Shear and Flexural Capacity of Salvaged High-Strength, Self-Consolidating Prestressed Concrete Bridge Girders

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ABSTRACT

Four high strength, self-consolidating concrete prestressed girders were salvaged from the 400 South I-15 bridge in Orem, Utah and tested at the Utah State University structures laboratory. The AASHTO Type–I girders were tested to failure with a point load at different locations to induce flexure, flexure-shear, and shear failures in addition to crack testing to estimate prestress losses. The results from the tests were compared to AASHTO LRFD Specifications (2012) and finite element model. The tested moment capacity of the girders was 9.6% higher than that of the AASTHO LRFD Design Manual specifications for the nominal moment capacity. The AASHTO LRFD Specifications simplified shear design method was 1.0% lower than the test results indicating that method can provide accurate results for high-strength self-consolidated concrete girders. The AASHTO LRFD refined, time dependent prestress loss and general prestress loss calculations were 8.8% less and 4.8% less, respectively, when compared to the average measured residual prestressing estimated from the cracking tests. The finite element model was able to predict to within 10% of the failure moment and shear values using measured concrete properties.

PCI Keywords: Concrete – SCC, Research

INTRODUCTION

High-strength concrete (HSC) is defined as a concrete with strength in excess of 55 MPa (8 ksi) and has numerous benefits to the precast industry. In the case of bridge girders, the high-strength properties can be used to: (1) increase span lengths, (2) decrease the number of girders per span, or (3) reduced the girder height if grade clearance is a problem. In addition, the size of other structural members can be reduced, resulting in a reduction of the overall weight. Concrete mix designs using microsilica, fly ash, chemical admixtures, or other materials, included individually or in various combinations, are substituted for portions of materials in conventional-strength concrete to obtain the higher compressive strength.

For the engineer, many of the empirically based equations for calculating girder design parameters such as prestress losses, flexural capacity and shear capacity have been based on the observed behavior of concrete members, with strengths typically below 41.4 MPa (6.0 ksi). Several researchers have investigated the accuracy of prestress loss prediction equations for girders made with high-strength concrete. Roller et al.[1] investigated four bridge girders made with high-strength concrete. Two girders were used to quantify early age flexural properties and the remaining two were studied for long-term behavior. The researchers concluded that the measured behavior could be adequately predicted with AASHTO Standard Procedures. Subsequently, researchers performed a comprehensive study for NCHRP 496[2] and recommended new equations found to be more comprehensive and accurate for a wide range in concrete strengths. The recommended procedures were subsequently adopted into the AASHTO LRFD Specifications. Several other programs have investigated prestress losses in bridge components[3,4,5,6,7,8].

The flexural capacity of precast, prestressed concrete girders made with high-strength concrete has also been of interest to design professionals and researchers. NCHRP 595[9] reviewed previous high-strength concrete research and focused on changes to the AASHTO empirical equations for more accurate flexural capacity calculations. The researchers suggested the modification of several material property relationships. They concluded that if these changes were made that the flexural capacity could be accurately calculated and the concrete strength restriction could be increased to 124.1 MPa (18.0 ksi). Other researchers have also investigated the flexural capacity of girders made with high-strength concrete[10,11,12].

Due to the brittle nature of members failing in shear, the accurate determination of its capacity is essential. Researchers for NCHRP 579[13] had a primary goal of determining whether the existing shear provisions in the AASHTO LRFD Specifications were applicable to concrete strengths greater than 68.9 MPa (10.0 ksi). Based on their research and the review of data from other researchers they concluded that the limit within the AASHTO Specifications could be increased to 124.1 MPa (18.0 ksi). This conclusion was supported by tests from other researchers[14,15].

Because of the continued interest and the readily available materials and admixtures, use of high-strength self-consolidating concrete in precast bridge girders an investigation into the applicability of current AASHTO[16] design procedures was performed. This paper presents a unique opportunity to study behavior of four precast, prestressed bridge girders after seven years of service that were fabricated with high-strength concrete.

BRIDGE DESCRIPTION

The girders studied for this case study were salvaged from the I-15 Bridge spanning over 400 South in Orem, UT. The bridges were originally constructed in 1960 and consisted of a North and Southbound structure that were each comprised of three spans. Spans 1 & 3 had lengths of 11.1 m (36.3 ft) and the center span had a length of 11.6 m (38.0 ft) over the roadway (Figure 1). The two bridges were expanded in 2004 to accommodate one additional lane of traffic in each direction. The median space between the two bridges was used for the expansion. The expansion girders were fabricated using a self-consolidating HSC mixture design. These girders remained in service for approximately seven years at which time both bridges were scheduled for demolition. Four girders were salvaged from the expanded portion of the bridge. The girders used in this research were salvaged from Span 1, and were numbered G1, G2, G3 and G4.

Fig. 1 Elevation View of the Bridge

All girders tested for this research were AASHTO Type 1, which consisted of a girder height of 711 mm (28.0 in.), bottom and top flange widths of 407 mm (16.0 in.) and 305 mm (12.0 in.), respectively. In addition to the girder, a 203-mm (8.0-in.) thick portion of the deck concrete directly above the top girder flange was present during testing. Figure 2 shows the cross sectional dimensions of the girder including the portion of the deck concrete above the top flange.

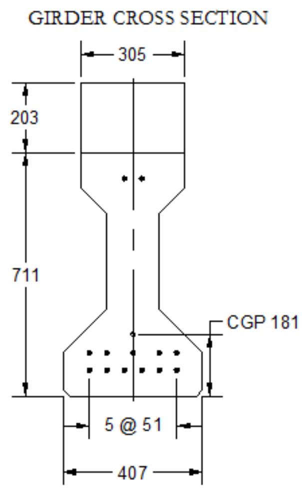

Fig. 2 Cross Section of Bridge and Girder

The reinforcement in the girders consisted of high strength, low relaxation prestressed strands and mild reinforcing. There were 13 prestressing strands in each girder with the centroid of the prestressing (c_{gp}) at 181 mm (7.13 in.) from the bottom of the girder. The prestressing force was obtained using 13 mm (1/2 in.) diameter low relaxation strands that had an ultimate tensile strength of 1860 kPa (270 ksi). The Grade 414 MPa (60 ksi) mild steel consisted of #16 M (#5) bars longitudinally in the deck as well as for vertical shear stirrups. The shear stirrups were designed using three spacing distances that were symmetrical about the center of the girder (Figure 3). The first two stirrups were placed at 51 and 153 mm (2 and 6 in.) from the end of the girder, respectively. The next eight stirrups were placed at 152 mm (6 in.) o.c. followed by an additional seven stirrups at a spacing of 305 mm (12 in.). Finally four stirrups were placed at a spacing of 457 mm (18 in.). The shear reinforcement extended into the deck concrete to enable composite action between the deck and girder concrete.

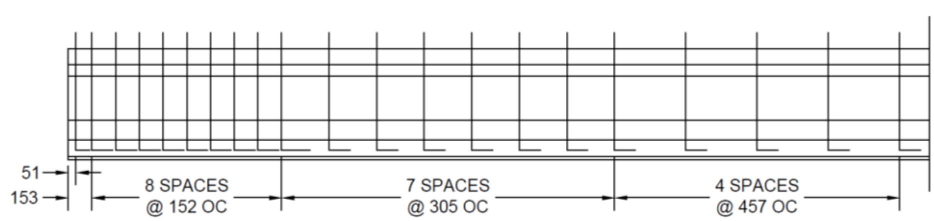

Fig. 3 Mild Shear Reinforcing Layout and Spacing

The girder concrete was developed as a high strength, self-consolidating mixture. Concrete cylinder compression tests were performed on cored specimens for both the girder and the deck concrete to obtain actual compressive strengths at the time of testing. The deck concrete (three cores) had a measured compressive strength of 55.2 MPa (8.0 ksi) and the girder concrete (four cores) had a strength of 77.9 MPa (11.3 ksi).

CRACKING MOMENT TESTING

Cracking tests were performed on each of the four girders in order to determine the effective prestress force. The cracking tests were performed by positioning the girders under a reaction frame and applying a load of 222 kN (50 kip) at the mid span of the simply supported girders. This load was intended to produce a visible transverse crack. The crack was marked and the load was removed. Two 120 Ohm foil strain gauges, that were 70 mm (2.75 in.) long, were attached to the bottom flange over the closed crack. After the gauge was securely attached to the girder the load was then reapplied and increased by 25% of the initial magnitude to ensure the crack fully opened. During the second loading the changes in load and strain were recorded at a sampling rate of 10 Hz. This same process was repeated for each girder. Decompression moment for each of the girders was determined according to procedure outlined by Pessiki et al[17]. Using this experimental decompression moment and other known loads, prestressing force was back calculated using basic elastic mechanics (i.e., P/A and My/I).

The individual and average calculated value of the residual prestressing stress from the cracking moment tests of Girders G1 through G4 are presented in Table 1. The average value of residual prestress was 1090 MPa (158 ksi). The residual prestressing in all the girders was consistent with a variation of only +/-2% from the average to the maximum or minimum values. This residual prestress represents a loss of 22% from the initial jacking stress of 1400 MPa (202.5 ksi).

Table 1 Comparison of Residual Prestress

Cracking Moment	Measured Residual Prestress	% from Test Average
Girder 1	1100 MPa (159 ksi)	-0.6%
Girder 2	1110 MPa (161 ksi)	-1.9%
Girder 3	1070 MPa (155 ksi)	+1.9%
Girder 4	1080 MPa (157 ksi)	+0.6%
Girder Average	1090 MPa (158 ksi)	-

FLEXURE TESTING

Flexure capacity testing was performed at the mid-span of Girder G1. The flexure capacity test setup consisted of a single applied point load on the deck concrete at the midspan with a hydraulic ram. Typically flexural testing is performed with two point loads near midspan (four point bending) to create a constant moment region, however, the single point load at midspan was used in order to preserve the ends of the girders for additional testing at the girder ends because of the short girder lengths. A load cell was positioned between the hydraulic ram and the top of the girder with spherical bearings placed on either side of the load cell so as to ensure the application of a vertical load. String pots were used to measure the magnitude of deflection directly under the applied load in addition to third points along the span.

During the test, the load was monotonically increased until failure; in this case exhibited by the concrete crushing and the rebar buckling in the compression block. Figure 4 shows a schematic of the general test setup. Flexural testing had α = 0.50 and β = 1.00.

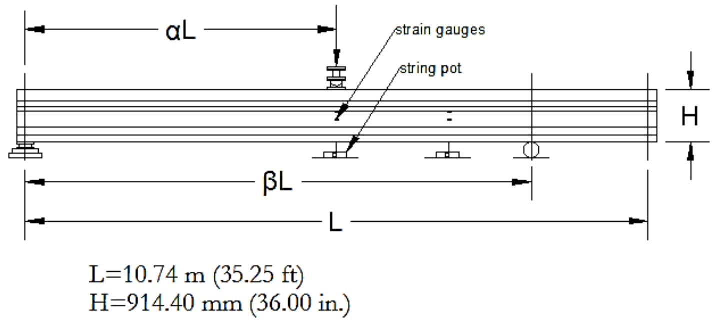

Fig. 4 General Loading Schematic

During testing, the girder initially responded in a composite, linear elastic manner prior to cracking. Significant visible cracking occurred at a load of 333 kN (75 kip) which corresponds to a moment of 890 kN-m (660 kip-ft). After cracking, the girder stiffness decreased as evident with an increase in deflection for a given applied load. The cracking initiated directly under the midspan load but increased in quantity and width until an ultimate moment of 1590 kN-m (1170 kip-ft) was achieved. At the max moment, failure occurred as the deck concrete crushed followed by buckling of the longitudinal mild reinforcing steel in the compression block. There was no indication that material deterioration or fatigue influenced the ultimate capacity. Figure 5 shows the measured moment-deflection relationship of Girder G1 through failure. For Figure 5, the curve is normalized by dividing the measured moment and deflection at a given load to the maximum moment and deflection achieved during testing 1590 kN-m (1170 kip-ft) and the deflection at the maximum moment of 50 mm (1.99 in.), respectively. Figure 6 presents girder failure.

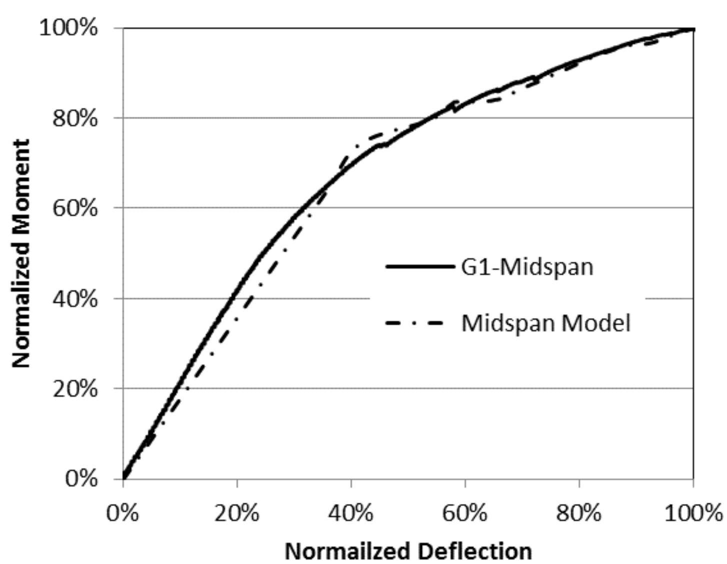

Fig. 5 Normalized Moment vs. Deflection for Flexural Test of Girder 1


Fig. 6 Girder 1 Midspan Test, Flexural Failure

1H TEST – SHEAR CAPACITY

In addition to the flexural test, a series of tests were performed to determine the girder shear capacity at various load locations. Two load tests, at a location of 1H from the support, were performed on Girder 2. For these tests, the value of H was the total height of the girder including the deck concrete 0.91 m (36 in.). The first test was titled G2-1HA where the "A" indicates the initial test on that girder, where $\alpha = 0.09$ and $\beta = 1.00$. After the girder was tested at the initial end, the support location was adjusted and the other end was tested in the same manner. For Girder G2, this second test was titled G2-1HB. For the G2-1HB test, the support at the tested end was adjusted so that $\alpha = 0.09$ and $\beta = 0.86$. For all tests, a string pot was used to measure changes in deflection directly under the applied load. For both tests (G2-1HA & G2-1HB) the load was increased monotonically through failure. Figure 7 shows the normalized shear-deflection relationship for the 1H tests. The tests were normalized to the maximum measured shears of 1770 kN (398 kip) and 1670 kN (375 kip) for tests G2-1HA and G2-1HB, respectively. The mid span deflection was also normalized. The recorded deflection at the time of maximum shear was 16 mm (0.64 in.) for each test. Figure 7 shows that for both tests, the stiffness of each test on Girder G2 was nearly constant until approximately 80% of the maximum load was obtained at which point the girder stiffness decreased by 70% and remained nearly constant until the maximum load was obtained. The failure mechanism for both of the 1H tests was brittle with the sudden widening of a diagonal crack at an approximate 45 degree angle from the support to the applied load as shown in Figure 8. This sudden failure was coupled with a substantial loss of load carrying capacity.

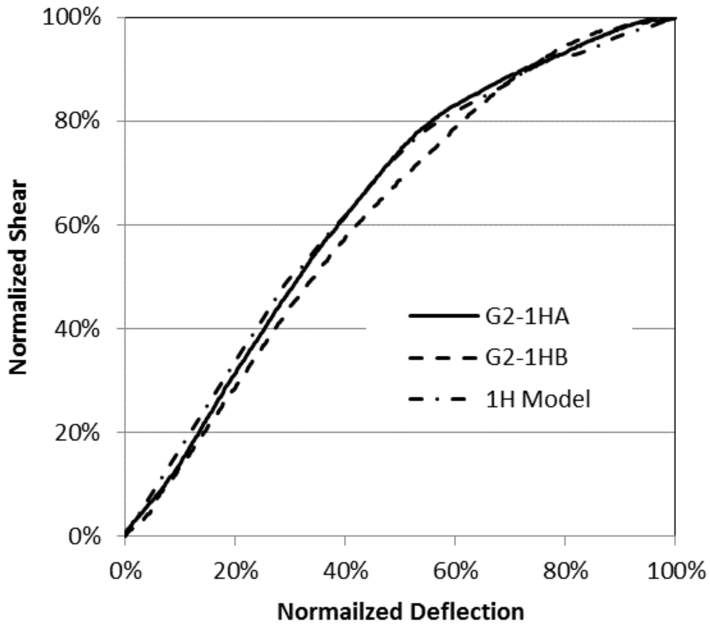

Fig. 7 Normalized Shear vs. Deflection for the 1H Test of Girder 2

Fig. 8 Failure Cracking the 1H Test of Girder 2

2H & 4H TESTS – COMBINED SHEAR/FLEXURE CAPACITY

In order to investigate the influence of load location on girder shear capacity, tests were also performed where the applied loads were placed at distances of 2H and 4H from the support. Similarly as the 1H tests, Girders G3 and G4 also had two tests (A and B) conducted on them; but with different load locations on each end. For example, Girder 3 had a 4H test performed on one end followed by a 2H test on the other end. These two tests were labeled G3-4HA and G3-2HB. For the G3-4HA test α was equal to 0.34 and β was equal to 1.0. For the G3-2HB test, the support at the tested end was moved in and α was 0.17 and β was 0.62. Girder 4 had the same two tests performed on it but in opposite order and therefore the Girder 4 tests are G4-2HA (α = 0.17 and β = 1.0) and G4-4HB (α = 0.34 and β = 0.79). Each of these tests were performed similarly as the 1H test with the load applied monotonically and a string pot was used to measure the change in displacement under the applied load.

Figure 9 shows the normalized shear-deflection relationship for the 2H tests. The curves were normalized in a similar manner as the 1H tests. For both tests, a bilinear load-deflection curve was observed. The initial stiffness of each girder was nearly identical. This initial stiffness remained constant until approximately 65% of the maximum load. The maximum shear capacity of the 2H tests were 987 kN (222 kip) and 915 kN (206 kip) for tests (A & B), respectively. This resulted in an average shear capacity of 951 kN (214 kip) for both 2H tests.

During testing, cracks became visible at an applied shear of approximately 448 kN (100 kip). Cracking initiated directly under the applied load and subsequently began to propagate along the length of the girder from the area of initial cracking. The cracking on the long side of the girder resembled the same pattern as was seen in the mid-span flexure test. The cracking pattern on the short side of the girder was inclined cracking that stopped propagating when it reached the support. After reaching the support, the crack widths increased with an increase in applied load. The cracks were also spaced about 76 mm (3.0 in.) apart. The cracks continued to widen until the maximum load was achieved at which point the concrete at the top girder flange crushed next to the applied load and buckling of the reinforcing steel occurred.

The shear versus deflection relationships for the 4H tests, presented in Figure 10, also showed a bilinear response with the change in stiffness occurring at approximately 70% of the failure load. The 4H tests had similar cracking patterns as the 2H tests with the exception that the cracking on the shorter side of the girder did not extend to the support. Visible cracks appeared at an applied shear of approximately 224 kN (50 kip). Significant cracking occurred at an applied shear of 328 kN (74 kip), which correlated with the reduction in stiffness seen in Figure 10. The failures of the 4H tests consisted of the formation of diagonal cracks propagating through the web and eventually in to the flange. At failure, the cracks suddenly widened simultaneously with the concrete crushing and reinforcing steel buckling adjacent to the point load as shown in Figure 11. The maximum recorded shear capacity for the 4H tests was 505 kN (113 kip) and 472 kN (106 kip) for tests (A & B), respectively. This resulted in an average shear capacity of 489 kN (110 kip).

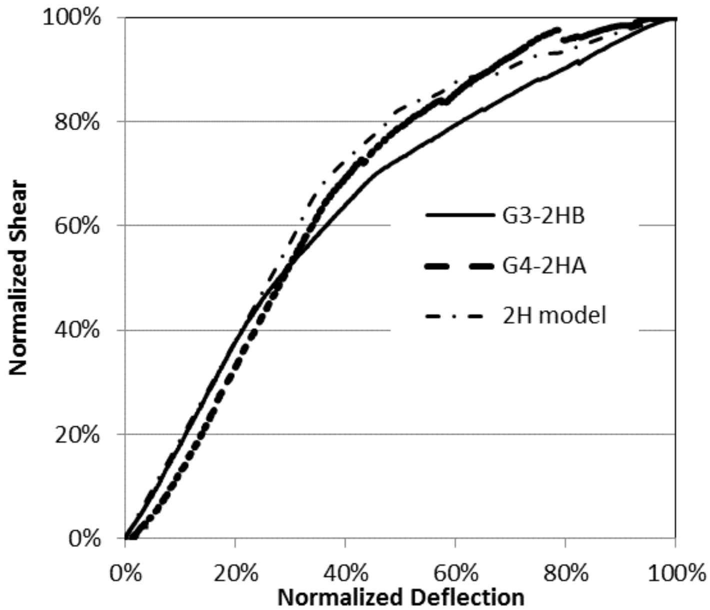

Fig. 9 Normalized Shear vs. Deflection for 2H tests of Girders G3 & G4

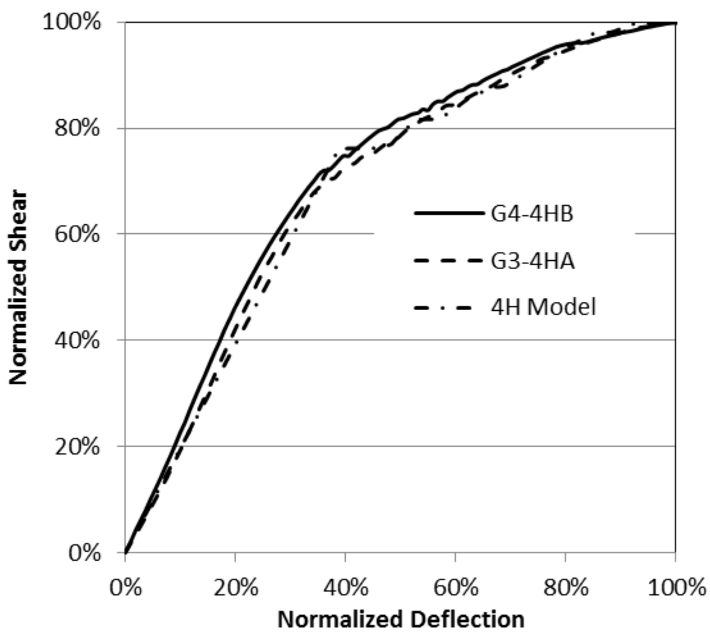

Fig. 10 Normalized Shear vs. Deflection for 4H tests of Girders G3 & G4


Fig. 11 G4-4HB Crushing near Applied Load

A summary of the measured capacities for each test of the four girders is presented in Table 2. The data shows that the recorded shear capacity decreases with an increase in distance from the support of the applied load. The largest capacity (at a load location of 1H) exhibited a nearly pure shear failure. As the load location was increased to 2H and 4H, flexural characteristics became more prevalent at failure presumably due to the testing with the shear-flexure region of the girder. While the failure of the 4H test was the most similar to the midspan flexure test, the sudden and brittle nature of the failure showed that shear significantly contributed to the ultimate failure capacity.

Table 2 Summary of Experimental Results

Test Designation	**Applied Moment kN-m (kip-ft)**	**Applied Shear kN (kip)**
G1-Midspan	1590 (1170)	296 (194)
G2-1HA	1620 (1190)	1770 (398)
G2-1HB	1530 (1130)	1670 (375)
Average 1H	**1570 (1160)**	**1720 (386)**
G4-2HA	1810 (1331)	987 (222)
G3-2HB	1673 (1230)	915 (206)
Average 2H	**1740 (1280)**	**951 (214)**
G3-4HA	1730 (1280)	505 (113)
G4-4HB	1850 (1360)	472 (106)
Average 4H	**1790 (1320)**	**489 (110)**

AASHTO LRFD PRESTRESS LOSS COMPARISON

Two methods were used from AASHTO LRFD Specifications (2012) to calculate the predicted prestress losses. The methods were the Approximate and the Refined Method. The Approximate Method is presented with Equation 2. This method uses a lump sum equation to group all the long-term losses (Δf_{pLT}). The approximate long-term loss equation is a function of annual average ambient relative humidity, concrete strength and strand relaxation. These long-term losses are added to the instantaneous elastic losses (Δf_{ES}) that occur at transfer to obtain total losses.

$$\Delta f_{pT} = \Delta f_{ES} + \Delta f_{pLT} \qquad \text{Eq. 2}$$

Where Δf_{pT} = total prestress losses, Δf_{ES} = elastic losses at transfer and Δf_{pLT} = long term losses. The refined method also uses the same general form as Equation 2 to calculate total losses; however, the individual components of the long-term losses are calculated in more detail. Overall, the long-term losses are divided into different stages of losses that occur before and after deck placement (Equation 3).

For the Refined Method, the long-term losses for shrinkage and creep of the girder concrete, relaxation of the prestressing strand and differential shrinkage of the deck and girder concrete are calculated independently. Each of these variables can be calculated as a function of time. The subscript *id* are the losses associated before deck placement and conversely the subscript *df* are the losses after deck placement.

$$\Delta f_{pLT} = \left(\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1}\right)_{id} + \left(\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} + \Delta f_{pss}\right)_{df} \qquad \text{Eq. 3}$$

Where Δf_{pSR} = prestress losses due to the shrinkage of the girder between transfer and deck placement, Δf_{pCR} = prestress losses due to creep of girder concrete between transfer and deck placement, Δf_{pR1} = prestress losses due to relaxation of prestress tendons between transfer and deck placement, Δf_{pSD} = prestress losses due to shrinkage of the girder after deck placement, Δf_{pCD} = prestress losses due to creep of girder after deck placement, Δf_{pR2} = prestress losses due to relaxation of prestress tendons after deck placement and Δf_{pss} = prestress gain due to differential shrinkage of deck and girder concrete. The two terms for relaxation (Δf_{pR1} and Δf_{pR2}) in Equation 3 sum to the same magnitude of 17 MPa (2.4 ksi) as in the Approximate Method for long-term losses. Elastic shortening loss is the same for both methods.

Table 1 lists the experimentally measured and AASHTO LRFD calculated residual prestress. Table 3 lists the approximate and refined AASHTO methods. The residual prestress was determined by subtracting the calculated losses from an assumed initial jacking stress 1400 MPa (202.5 ksi). In general, both the AASHTO Approximate and Refined Method accurately predicted the measured residual prestress. The AASHTO Approximate Method calculated a residual prestress of 1060 MPa (154 ksi), which was 3.8% conservative (positive) when compared to the average measured results. The Refined Method calculated a residual prestress of 1100 MPa (160 ksi) which slightly under predicted the magnitude of prestress loss by 1.3% (negative) from the measured results.

Table 3 Calculated Approximate and Refined Losses

AASHTO LRFD Prediction		Calculated Effective Prestress MPa (ksi)	% from Test Average
Elastic Shortening	Δf_{pEs}	+92.4 (+13.4)	-
Approximate Method	Δf_{pLT}	242 (35.1)	-
	f_{pe}	1060 (154)	+3.8%
Refined Method	Δf_{pSR}	+60.0 (+8.7)	-
	Δf_{pCR}	+107 (+15.4)	-
	Δf_{pR1}	+8.27 (+1.2)	-
	Δf_{pSD}	+42.7 (+6.2)	-
	Δf_{pCD}	+13.1 (+1.9)	-
	Δf_{pR2}	+8.27 (+1.2)	-
	Δf_{pss}	-38.7 (-5.6)	-
	f_{pe}	1100 (160)	-1.3%

AASHRO LRFD MOMENT CAPACITY COMPARISON

The measured girder flexural capacity was compared with the calculated capacity in accordance to the procedures in the AASHTO LRFD Specifications (2012). This comparison was performed in order to determine the accuracy of code recommended procedures for high-strength concrete. For the calculated capacity of prestressed concrete girders, the AASHTO LRFD Specifications provides Equation 4 for determining the nominal moment capacity.

$$M_n = A_{ps}f_{ps}\left(d_p - {}^{a}/_{2}\right) + A_s f_s\left(d_s - {}^{a}/_{2}\right) - A'_s f'_s\left(d'_s - {}^{a}/_{2}\right) + 0.85 f'_c (b - b_w) h_f \left({}^{a}/_{2} - {}^{h_f}/_{2}\right) \quad \text{Eq. 4}$$

Where M_n = nominal moment capacity; A_{ps}= total cross sectional area of prestressing steel; f_{ps}= stress in prestressing steel at failure; A_s= total cross sectional area of tensile mild steel reinforcement; f_s= stress in tensile mild steel; A'_s= total cross sectional area of compression mild steel; f'_s= stress in compression mild steel; f'_c=concrete compressive stress in compression block; d_p= distance from top of compression block to centroid of the prestressing steel; d_s= distance from top of compression block to centroid of mild tensile steel; d'_s= distance from top of compression block to centroid of mild compression steel; b= compression flange width; h_f = depth of compression flange; a= depth of effective concrete compressive stress from top of compression block. For this research the depth of the compression concrete block did not extend into the girder and therefore the concrete compressive stress (f'_c) was for the deck concrete only.

The calculated nominal moment capacity of the bridge girders was 1480 kN-m (1090 kip-ft). The nominal moment capacity was compared to the measured mid span test capacity for Girder 1. The recorded capacity of the mid-span test was 1590 kN-m (1170 kip-ft) which is 7.3% conservative in comparison to the calculated AASHTO LRFD nominal moment capacity. This conservatism is due to several factors including strain hardening of steel, the actual stress in the prestressing strand at failure and concrete material properties.

AASHRO LRFD SHEAR CAPACITY COMPARISON

The shear capacity using the AASTHO LRFD General Method for Prestressed and Nonprestressed Members were calculated for all three load locations (1H, 2H and 4H). The General Method provides relationships to calculate whether the member failure is caused by web cracking or a combination of shear and flexural cracking. The general nominal shear capacity in accordance to this method is listed as Equation 5. This method permits the total nominal capacity to be taken as the summation of the concrete, mild steel and prestressing steel contributions.

$$V_n = min \begin{Bmatrix} V_{cw} \\ V_{ci} \end{Bmatrix} + V_s + V_p \qquad \text{Eq. 5}$$

Where V_n = nominal shear capacity; V_{cw} = shear resistance provided by concrete when inclined cracking occurs from excessive principal tension forces in the web; V_{ci} = Shear resistance provided by concrete when inclined cracking occurs from combined shear and flexural forces; V_s = Shear resistance due to the mild steel reinforcing; V_p = Shear resistance due to the component of prestressing in the direction of applied shear. For these particular bridge girders the prestressing shear contribution was zero because the prestressing strands were straight along the entire girder length. The smaller of the two concrete shear resistances (V_{cw} or V_{ci}) is used in the calculation of the nominal shear resistance (V_n).

The average magnitude of measured shear capacities (V_i) are provided in Table 4. Also provided in Table 4 are the calculated concrete shear capacities and the mild steel shear capacity. For the 1H and 2H tests the concrete capacity was governed by the web shear cracking term (V_{cw}). In contrast, the concrete capacity for the 4H shear test was governed by the flexure-shear case. The shear resistances for the 2H and 4H tests were 944 kN (212 kip) and 483 kN (108 kip) respectively. The calculated shear capacity of the 2H and 4H tests were 0.9% and 1.9% (Table 4) conservative as compared to the measured values, respectively. The comparison showed a close conservative correlation between the measured and predicted values using the AASHTO Simplified Method.

Table 4 Comparison of Calculated to Experimental Shear Capacity

Test	V_{ci} kN (kip)	V_{cw} kN (kip)	V_s kN (kip)	V_n kN (kip)	V_i kN (kip)	% Diff.
1H	935 (210)	**308 (69)**	**1240 (277)**	1540 (347)	1720 (386)	+11.2%
2H	482 (108)	**315 (71)**	**629 (141)**	944 (212)	951 (214)	+0.9%

4H	**245 (55)**	325 (73)	**236 (53)**	483 (108)	489 (110)	+1.9%

The AASHTO LRFD nominal shear capacity in accordance to the simplified method for the 1H test was 1540 kN (347 kip) which was 11.2% conservative (Table 4). This value is reasonably accurate; however due to the closeness of the applied load to the support a Strut-and-Tie model may be more appropriate to calculate capacity.

FINITE ELEMENT MODEL

The finite element analysis program ANSYS[18] was used to model the girders because of its nonlinear modeling capabilities. The element types were the LINK 8, SOLID 45 and SOLID 65 elements which were used to model the mild reinforcing steel, prestressing steel, deck concrete and girder concrete. These elements were assigned measured material properties to replicate the actual girder materials.

The LINK 8 element was used to explicitly model the prestressing steel in the girders. The Link 8 elements were assigned an initial strain to simulate the residual prestressing force based on the cracking test results. The individual elements were also assigned the cross sectional area equal to that of a strand. The measured material properties of the prestressing strands were used to construct the bilinear stress-strain relationship.

The SOLID 65 element was selected to model the deck and girder concrete materials. The SOLID 65 element has the ability to crush in compression and crack in tension. In addition, smeared reinforcing can be applied to this element in multiple orientations. The smeared reinforcing is defined as a ratio of reinforcing-to-concrete by volume. The stress-strain relationship for the concrete materials was developed using the measured material properties. The deck concrete had longitudinal reinforcing smeared throughout with a reinforcing-to-concrete ratio of 0.013 in two layers. The deck concrete was modeled using measured material properties for compression and a rupture stress of 5.5 MPa (800 psi).

The vertical stirrups for the girder concrete were also modeled using smeared reinforcing. Because the stirrup spacing varied along the length of the girder, three different regions of reinforcing-to-concrete ratios were applied. The ratios used were 0.026, 0.013 and 0.005 which correlated to the 152 mm (6 in.), 305 mm (12 in.) and 457 mm (18 in.) stirrup spacing, respectively. The rupture strength for the girder concrete was 6.9 MPa (1.0 ksi) and the measured concrete compressive strength was applied. The shear transfer coefficients for the deck and girder concrete for an open crack and a closed crack were 0.3 and 0.6, respectively. To model the test support conditions, the SOLID 45 element was used to replicate the bearing plates. This element has the ability to model mild steel with behavior such as plasticity, strain hardening and large deflections[18]. The SOLID 45 element was assigned a yield stress of 344 MPa (50 ksi) and a modulus of elasticity of 200 GPa (29000 ksi).

A single girder model was developed and used for all of the load cases. The only difference between the models for each test was the location and magnitude of the applied load. The model analytical results were compared to the respective measured results in terms of ultimate capacity and deflection. Figures 5, 7, 9 and 10 show a comparison between the measured and analytical response. A coefficient of correlation (R^2) was calculated for each comparison and was used to compare the analytical load versus deflection curve to the measured results in order to quantify the comparison. Figure 12 shows and example comparison to the ANSYS crack predictions and the crack patters from selected test, indicating reasonable agreement. Additional comparisons to observed and predicted cracks and stress contours can be found Reference 19.

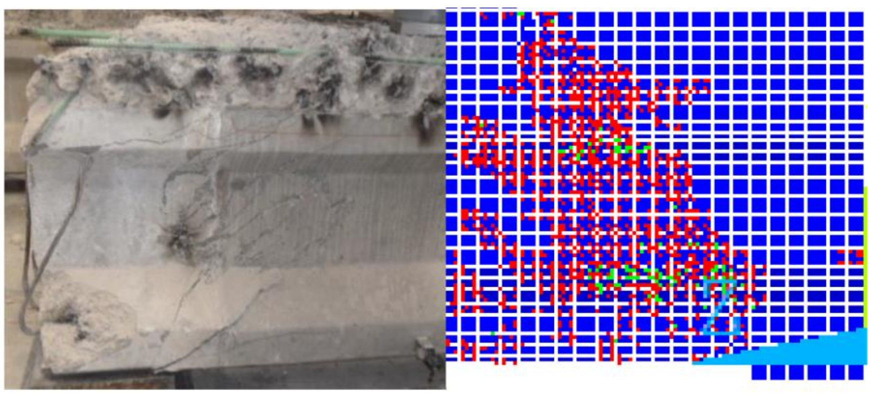

Fig. 12 G2-1HA Observed Cracking (left) and ANSYS Predicted Cracks (right)

In general, the model is slightly stiffer than the actual girder for a load placement near the support. Conversely, the model became less stiff than the tested girder as the load was applied towards the middle of the girder. These differences in stiffness can be seen in Figures 6 through 9. However, overall the model was considered an excellent match with calculated R^2 values for the 1H, 2H, 4H and mid-span tests of 0.95, 0.94, 0.98 and 0.98, respectively. The finite-element model also had ultimate capacities near the measured capacities of the girders. The 1H model reached an ultimate capacity of 1640 kN (370 kip) of shear prior to failure which was 4.6% less than the recorded magnitude. The 2H model reached an ultimate shear capacity of 916 kN (206 kip) which was 3.7% less than the measured capacity. The 4H model attained an ultimate shear strength of 435 kN (98 kip) which was 10.9% less than measured results. The midspan load case achieved a maximum flexure capacity of 1560 kN-m (1150 kip-ft) which is 1.7 % less than the experimental result.

CONCLUSIONS

Four decommissioned bridge girders made with 77.2 MPa (11.2 ksi) self-consolidating concrete were tested to quantify the residual prestress, flexural and shear capacity. Cracking tests were performed on all four girders to determine the residual prestress force. Girder 1 was tested to obtain it's flexural capacity by increasing a load at midspan until failure occurred. Each end of Girders 2 through 4 was tested in shear so that two tests were performed at locations of 1H, 2H and 4H from the center of the support. The measured residual prestress, flexural capacity and shear capacities were compared with calculated capacities in accordance to the appropriate AASHTO LRFD Specification design procedure. In addition, a nonlinear finite-element analysis was conducted using ANSYS. The analytical deflections, capacities and cracking from the finite element model were compared with the experimental results. The following conclusions were obtained based on the research results:

- The residual prestress from the cracking tests ranged from 1070 to 1110 MPa (155 to 161 ksi) with and average value of 1090 MPa (158 ksi). This average value represents a total prestress loss of 22%. The Approximate and Refined Methods in the AASHTO LRFD Specifications predicted values of 1060 MPa (154 ksi) and 1100 MPa (160 ksi), respectively. The Approximate and Refined Methods were 3.8% conservative and 1.3% unconservative, respectively.

- The measured flexural capacity of Girder 1 was 1590 kN-m (1170 kip-ft). The mode of failure was crushing with the deck concrete followed by buckling of the longitudinal deck reinforcement. The calculated AASHTO LRFD nominal moment capacity for the bridge girders was 1480 kN-m (1090 kip-ft) which is 7.3% conservative.

- Girder 2 had an average shear capacity of 1720 kN (386 kips) when the load was applied at a distance of 1H from the support. The predicted capacity of the AASHTO LRFD General Procedure was 1540 kN (347 kips) which was 11.2% conservative.

- When the load was placed at 2H and 4H from the support (Girders 3 and 4) the nominal shear capacity was 951 kN (214 kips) and 489 kN (110 kips), respectively. The AASHTO General Procedure had calculated capacities of 944 kN (212 kips) and 483 kN (108 kips), respectively. For these two load locations the calculated capacities were within 1 and 2%, respectively.

- Overall, for these four full-scale specimens and using measured material properties, the provisions in the AASHTO LRFD Specifications accurately predicted the measured prestress losses, flexural capacity and shear capacity at various locations from the support for HSC SCC.

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