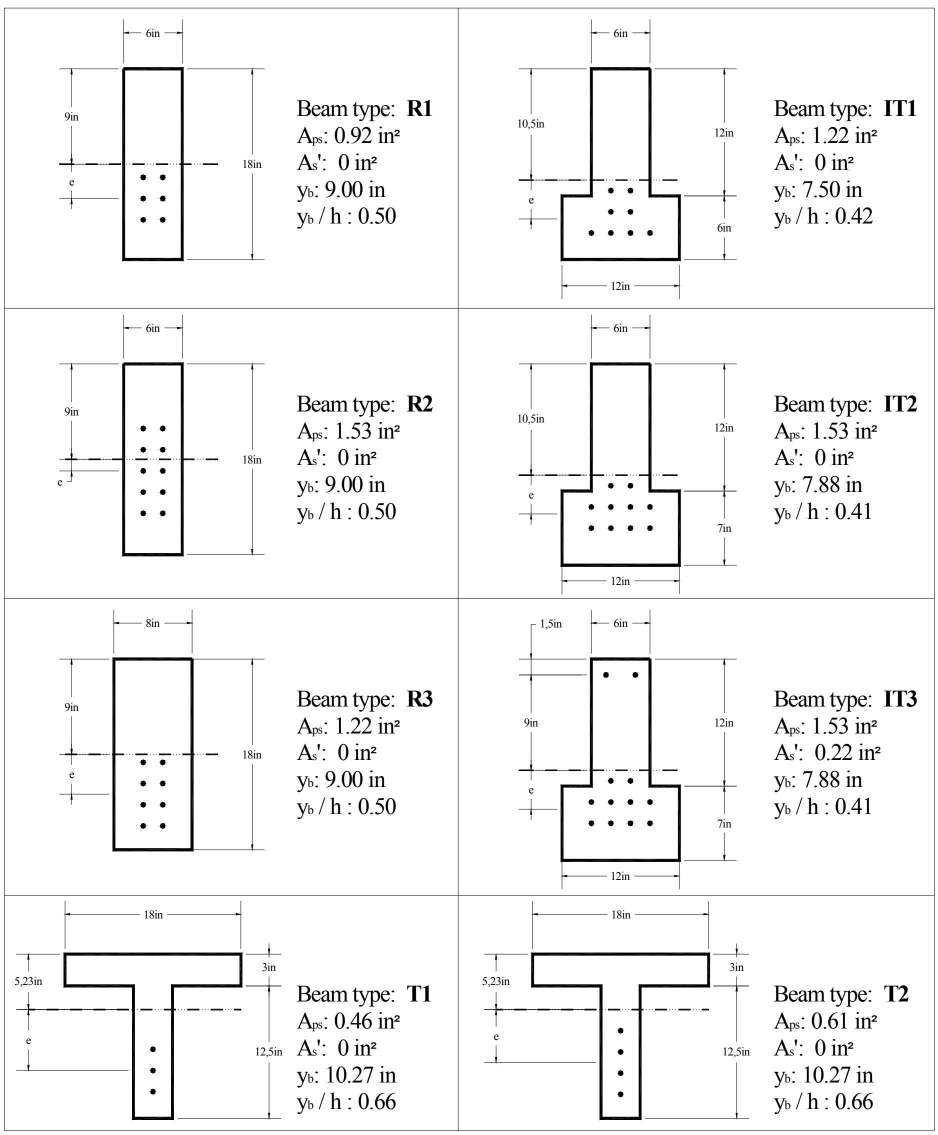

Figure 5: Different beam types fabricated in Project 4086[12]

In the fabrication of these beams, three concrete mix designs were used. Mix 1 and Mix 2 were identical with the exception of the coarse aggregate type. The first mix used river rock while the second mix used crushed limestone. Mix 3 was similar to Mix 2 except some of the Type III cement was replaced with class C fly ash. This mix represented an option in Texas to reduce the alkali content of the mix which subsequently reduces the rate of strength gain. The properties of the three mixes are listed in Table 3.

Table 3: Components (per cy) and properties of the mix designs used in Project 4086[12]

	Mix 1	**Mix 2**	**Mix 3**
Water / Cementitious Materials Ratio	0.33	0.33	0.34
Water (lbs)	204	203	182
Alamo Type III Cement (lbs)	608	608	373
W.A. Parish Class C Fly Ash (lbs)	-	-	170
Natural River Sand (lbs)	1183	1177	1322
1-inch River Rock (lbs)	2044	-	-
1-inch Crushed Limestone (lbs)	-	2042	2006
High-range water-reducing admixture (oz) -Rheobuild 1000 by Master Builders-	158	158	109
Retarding admixture (oz) -Pozzolith 300R by Master Builders-	21	21	16
Unit weight (lbs/ft^3)	154	158	150
7-day Compressive Strength (psi)	8330	8670	6375
28-day Compressive Strength (psi)	10030	10000	7390
28-day Modulus of elasticity (ksi)	5900	4850	5010
Slump (in)	7	8.5	9

TEST SETUP AND INSTRUMENTATION

All 24 beams were subjected to four-point loading. Each beam was loaded to 30-percent above the measured cracking load in most cases. In a few tests, the beam was unloaded after the cracking load was clearly defined but before this 30-percent level was reached. A double-acting hydraulic ram was positioned at the midspan of the specimen to apply the load. The load was transferred to the third points of the girder with a spreader beam. At each end, the beam reacted against concrete blocks. Two steel plates 'sandwiching' a round steel bar were positioned between the blocks and each end of the girder to imitate simply supported boundary conditions. For the pinned condition, the bar was welded to the bottom steel plate. For the roller condition, the bar was permitted to roll freely. The test setup is illustrated in Figures 6 and 7.

During each test, a variety of instrumentation was used. To measure the applied load, a 100-kip capacity load cell was attached to the hydraulic ram at midspan. A pressure gauge on the hydraulic pump was also monitored to verify the applied load. The midspan deflection was obtained continuously with a string potentiometer at midspan. Lastly, several devices were used to measure the longitudinal strains in the section. Direct current displacement transducers (DCDTs) were fixed 1-inch from the top and bottom of the extreme fiber on both sides of the beam. Also, internal strain gauges attached to the prestressing strands were monitored during each test. All of these devices with the exception of the internal strain gauges are depicted in Figure 6.

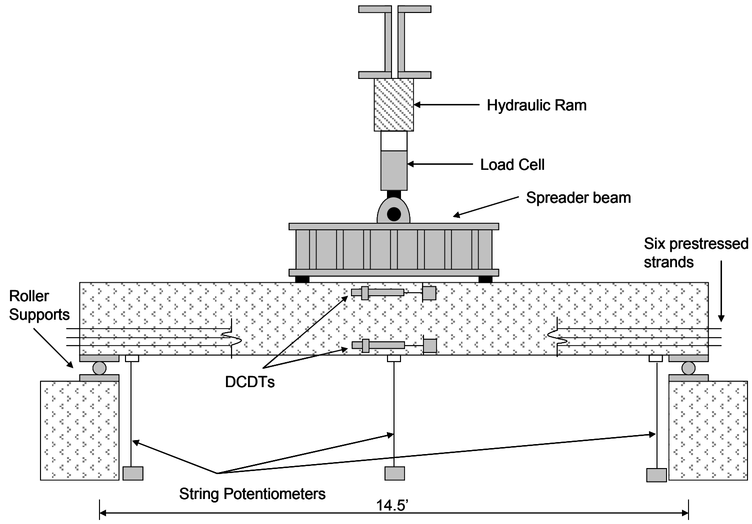

Figure 6: Test setup for static load testing of scaled girders

Figure 7: Picture of static test setup for scaled girders

STATIC TEST RESULTS

During each static test, the cracking load was measured with the aforementioned instrumentation and verified with visual observations. These measured cracking loads were compared to cracking loads predicted using two typical design procedures. In each procedure, an estimate of the short- and long-term losses of the prestressing force was made and then, the cracking load was predicted. In both cases, assumptions consistent with a typical prestressed concrete designer were utilized. The accuracy of the cracking load prediction was plotted versus the maximum compressive stress at release for both procedures. These comparisons evaluated the impact of the compressive stress at transfer on the live-load performance of pretensioned girders.

MEASURED CRACKING LOADS

In prestressed concrete members, flexural cracks form when the tensile stress in the concrete exceeds the modulus of rupture value, f_r. Until this stress is reached, the beam essentially deforms as a linear-elastic material. For instance, the relationship between the load and the midspan deflection is linear until the beam cracks. As a result, one clear indication of the cracking load is that at which this relationship ceases to be linear. For all of the beam tests, the measured cracking load was obtained with this information. A sample load-deflection plot is provided as Figure 8.

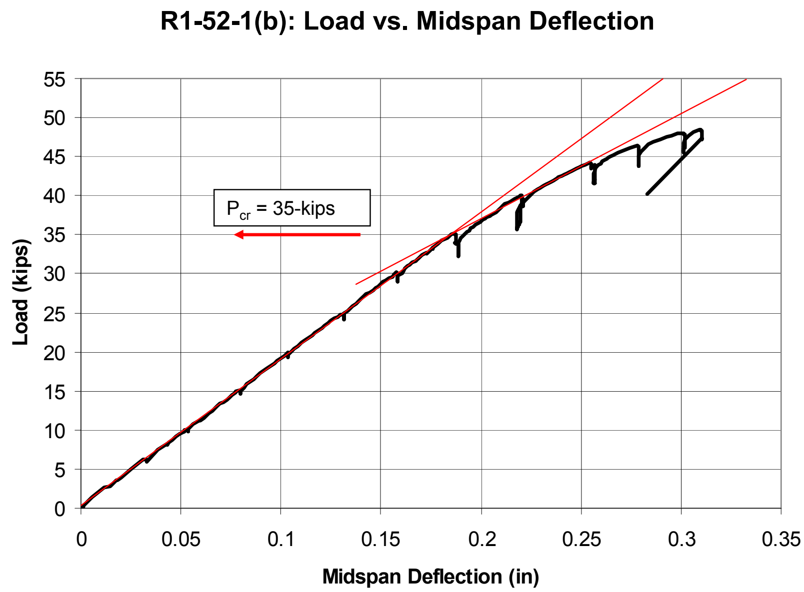

Figure 8: Sample load versus midspan deflection plot

The same linear relationship exists between the applied load and the top and bottom strain in the section. From the bottom DCDT measurements, the deflection was measured between two fixed points and then divided by the gauge length to determine the strain in the concrete at that location. This strain was plotted with the applied load and examined to determine the measured cracking load. A sample plot is provided in Figure 9.

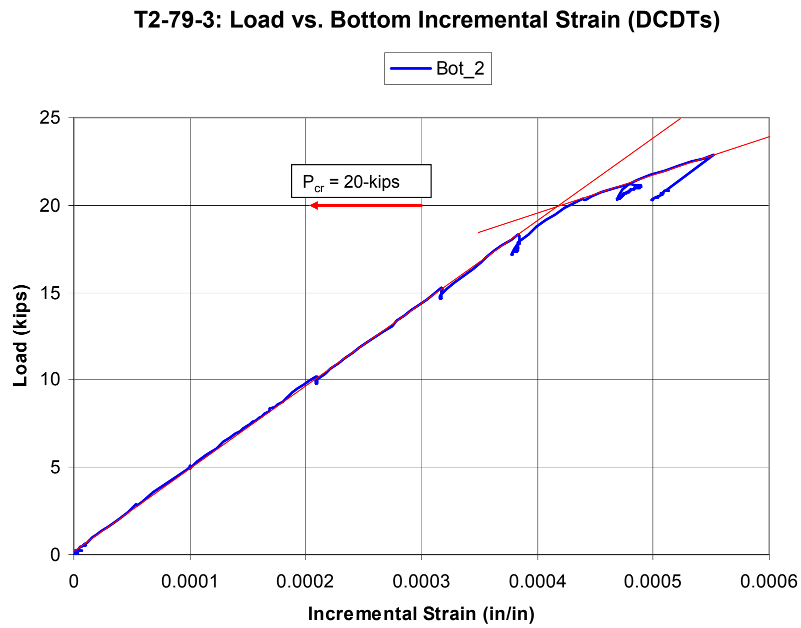

Figure 9: Sample load versus bottom strain (1-inch from extreme fiber) plot

In addition, the cracking load was verified with visual observations. T. Y. Lin once wrote, *"Attention must be paid to the fact that the modulus of rupture is only a measure of the beginning of hair cracks which are often invisible to the naked eye."*[21] As such, the ability to locate the first crack in each test was not critical. The important criterion of measuring the cracking load was to obtain a value for each beam using the same approach for all.

PREDICTED CRACKING LOADS

Two procedures were used to predict the cracking loads for all of the scaled girders. The procedures included the PCI Design Handbook method[3,22] and the AASHTO LRFD Bridge Design Specifications 2005 method.[4] For each beam, each of these procedures was used to estimate the effective prestressing force and to predict the cracking load. The procedures were identical for each beam regardless of the compressive stress at release. This consistency was essential to evaluate the impact of increasing the allowable stress limit on current design procedures.

The prestress loss procedure in the PCI Design Handbook was developed by ACI-ASCE Committee 423 in 1979.[3,22] In this simple method, the total loss of the prestressing force was divided into four categories: elastic shortening, creep of the concrete, shrinkage of the concrete, and relaxation of the strands. Throughout the procedure, the gross section properties were utilized.

The prestress loss procedure in the 2005 edition of the AASHTO LRFD Bridge Design Specifications[4] was based on the findings of the NCHRP Report 496.[23] In the NCHRP Report, a procedure was developed that incorporated material property equations for creep, shrinkage, and the modulus of elasticity of concrete into a loss procedure that divided the total loss calculation into several time-dependent stages. The stages accounted for in this study included the losses due to elastic shortening, creep and shrinkage of the girder concrete, and relaxation of the prestressing strands. For the most part, this procedure was adopted in the AASHTO Specifications with only minor simplifications. The most important simplification was the use of gross section properties in the AASHTO procedure as opposed to the transformed section properties in the NCHRP Report 496. This change affected the elastic shortening loss equation and subsequently, the amount of sustained stress in the concrete, f_{cgp}, used in the computation of creep losses.

In each of these procedures, the effective prestressing force was used to calculate the cracking load. This force accounted for the reduction of the initial prestressing force by the previously mentioned short- and long-term losses. The cracking load was predicted with Equations 2 and 3.

$$M_{cr} = \left(\frac{I_g}{c_b}\right)\left(\frac{P_{eff}}{A_g} + \frac{P_{eff} e_p y_b}{I_g} - \frac{M_g y_b}{I_g} + f_r\right) \qquad \textbf{\textit{Equation 2}}$$

where I_g = moment of inertia of gross section (in.^{4})
y_b = distance from geometric centroid to extreme bottom fiber (in.)
P_{eff} = prestressing force after all losses (kips)
A_g = area of gross section (in.^{2})
e_p = eccentricity of prestressing strands of gross section (in.)
M_g = moment due to dead load (in.-kips)
f_r = tensile strength of concrete taken as $\frac{7.5}{1000}\sqrt{f`_c}$ (ksi)
f'_c = 28-day compressive strength of concrete (psi)

$$P_{cr} = \left(\frac{6}{L}\right)(M_{cr}) \qquad \textbf{\textit{Equation 3}}$$

where L = centerline to centerline span, 14.5-ft

COMPARISON OF MEASURED TO PREDICTED CRACKING LOADS

For each static test, the measured cracking load was compared to the predicted load from both analysis procedures. The accuracy of the prediction was evaluated with Equation 4.

$$Accuracy = \left(\frac{P_{measured} - P_{predicted}}{P_{measured}}\right) \times 100 \qquad \textbf{\textit{Equation 4}}$$

The prediction accuracy for each procedure was plotted versus the compressive stress at release. These plots are provided for the PCI procedure and the AASHTO procedure as Figure 10 and Figure 11, respectively.

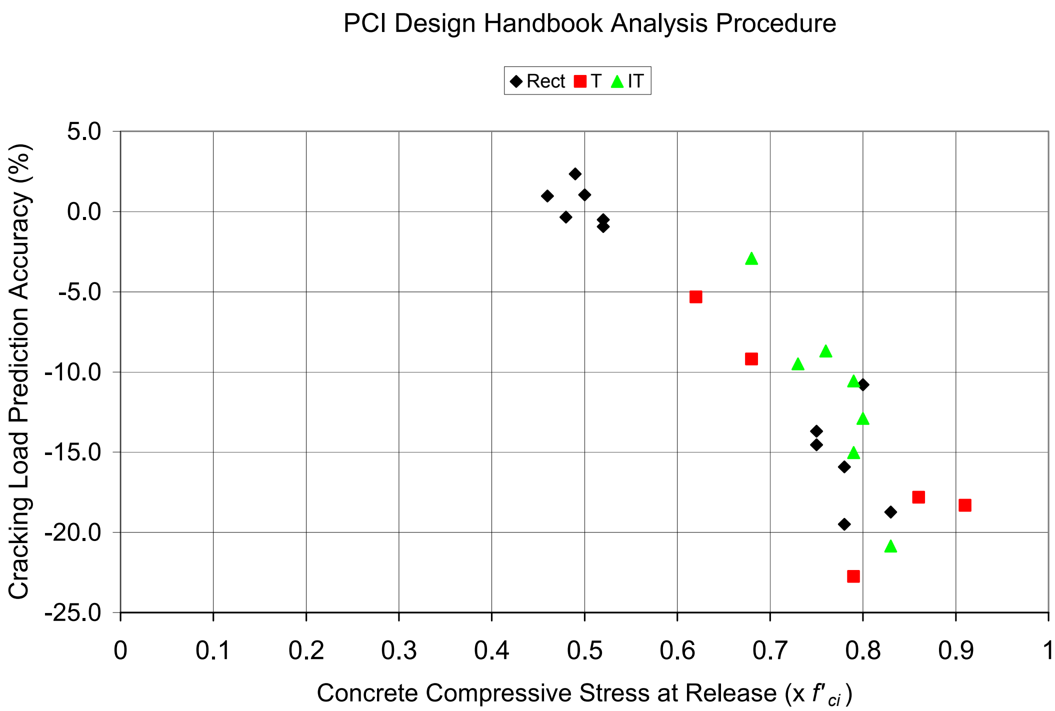

Figure 10: Comparison of measured to predicted cracking loads using PCI procedure

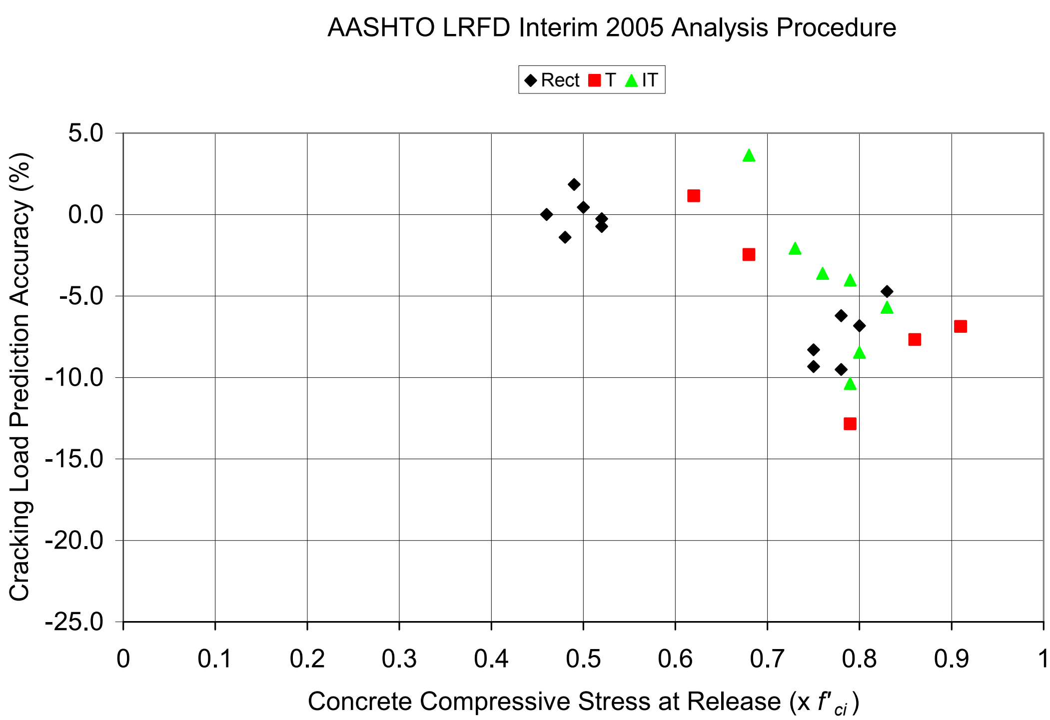

Figure 11: Comparison of measured to predicted cracking loads using AASHTO procedure

In each plot, it is clear that a discrepancy exists between the predicted and measured cracking loads as the compressive stress at release increases. This discrepancy is the result of two related factors. The first reason is due to internal damage discussed previously in the literature review. When concrete is loaded to extreme levels of compressive stress, the internal microstructure of the concrete begins to break down. This damage was represented as a loss in tensile capacity.[19] It seems plausible that some of the discrepancy between the response of the conventional and overstressed beams is a due to this internal damage.

The second reason for this discrepancy is related to the design assumptions within each analysis procedure. In both of these methods, the member was assumed to behave linearly. For overstressed beams, this assumption was not valid at prestress transfer. As a result, each procedure estimated less elastic shortening losses than if the nonlinearity of the concrete was taken into account. In addition, a nonlinear procedure would calculate a lower compressive stress in the bottom fiber of the beam at release. This smaller stress would have to be incorporated into the calculation of the predicted cracking load to account for the nonlinear behavior at prestress transfer.

Therefore, it is likely possible to better estimate the cracking load of overstressed beams if the nonlinear behavior of the material is accounted for. However, accounting for this inelastic response does not mitigate the presence of internal damage; it provides a means to account for it. For this reason, it seems inappropriate to account for the nonlinear behavior, or the internal damage, of overstressed concrete.

It must be remembered that the purpose of the allowable release stress limit in compression is to ensure the quality fabrication of prestressed concrete products. One of the primary purposes of this limit is to ensure satisfactory behavior of the precompressed tension zone of a pretensioned beam. In this sense, this serviceability limit is no different than a heat of hydration (or maximum temperature) limit for precast concrete or a temperature specification for the fabrication of rolled steel shapes. The appropriate allowable release stress limit is one guarantee that the designed beam performs as it was intended to perform.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the experimental program, the following conclusions were reached:

- Subjecting early-age concrete to excessive compressive stresses at release damages the internal microstructure of the concrete.
- The current allowable concrete release stress in compression of $0.60f'_{ci}$ has been utilized in prestressed concrete design with satisfactory results for 50 years. The findings of this research study further justify this limit.
- Increasing the allowable stress to a value between $0.60f'_{ci}$ and $0.70f'_{ci}$ may be a possibility pending the test results of full-scale girders subjected to this stress range.
- Increasing the allowable stress in excess of $0.70f'_{ci}$ is not acceptable.

Currently, full-scale testing is being performed on girders subjected to release stresses in the range of $0.55f'_{ci}$ to $0.75f'_{ci}$. This work is being conducted under the project discussed within this paper, TxDOT Project 5197. A reference for these tests and the material discussed within this paper will be completed by the end of 2006.

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