Applications of High-Strength Concrete to Long-Span Prestressed Bridge Girders

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High-strength concrete can be used to achieve longer-span or more widely spaced prestressed bridge girders. Two full-size long-span, high-strength prestressed bridge girders were constructed to investigate the effects of high-strength concrete on transfer length, camber, prestress losses, fatigue, flexure, and shear strength. The results of studies on constructibility, transfer length, prestress losses, and camber are presented.

The design strength of concrete has steadily increased in prestressed bridge girder construction over the years. For example, standard concrete mix designs for prestressed bridge girders built for the Minnesota Department of Transportation (MnDOT) have gone from 31 to 48 MPa in the past decade. These higher concrete strengths are being produced with little difficulty from readily available materials. To achieve further economy, it is likely that these design strengths will continue to increase.

High-strength concrete (HSC) offers many advantages to prestressed bridge girders, including increased span lengths or wider girder spacings, or both. HSC alone can be used to increase span lengths for a fixed bridge cross section. Shallower HSC sections can be used in place of normal-strength concrete members of the same length, enabling greater bridge underclearances or lower bridge embankments. Alternatively placing HSC girders at a wider spacing enables fewer girder lines per constant length bridge. Fewer girders required leads to lower fabrication, transportation, and erection costs.

In the first part of the study reported herein, a parametric study was conducted to determine the viability of using high-strength concrete in prestressed bridge girders. For a given bridge cross section and loading parameters, the maximum span length and required number of strands were determined as a function of concrete strength and transverse girder spacing. The allowable stresses that control the design of the girders were considered at release and final conditions. For the case of wider-spaced girders, the allowable stresses at release tended to control because a large amount of prestress must be "stored" in the girders for use at service conditions. For the case of longer spans, the allowable stresses at final conditions tended to control because the self-weight of the girders is a much larger portion of the total loads. Allowable stress limits at release were taken as compression of $0.6 f'_c$ and tension of 1.38 MPa, unless supplemented by mild steel reinforcement, in which case the tension limit was 4.9 MPa. Allowable limits at final were compression of $0.4 f'_c$ and tension of $0.5 f'_c$, (f'_c in megapascals).

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An example of the results is shown in Figure 1 for a MnDOT 45M girder 1140 mm deep. This girder is similar to an AASHTO Type IV girder except that it has a wider top flange and a shorter overall height (Figure 2). As can be seen from Figure 1, for a given span length, an increase in concrete strength enables a minor reduction in strands because of the increased allowable stresses corresponding with the increased concrete strength. Increased span lengths may be achieved with high-strength concrete by increasing the number of strands (amount of prestress) in the cross section. As more and more strands are added to the cross section, their effectiveness is reduced as they are placed further up in the cross section (at a lower eccentricity). Consequently, the viability of HSC prestressed bridge girders depends not only on reliably achieving higher strengths, but also on the amount and strength of prestressing strand that can be placed in the cross section. There is not a significant advantage in increasing the concrete compressive strengths much above 83 MPa for the example shown. Figure 1 also indicates the effect of high-strength concrete on transverse girder spacing. It may be possible to increase the girder spacing of a girder 35 m long, for example, from 1.22 to 2.13 m by increasing the concrete strength from 48 to 83 MPa.

Current design provisions of the American Concrete Institute (ACI) (*1*) and Transportation AASHTO (*2*) are based on empirical relationships developed from isolated tests of specimens with concrete compressive strengths in the range of 41.4 to 55.2 MPa. The scarcity of empirical data on higher-strength concrete has led to limits on the maximum compressive stress of 69 MPa to be used for shear and rebar development length provisions in design codes such as those of ACI. Consequently, it is of interest to investigate the implications of using higher-strength concretes.

DESCRIPTION OF EXPERIMENTAL PROGRAM

Two full-size high-strength concrete composite prestressed bridge girders have been instrumented and constructed and are being tested at the University of Minnesota to investigate issues such as transfer length, long-term prestress losses, fatigue, ultimate flexure, and shear strengths. The girders are MnDOT 45M sections 1140 mm deep and reinforced with 46 prestressing strands 15.3 mm in diameter and 1860 MPa spaced 50.8 mm on center. The cross section of the girders is shown in Figure 2. High-strength concrete was utilized to maximize the girder lengths. Such a case required closely spaced girders (1.22 m on center) in which case allowable service load stresses were the controlling factors rather than allowable release stresses. The span length of 40.5 m represented a 48 percent increase over the maximum span length currently produced using conventional 48 MPa concrete fabricated with 1860-MPa strands 12.7 mm in diameter.

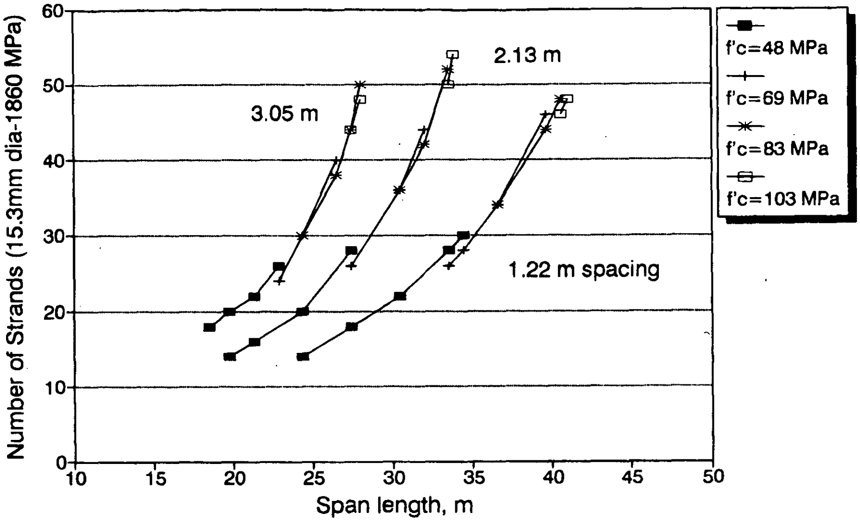

FIGURE 1 Effect of HSC on girder span length: strands required versus span length, MnDOT, 45m section.

The girders were designed to incorporate the following variables: concrete mix design, draping/debonding combinations, and stirrup configuration. Laboratory tests in a companion study (*3*) showed that local aggregates could be used to achieve high-strength concretes. Limestone aggregates provided mixes with the highest strengths, but mixes with glacial gravel aggregates reached similar strengths with the addition of microsilica. One girder (Girder I) was therefore cast with a limestone mix, and the second girder (Girder II) was cast with a glacial gravel mix incorporating 7.5 percent (replacement by weight of cement) microsilica. To investigate the effect of strand draping versus debonding on girder behavior, one end of one girder was fully draped (12 draped strands) to control end stresses, whereas the remaining three ends utilized a draping/debonding combination (optimized at four draped and eight debonded strands). Two epoxy-coated shear stirrup details were considered: a typical U stirrup and a modified-U stirrup using leg extensions along the length of the member. The modified-U stirrup was used to investigate the effect of better anchorage conditions. Figure 3 depicts the two test girders and their respective variables.

The girders were cast in August 1993 and have been monitored to determine the prestress losses and changes in camber over time. Each girder will be subjected to fatigue loading intended to simulate the maximum moment caused by a standard HS25 truck load. After approximately 1 million cycles, a static load test will be performed to determine the damage status before increasing the load level to the cracking load. Occasional overloads in both static and fatigue testing as previously recorded by MnDOT will be included.

The fatigue strength of prestressed concrete is controlled by the strand stress range. Presently, AASHTO specifications employ an indirect design criterion for flexural fatigue strength of prestressed concrete girders through limitation of the nominal concrete tensile stress in the precompressed tensile zone. Full-size girder tests can verify fatigue strength by investigating the strand stress ranges

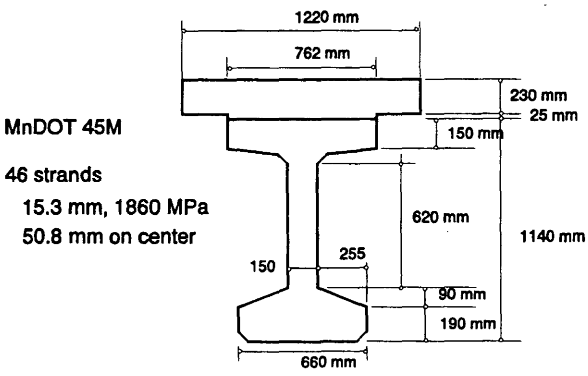

FIGURE 2 Cross section of test girders.

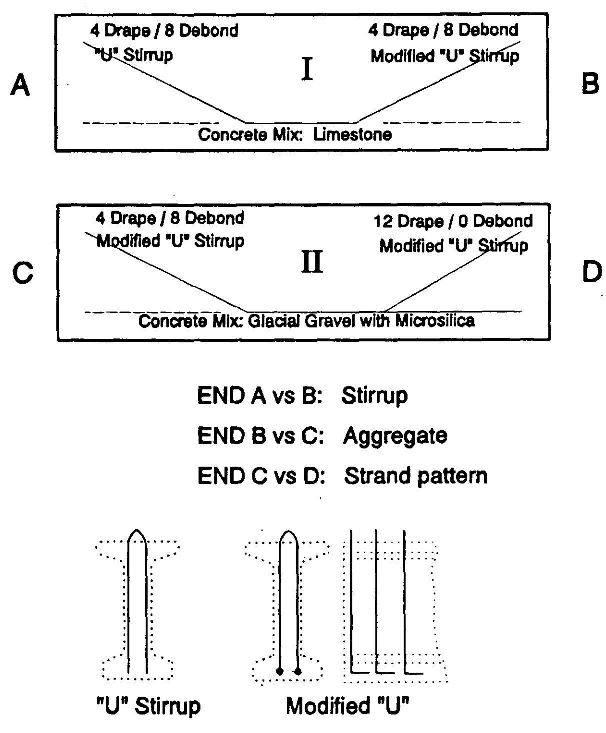

FIGURE 3 Test girder variables.

under load combinations. Strand stress ranges induced by the loading will be measured, along with the compressive stress distribution through the section depth. Comparisons will be made of the actual versus the predicted cracking load-deformation behavior. Additionally, any permanent deformation because of fatigue damage will be noted.

After the fatigue studies, each girder will be tested to ultimate in flexure. As with the fatigue tests, induced strand stresses will be measured, along with deflection and concrete stress distribution. Compression stress block distribution will be measured for verification of the neutral axis and composite action. Shear tests will then be performed by loading the ends at a low shear span-to-depth (a/d) ratio.

MATERIAL STRENGTHS

Prestressing Strands

The Grade 270 (1860-MPa) low-relaxation strands 15.3 mm in diameter used for the two girders were tested for ultimate strength and modulus of elasticity. The ultimate strength was found to be 1850 MPa, whereas the modulus was determined to be 200 700 MPa. The actual area was found to be 147 mm^2, compared with the nominal area of 139 mm^2. The rolls of prestressing strand were covered and stored outdoors at the prestressing yard for approximately 1 week before use. Strand surface condition appeared to be free of surface rust and oil.

Girder Concrete

The concrete mix used in Girder I consisted of Type III portland cement, sand, crushed limestone aggregate, and a superplasticizer. The mix had an average water/cement ratio of 0.32. The mix used for Girder II also used Type III portland cement, sand, and superplasticizer. However, rounded glacial gravel aggregate was used. In addition, microsilica was used at a rate of 7.5 percent replacement by weight of cementitious material. The mix for Girder II had an average water/cement ratio of 0.36.

A release strength of 61.5 MPa and a 28-day strength of 72.4 MPa were required of the girders. To ensure that these strengths would be achieved, the target nominal mix design strength was 83 MPa at 28 days. Actual concrete strengths achieved in the field were above specifications and are shown in Table 1. The glacial gravel mix incorporating microsilica achieved a strength of 71.9 MPa in 18 hr and continued to increase to 78 MPa by 28 days. The limestone mix surpassed the required release strength at 21 hr with a strength of 64 MPa. The strength then continued to increase well above the design requirement to a strength of 83.4 MPa at 28 days. Both mixes showed good workability and consolidation during placement, and no modifications were made to the standard construction techniques followed by the precast manufacturer. The high strengths obtained in these girders represented a 30 percent increase over those typically achieved with the manufacturer's current practice.

Deck Concrete

A standard MnDOT bridge deck mix was specified for the deck with a 27.6-MPa required strength. The required 28-day concrete strength of 27.6 MPa was surpassed at 7 days. Test results indicated the strength to be 35.3 MPa at 7 days and 40.4 MPa at 28-days. These results are tabulated in Table 1.

FABRICATION AND TRANSPORTABILITY

As noted previously, the girders were constructed in August 1993. Strands were specified to be tensioned to a level 0.75 f_{pu} after seating losses. Hydraulic jacks pulled each strand individually to 198 kN. On the basis of the nominal strand area of 139 mm^2, strands were tensioned to 76.7 percent before seating (72.4 percent before seating using actual measured area of 147 mm^2). A total of 92 strain gauges were installed on the strands and monitored throughout the tensioning process. A total of 83 percent of the gauges indicated that

TABLE 1 Material Properties

	Required	Actual
Girder Concrete Compressive Strength	18 hrs = 61.5 MPa 28 day = 72.4 MPa	I 21 hrs = 64.0 MPa 28 day = 83.4 MPa II 18 hrs = 71.9 MPa 28 day = 78.0 MPa
Deck Concrete Strength	28 day = 27.6 MPa	7 day = 35.3 MPa 28 day = 40.4 MPa
	Nominal	**Measured**
Prestressing Strand	Area = 139 mm^2 E_{ps} = 200,000 MPa f_{pu} = 1860 MPa	Area = 147 mm^2 E_{ps} = 200,700 MPa f_{pu} = 1853 MPa

the strands were tensioned to 1280 MPa or 72.9 percent of f_{pu} after seating based on the nominal area (69.1 percent after seating based on the measured strand area). A standard deviation of 40 MPa was recorded, and the remaining 17 percent of gauges were damaged during the tensioning process.

Concrete was mixed at the plant batch station. No additional heat or steam curing was incorporated. Formwork was stripped shortly after each girder reached the required release strength (18 to 21 hr). Because both girders were cast on the same bed, the girder with the glacial gravel mix was stripped and sat for nearly 4 hr before strands were released while waiting for the limestone mix to reach the required strength. Consequently shrinkage and temperature cracks occurred near the center and quarter points of the girder cast with the glacial gravel/microsilica combination. Cracks penetrated the cross section from the top flange to the bottom flange. These cracks fully closed once the strands were released. Cracks were not seen in the limestone girder because the strands were released within 1 hr after the formwork was removed.

At each end of the girders, strands were simultaneously flame cut. One to three cracks developed in each of the debonded end regions (*A* through *C*) during detensioning. Web cracking was observed in the draped end (*D*) of Girder II. The girders were then relocated in the prestressing yard until they were moved to the test site. Lift hooks were placed in the girder to ensure stability during handling. Placement was dictated by not exceeding allowable stress limits while assuming a small rotation about the roll axis in accordance with PCI stability criteria (*4*). A factor of safety of 2.0 governed the hook locations. No stability problems were encountered during handling.

In addition to the shrinkage cracks, the following cracks were observed. One to three cracks appeared in the bottom flange at each beam end, starting at the web-to-bottom flange interface and proceeding at a 45-degree angle longitudinally down the extreme edge of the bottom flange (approximately 8 in. from the end), then extending vertically downward to the lower edge of the bottom flange. Cracks did not occur on the bottom of the girder, and no splitting cracks were observed along or between the strands.

Both girders were transported to an off-campus testing facility in October 1993. No stability problem or additional cracking was observed during transportation. Deck formwork placement began shortly after the girders were located in the testing facility. A composite deck was cast on each girder individually in February 1994 using unshored construction.

INSTRUMENTATION

Instrumentation included strain gauges applied to the strands and transverse reinforcement, embedded concrete strain gauges and vibrating wire strain gauges, DEMEC (Detachable mechanical) gauges, and displacement transducers. The instrumentation was located in both the girder and the deck to optimize the information on prestress losses, creep and shrinkage, long-term deflections, transfer length of strands, composite interaction, flexure, shear, and cracking.

Of the 46 strands a total of 9 were instrumented with strain gauges to investigate tensioning and transfer length. The pairs of debonded strands were instrumented at 380, 560, and 760 mm from the end of their respective sheath points. At 0.45L (45 percent of the member length) and midspan of the girders, strands were instrumented to obtain data about the strain along the length of the strands at the time of prestress, stress ranges in the strands during fatigue tests, and strains in the strands at ultimate in flexure. In addition, 60 DEMEC gauges were applied to the surface of the concrete at the end regions of the beam spaced 150 mm on center to measure the external concrete strains. Vibrating wire gauges were used to investigate the change in concrete stress over time because of creep, shrinkage, and other environmental effects, and to provide additional information on prestress losses. Embedded concrete gauges were also placed at 0.2L, 0.3L, and midspan in the girders to investigate the width and depth distribution of concrete compression stress block during the ultimate flexure tests.

Deck instrumentation consisted of strain gauges on the longitudinal reinforcement and embedded concrete gauges to investigate width and depth distribution along the length of the member. Vibrating wire gauges were placed to maximize information on deck shrinkage and also to investigate compression stress block distribution during the flexure tests.

External instrumentation included tiltmeters, linear variable differential transformers (LVDTs), and acoustic emissions equipment for monitoring rotation, deflection, and first crack detection, respectively. Ambient temperature and relative humidity were continuously monitored. Data collection began with strand prestressing and continued through fabrication and deck casting. Data are being collected periodically until the tests are completed.

RESULTS

Transfer Length

Instrumentation used to investigate transfer length included surface-embedded DEMEC gauges, strain gauges mounted on individual strands, and vibrating wire gauges (Figure 4). Figure 5 shows the strain that was measured on Girder I (End A) using the DEMEC surface gauges at release. The figure also includes data obtained from a vibrating wire gauge located at a slightly greater eccentricity (Figure 4). Superimposed on Figure 5 is the calculated strain distribution obtained from the following equation:

$$\epsilon = \frac{\sigma}{E_c} = \frac{1}{E_c} * \left[-\frac{P}{A} \pm \frac{Pe * c}{I} \mp \frac{M_g * c}{I} \right] \quad (1)$$

where

ϵ = strain at distance c from center of gravity (cg) of cross section,
σ = stress at distance c from cg of cross section,
P = effective prestress,
e = strand eccentricity measured from cg of cross section,
A = cross-sectional area of concrete,
I = gross moment of inertia of concrete,
c = distance from cg of cross section to location at which strain/stress is to be determined,
M_g = external moment caused by self-weight, and
E_c = modulus of elasticity of concrete.

The calculated strain was determined using actual data obtained during tensioning and release to solve for the effective prestress level. The calculated strains were also based on the AASHTO assumed transfer length of 50 d_b (strand diameters). The shallow dips in the calculated strains reflect the effect of gravity load, causing a decrease in the concrete compression strain at the level of the

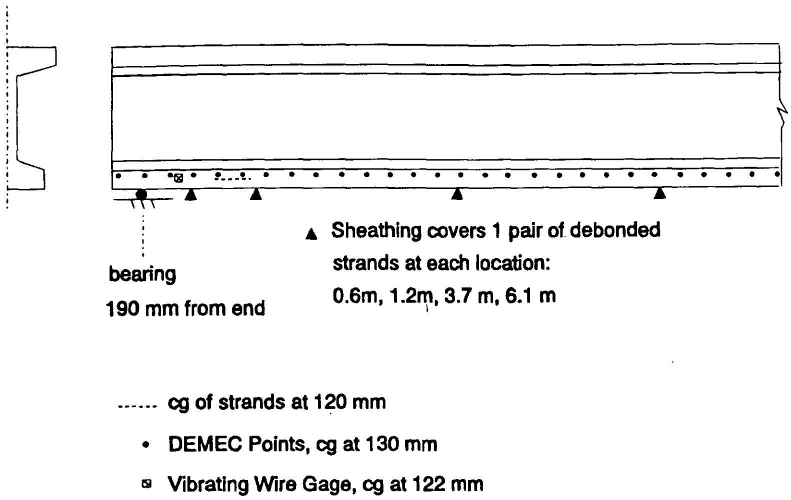

FIGURE 4 Instrumentation for transfer length measurements.

strands. The slight increases following the dips represent the initiation of bonding the pairs of debonded strands along the length of the girder. This increase effect appears minor because of the percentage of debonded strands becoming bonded versus the total bonded strands already in the cross section. The calculated strain is generally lower than the actual data indicate, although within reason. DEMEC gauges were placed near the center of gravity of strands to measured surface strains, and data include the localized effect of force concentration in the bottom flange of the girder.

The calculated transfer length values using the assumptions above give reasonable results. In addition, transfer lengths were determined graphically from the experimental data using methods proposed by Russell and Burns (*5*) and Cousins et al. (*6*). In the 95 percent average maximum strain method proposed by Russell and Burns, strain readings were first smoothed by averaging the data over three gauge lengths to reduce anomalies in the data. The average maximum strain was determined by computing the numerical average of the smoothed strains contained within the strain plateau. The intersection of a line corresponding with 95 percent of the average smoothed strain data and the smoothed strain profile represents the transfer length (Figure 6*a*). Using this procedure, transfer lengths in the range of 570 to 725 mm were obtained for the four girder ends.

The final average method employed by Cousins et al. (*6*) eliminates data points that lie outside the range of one standard deviation from the averaged strain plateau. The average strain of the remaining data points is then determined, and the transfer length is defined as the intersection of the final average strain and the data (Figure 6*b*). The results obtained with this method were similar to those obtained using the 95 percent average maximum strain method and ranged from 630 to 696 mm.

Reported transfer lengths are for straight strands that were fully bonded in the section and exclude the debonded strands. As shown by the calculated strain in Figure 5, the additional force introduced by the pairs of debonded strands is on the order of 4 percent of the total stress introduced at the ends of the girders. Reported transfer

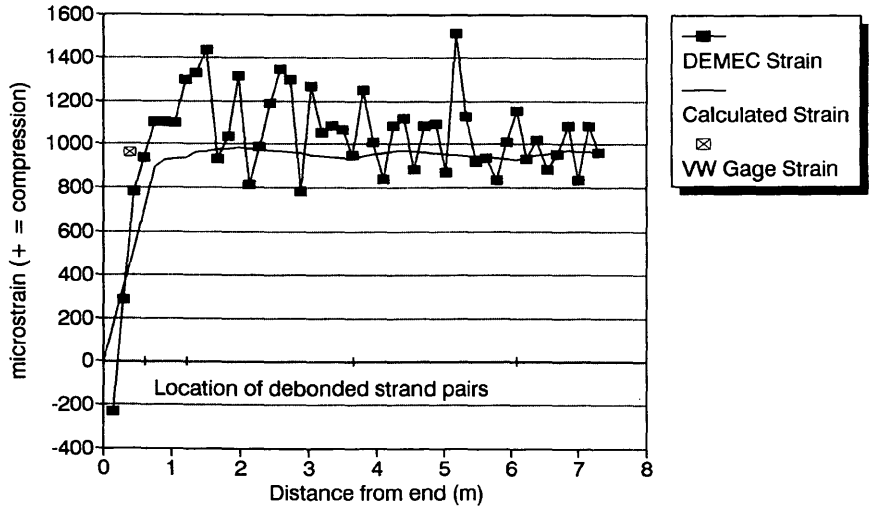

FIGURE 5 Measured and calculated strains for transfer length: UMN Test Girder I, End A, bottom flange.

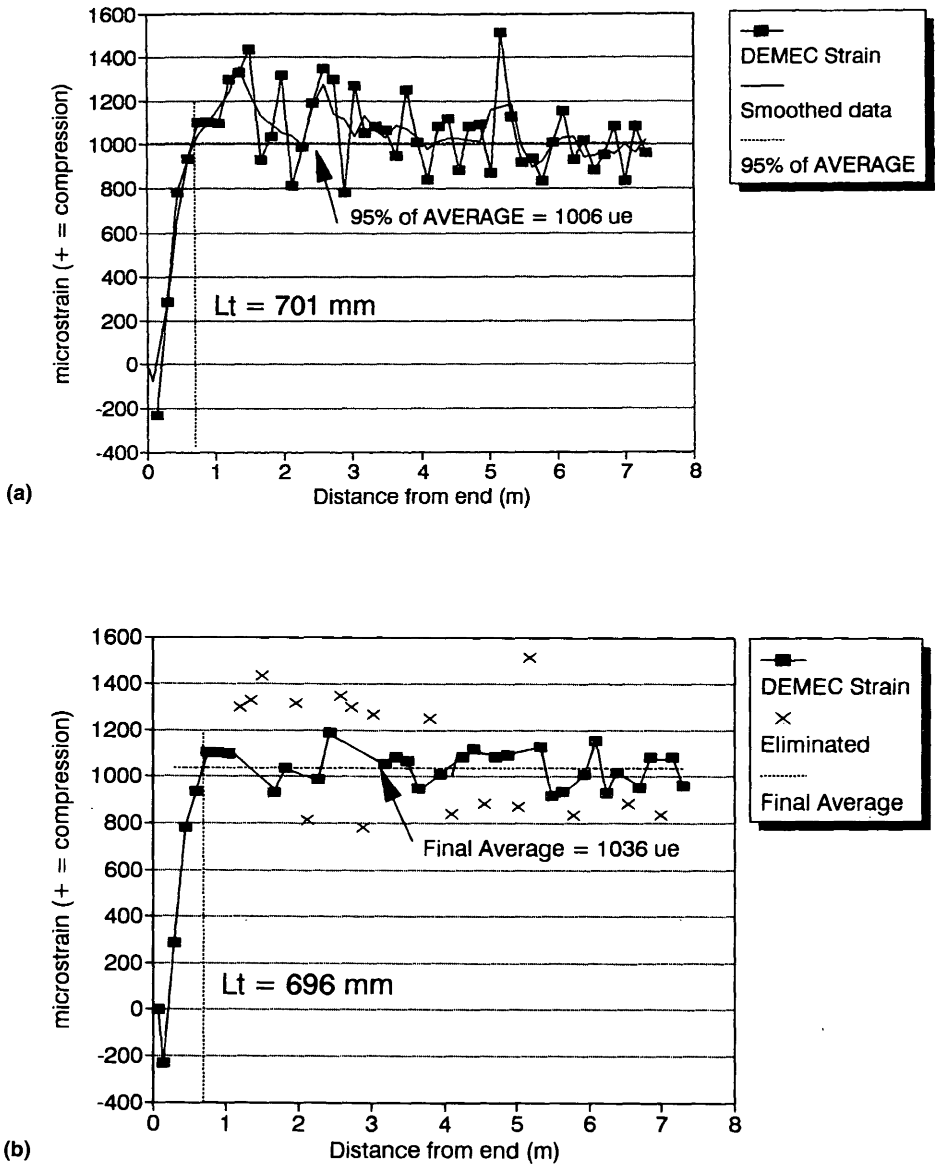

FIGURE 6 Transfer length, UMN Test Girder I, End A: (*a*) 95 percent average maximum strain method; (*b*), final average method.

lengths are therefore only for the initial force transfer of the bonded strands in the end of the girder.

A wide variation in data exists for determination of transfer length, and there is a lack of data available for strands embedded in high-strength concrete. One study about the effect of high-strength concrete on transfer length of strands 15.7 mm in diameter was reported by Mitchell et al. (*7*). Concrete strengths ranged from 31 to 89 MPa. It was shown that an increase in concrete compressive strength at the time of strand release resulted in a reduction of transfer length. Transfer lengths for single-strand rectangular specimens (200 by 250 mm) ranged from 435 to 872 mm for concrete strengths up to 65 MPa. Design expressions were proposed that take into account the concrete compressive strength at the time of force transfer and service. The Mitchell study varied from the current investigation in two ways. The Mitchell study used a third method—the slope-intercept method—for determining transfer length, and a gradual release method was employed for detensioning the strands. Both of these variations can result in shorter transfer lengths.

Russell et al. (*5*) investigated the effect of lateral spacing on the transfer length of strands 15.3 mm in diameter. Concrete strengths varied in the study from 34 to 53 MPa for these tests. Results indicated that higher concrete strengths at release result in shorter transfer lengths. Transfer lengths were 1057 mm for single-strand and 1070 mm for three- and five-strand rectangular specimens and AASHTO-type (560-mm-deep) girders with 50.8-mm strand spacing.

Table 2 summarizes transfer length estimations for the four girder ends using the two graphical approaches described earlier. Transfer lengths can be predicted using relationships from AASHTO (*2*) and ACI (*1*) (Table 2). Table 2 also summarizes the observed test data from Mitchell et al. (*7*) and Russell et al. (*5*).

TABLE 2 Transfer Length Summary

Measured Transfer Length - UMN Test Girders (mm)				
Graphical Methods Using Measured Surface Strain Data	Girder End			
	I-A	I-B	II-C	II-D
95% Average Maximum Strain Method (Russell, *5*)	701	725	570†	565
Final Average Method (Cousins, *6*)	696	569	630	676
AVERAGE of Methods (mm)	699	647	600	621
Predicted Transfer Lengths (mm)				
AASHTO (*2*)	780			
ACI (*1*)	782			
Observed Transfer Lengths of 15.7 mm Strand from Other Studies (mm)				
Mitchell (*7*)	435 - 872 (single strand rectangular specimens, slow release)			
Observed Transfer Lengths of 15.3 mm Strand from Other Studies (mm)				
Russell (*5*)	1057 (single strand rectangular specimen) 1070 (3 and 5 strand rectangular specimens, AASHTO type 560 mm deep girders, all with 50.8 mm strand spacing)			

† A spuriously high data point was omitted which indicated a transfer length of 376 mm.

Prestress Losses

Prestress losses occur instantaneously because of elastic shortening at release and over time because of steel relaxation and creep and shrinkage of concrete. Nearly all prestress losses occur in the girder within the first 6 to 12 months of being cast. Creep and shrinkage will considerably slow down after this time, and deflections will have stabilized. Vibrating wire gauges were installed at the center of gravity of the strands in each girder to measure prestress losses with time. One gauge was installed at each of the following locations: 0.45L, 0.50L, and 0.55L. The gauges were monitored continuously for 2 weeks after casting and periodically to date. Figure 7 illustrates the strain change over time for a gauge located at the center of gravity of the strands near midspan of Girder I.

Table 3 summarizes initial and time-dependent losses. Concrete strains measured immediately after release were used to provide an indication of prestress losses because of elastic shortening. Losses were also determined 28 and 200 days after girder casting. Day 200 corresponds to the week before deck casting. The level of stress loss in megapascals is tabulated for each time step (noncumulative), whereas the percent loss listed represents the cumulative loss through that time step. The percent loss was based on the measured strand tensioning level of 1280 MPa immediately before release.

AASHTO (*2*) specifications contain provisions for calculating initial and long-term prestress losses caused by elastic shortening, steel relaxation, concrete creep, and concrete shrinkage. PCI (*8*) and Naaman (*9*) use a time-step approach to determine losses for any given time. Using known material properties and transformed geometric section properties for each girder, losses were calculated for each method and are summarized in Table 3. Known properties include concrete compressive strengths and moduli of elasticity at different ages, measured creep coefficient of 1.0, prestressing steel yield and ultimate strengths, and strand tensioning level.

Measured losses were lower in Girder II than Girder I, although within a reasonable range. The difference in losses between the two girders reflects the difference in initial concrete compressive strength and elastic moduli. The greatest portion of losses occurred at transfer because of elastic shortening and varied in each beam from 173 MPa in Girder I (limestone) to 156 MPa in Girder II (glacial gravel/microsilica), as indicated in Table 3. An additional 76.5 and 46.6 MPa occurred in each girder, respectively, within the first 28 days after casting, and much smaller losses have occurred since (24.8 and 13.2 MPa). The calculated losses give reasonable estimates of the observed prestress losses when using properties appropriate for high-strength concrete.

Camber

Estimating camber and deflection of precast prestressed members is complex because of the interaction of prestress losses, loadings, and concrete strength gain with time. Camber calculations take into consideration prestressing effects, dead load, live load, erection loads, creep and shrinkage of concrete, and steel relaxation.

A rational method of estimation using multipliers to predict camber and deflection at erection and final service conditions has been adopted by the PCI (*4*). The PCI Design Handbook tabulates multipliers for prestressed concrete beams with and without a composite topping. These multipliers were based on general experience from use of normal-strength concrete and need to be reevaluated for use with high-strength concrete.

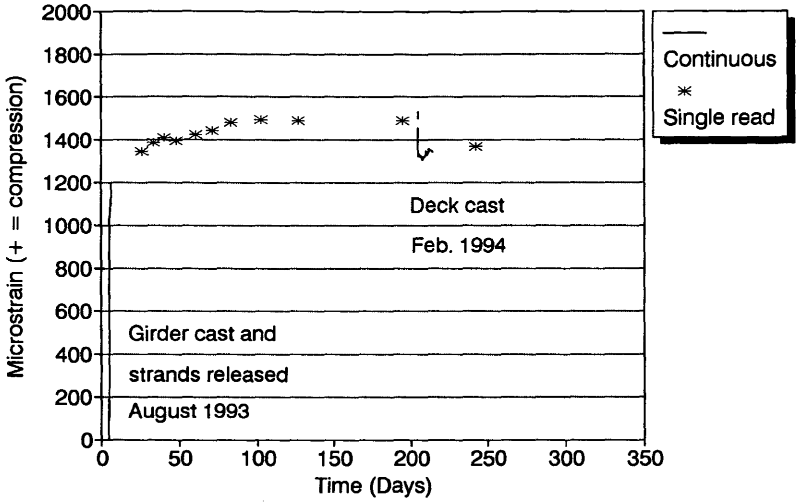

FIGURE 7 Strain change over time for prestress loss calculations: UMN Test Girder I, midspan.

Camber and deflection have been monitored for both girders since the time of strand release. Figure 8 illustrates centerline camber of both girders over time. Girder I (limestone) had an initial camber of 121 mm and increased to a maximum of 230 mm within 2 months. Girder II (glacial gravel/microsilica) had an initial camber of 97 mm and increased to a maximum of 146 mm within 2 months. Both girders then showed slight reductions in camber until the time of deck casting. Neither girder reached the camber levels of 210 and 193 mm, respectively, as predicted by the PCI method. Each girder deflected relatively as predicted because of deck casting; however, neither again achieved the overall PCI predicted levels of 152 and 133 mm. The cambers of each girder are slowly leveling out with time.

SUMMARY

Two MnDOT 45M girders (1140 mm deep) with spans of 40.5 m designed to be placed 1.22 m on center have been constructed. One girder was cast with limestone mix, and the second girder was cast with round glacial gravel mix incorporating microsilica. The girders were fabricated with 1860-MPa strands that were 15.3 mm in diameter, debonded at three ends (Ends A through C), and draped at one end (End D). The girders will be loaded cyclically to investigate fatigue and subsequently will be tested to determine their flexural and shear strengths. Data about constructibility, transfer lengths, prestress losses, and camber have been obtained.

TABLE 3 Prestress Losses

	† Measured		‡ PCI Committee (8)		‡ Naaman (9)		‡ AASHTO (2)	
	MPa	Losses	MPa	Losses	MPa	Losses	MPa	Losses
Girder I - Limestone Aggregate								
Release	173	13.5%	184	14.4%	184	14.4%	150	11.8%
28 days	76.5	19.3%	80.7	20.7%	50.3	18.3%		
200 days	24.8	21.5%	71.4	26.3%	37.6	21.3%		
Long-term losses*							258	31.9%
Girder II - Glacial Gravel with Microsilica								
Release	156	12.2%	170	13.3%	170	13.3%	139	10.8%
28 days	46.6	15.8%	81.7	19.7%	50.8	17.3%		
200 days	13.2	16.8%	72.6	25.4%	38.3	20.3%		
Long-term losses*							258	31.0%

† Measured losses were obtained by averaging 3 gages located at the center of gravity of the strands at 0.45L, 0.50L, and 0.55L along the girder length. This change in concrete strain was assumed to be equal to the change in steel strain.

‡ Losses were calculated at the center of gravity of the strands at 0.50L along the girder length. Losses at locations 0.45L and 0.55L were within ½% of that tabulated.

* Long-term losses include effect of long-term loads due to deck.

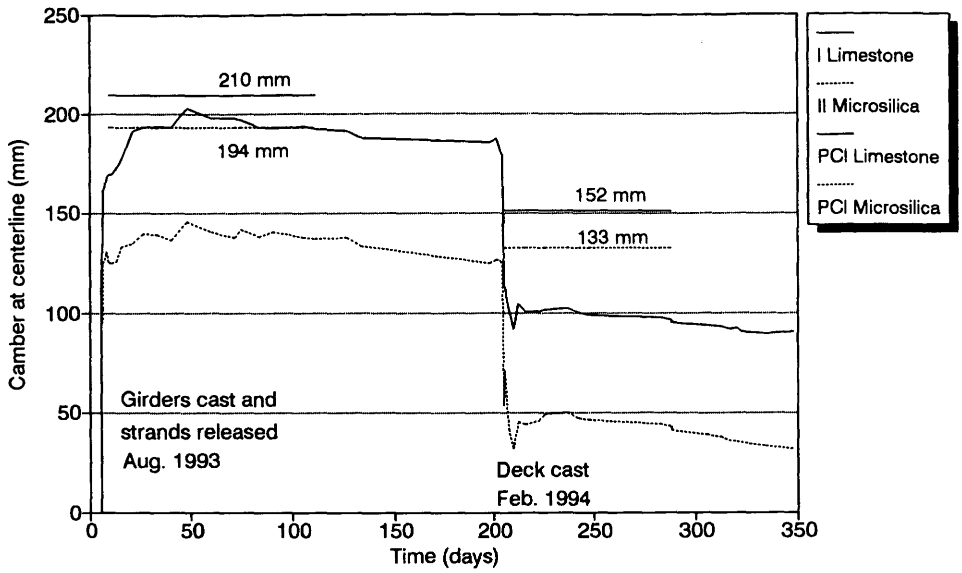

FIGURE 8 Camber change over time: UMN-HSC bridge girder tests.

The girders were fabricated with readily available materials and achieved their release strengths within 18 to 21 hr from the time of casting. There were no problems encountered in the fabrication or transportation of the long-span girders. Results indicated that transfer lengths were approximately 80 percent of those predicted by AASHTO (*2*) and ACI (*1*). Prestress losses measured to 200 days were consistent with those predicted by PCI (*8*), Naaman (*9*), and AASHTO (*2*). Girder cambers were lower than those predicted by PCI (*4*), particularly for the girder containing the glacial gravel/microsilica mix with the draped and draped/debonded strand configurations in the end regions.

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