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# Ultimate and fatigue response of shear dominated full-scale pretensioned concrete box girders

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**Abstract.** Two full-scale, precast, pretensioned box girders were subjected to shear-dominated loading, one under monotonic loads to failure and the other subjected to one-half million cycles of fatigue loads followed by monotonic ultimate loads. The number of cycles was selected to allow for comparison with previous research. The fatigue loads were applied in combination with occasional overloads. In the present study, fatigue loading reduced the shear capacity by only six percent compared to the capacity under monotonic loading. However, previous research on flexure-dominated girders subjected to the same number of repeated loads showed that fatigue loading changed the mode of failure from flexure to shear/flexure and the girder capacity dropped by 14 percent. The comparison of the measured data with calculated shear capacity from five different theoretical methods showed that the ACI code method, the compression field theory, and the modified compression field theory led to reasonable estimates of the shear strength. The truss model led to an overly conservative estimate of the capacity.

**Keywords:** analysis; box girders; failure; fatigue; full-scale; precast; prestressed; shear; testing.

## 1. Introduction

Prestressed concrete box girders are commonly used in bridge construction. Several models have been developed to estimate the shear capacity of prestressed concrete members including box sections. Many of the models are empirical and are developed based on matching results from testing of small to moderate scale specimens. The high cost of full-scale testing and the relative scarcity of large-scale testing facilities have hindered the generation of data on full-scale prestressed box girder elements. Several studies on the shear strength of other types of prestressed members have been conducted that have provided valuable data based on which shear design methods can be evaluated (Elzanaty *et al.* 1986, Kaufman and Ramirez 1988, Maruyama and Rizkalla 1988, Gregor and Collins 1995, Shahawy and Batchelor 1996, Teng *et al.* 2000).

Information on the shear strength of prestressed box girders under fatigue loading is even scarcer.

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Several studies have reported data on the shear fatigue behavior of I-girders (Hanson and Hulsbos 1965, Hanson *et al.* 1970, Price and Edwards 1971, Kreger *et al.* 1989). While the first two references indicated no appreciable effect of fatigue loading on the shear strength, the latter two reported a reduction in the shear capacity caused by repeated loads. An analytical model for deep beams calibrated with experimental data was reported in Teng *et al.* (2000). The results showed substantial reduction in the shear capacity as a result of fatigue loading. In Labia *et al.* (1997) the flexural capacity of a full-scale box girder was examined under fatigue loads. The ultimate strength test of the girder showed that fatigue loading reduced the shear capacity of the box girder and changed the mode of failure from flexure to shear/flexure.

This article presents the results of shear tests on two full-scale prestressed box girders, one under monotonic loading and the other under fatigue loading. The measured results are compared with those based on several existing shear analysis and design methods. The study was the third phase of a research project focused on the behavior of full-scale pretensioned concrete box girders. The first two phases of the study were reported in Labia *et al.* (1997) and Saiidi *et al.* (2000).

## 2. Description of the girders

Two 34.15-ft (10.41-m) long full-scale precast, pretensioned bridge box girders were tested. Each girder was one-half of a 68.3-ft (20.82-m) box beam that was removed from a viaduct in Reno, Nevada. The viaduct was a 20-span structure with a total length of 1600 ft. (488 m). Construction of the viaduct was completed in 1968.

In 1988 the viaduct was decommissioned due to severe cracks that had developed in the bearing seats and errors in construction of the cast-in-place portion of the bridge. The girders were approximately 20 years old when they were removed. It is important to note that the reason for the demolition of the viaduct was not any deficiencies in the pretensioned girders, which were the subject of this study.

The objective of the first phase of this research was to evaluate the structural behavior of bridge girders that have been in service for a number of years (Labia *et al.* 1997). Three full-length girders, identical to the girder that was cut in the present study, were used in that study. Two girders (Girders 1 and 2) were tested to destruction and both failed in flexure. An identical third girder (Girder 3) was the subject of a study in the second phase to evaluate the effectiveness of a repair method on severed strands (Saiidi *et al.* 2000). This girder was subjected to 500,000 cycles of service fatigue loading (HS20-44) prior to failure testing. It was found that fatigue loading changed the failure mode in Girder 3 to shear/flexure and reduced the girder capacity and ductility. A fourth girder was cut into two identical halves. The half beams were designated as Girders 4 and 5 and were the subject of the study presented here.

The cut ends of Girders 4 and 5 were strengthened by making the hollow ends into a solid block to facilitate load transfer at the bearing. Steel bars in three orthogonal directions were placed over a 2-ft (0.61-m) length from the cut ends prior to placing concrete. The steel quantity and details in the strengthened areas were similar to those in the existing solid ends.

Details about the viaduct and the girders are provided in Buzick and Saiidi (1999). Fig. 1 shows the cross section of the girders. The girders were of the American Association of State Highway and Transportation Officials (AASHTO) type BII-48 pretensioned with 30-0.5 in. (12.7 mm) diameter, Grade 270 ($f_{pu}$ = 1862 MPa), stress relieved strands. The stirrups in the girders were No. 3

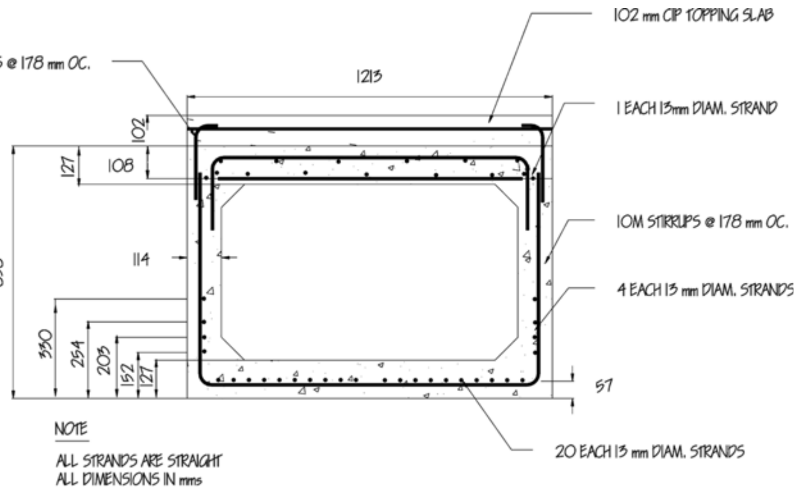


Fig. 1 Box girder cross section (1 mm = 0.0394 in.)

Gr. 40 bars spaced 7 in. (178 mm) on center. However, the original drawings specified No. 4 bars spaced at 7 in. (178 mm). Since the cross sectional area of a No. 3 bar is approximately one half of that of a No. 4 bar, the as-built shear strength of the girders was significantly reduced.

The specified initial prestress was 70 percent of the strand ultimate strength. The estimated design prestress loss was 35 ksi (241 MPa) due to the combined effect of elastic shortening, creep, shrinkage, and relaxation. The specified 28-day compressive strength of the concrete was 5,500 psi (38 MPa). The topping slab had a specified 28-day compressive strength of 3,000 psi (20.7 MPa). The bridge was designed using the 1965 AASHO Specification with HS20-44 live loading (AASHTO 1965).

### 2.1 Material properties

Several 3.75 in. (95-mm) diameter cylindrical concrete cores were extracted from the existing end blocks of the girders. An average measured concrete core compressive strength of 7890 psi (54.4 MPa) was obtained and used in the analysis of both girders. Note that the measured compressive strength was 43 percent higher than the specified value. The average concrete compressive strength in the girders tested previously was 8,400 psi (58 MPa), which is six percent higher than that measured for Girders 4 and 5. This difference was not expected to affect the comparisons that will be discussed in subsequent sections because shear and flexural properties of the girders are not sensitive to small changes in the concrete strength.

The measured and specified strength of the prestressing strands were within one percent. The difference between the specified and measured strand moduli of elasticity was three percent. Because the specified and the measured properties were very close, the specified strand properties were used in the analyses. The specified ultimate strength and modulus of elasticity were 270 ksi (1860 MPa) and 29,000 ksi (199,950 MPa), respectively.

Three No. 3 reinforcing bars (used as stirrups) were also tested. The average yield and ultimate strengths were respectively, 43.9 and 74 ksi (303 MPa and 510 MPa). Both the mild steel and prestressing strands appeared to be in good condition with no sign of corrosion.

### *2.2 Existing prestress*

The existing prestress force had been determined in previous tests (Labia *et al*. 1997) using data from a crack reopening test, strand severing, and a hanging weight test. The average measured effective prestress from these methods was 124 ksi (855 MPa) and the range was 120 to 129 ksi (827 to 889 MPa). The measured prestress force in the beams used in the present study was 130 ksi (896 MPa) and was determined using the hanging weight test.

## 3. Test setup and instrumentation

The general test setup was similar for both girders and is shown in Fig. 2. Each girder was loaded by one actuator. However, to ensure stability of the loading system the loads were applied at two points that were 7 in. (178 mm) apart. The girder was loaded in an unsymmetrical pattern to have a short shear span on one side and to ensure shear rather than a flexural failure. The critical girder shear span measured to the center of the applied loads was 13.83 ft. (4.22 m) and was considerably longer than the prestress development length of 7 ft. (2.13 m). The shear span to the effective depth ratio was 5.4 and provided ample margin against flexural failure. The monotonic load was applied using a hydraulic actuator with a capacity of 440 kips (1957 kN), whereas for fatigue loading a fatigue rated actuator with a capacity of 220 kips (1000 kN) was used.

The loading point deflection was measured using a wire transducer in the ultimate load tests and linear variable differential transformers (LVDT's) in fatigue tests. The reason for switching to an LVDT in the fatigue tests was that the wire transducer could not collect data at relatively high sample rates. A pair of LVDT's was used across the width of the girder to detect any torsion that might occur during fatigue loading. Additional LVDT's to measure the strain profile and to monitor crack opening were also installed. High resolution LVDT's were mounted across prominent shear cracks to monitor crack propagation.

Strain gages were installed on two prestressing strands at and near the loading point. Each strand was instrumented by at least six gages. In addition, ten stirrups in the left shear span were

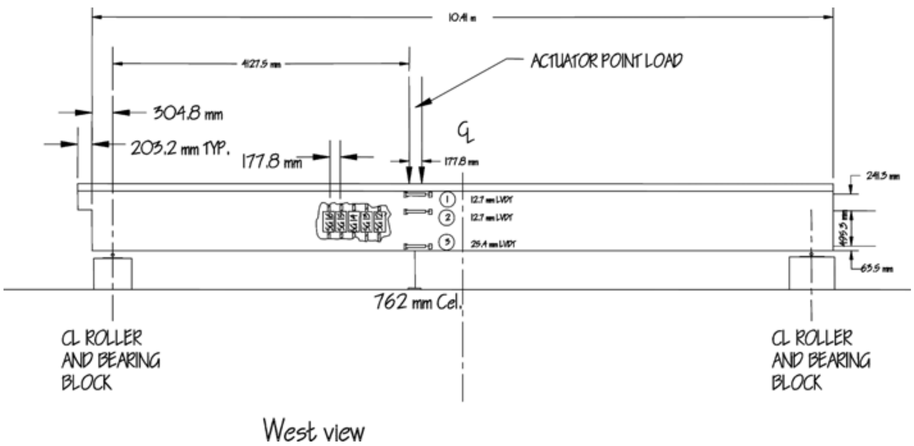


Fig. 2 General test setup for box Girder 4

instrumented with strain gages. The gages were installed at mid height and were staggered on the two side faces of each girder. Fig. 2 shows the gages on the west face of Girder 4. To access the strands and the stirrups, the surface concrete was carefully removed using a pneumatic device. The data were recorded using a “Megadac” data acquisition system at a rate of 10 samples per second.

To improve the visibility of the cracks, the girders were painted with a thin coating of white wash over the left shear span and 10 ft (3 m) to the right of the load point.

## 4. Loading program

Prior to testing, the girders were inspected to detect and mark any existing cracks. No cracks were noted. All the loadings were applied as shown in Fig. 2 with respect to the span length and load points. Girder 4 was subjected to monotonic loading until failure. The loading for Girder 5 consisted of four groups: (1) preliminary static tests, (2) daily static tests, (3) fatigue loading, and (4) the failure test. The purpose of the preliminary static tests on Girder 5 was to load the girder to slightly beyond cracking and to determine the cracking load. A diagonal crack was noted in the vicinity of the load at 192 kips (854 kN). After the first static test, the girder was unloaded and two highly sensitive LVDT’s were installed across the crack as shown in Fig. 3. Three more static tests were performed to examine the initial stability of the crack and the adequacy of data collection system.

The fatigue loading was applied over a nine-day period. Prior to fatigue loading on each day the girder was subjected to a static overload to simulate an upper bound truck load. Despite weight

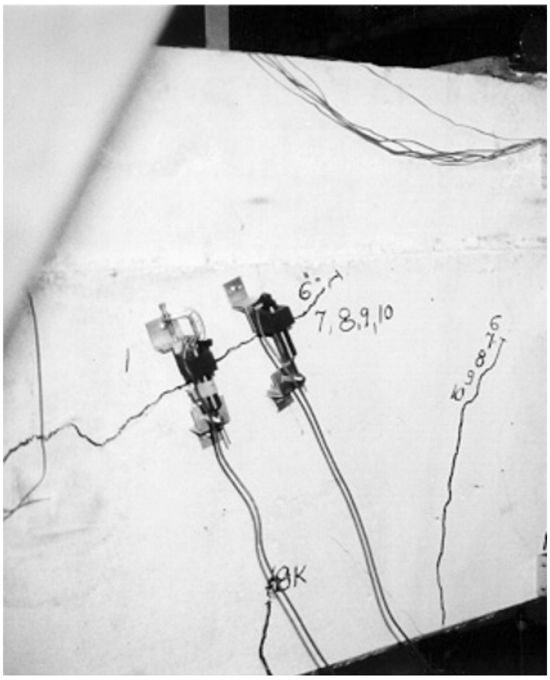


Fig. 3 LVDT’s to monitor crack width in Girder 5

Table 1 Fatigue and static overload program for Girder 5

| Day number | Daily cycle count (cycles) | Peak static overload kips (kN) | Cumulative cycles prior to static test (cycles) |
|---|---|---|---|
| 1 | 14,696 | 184.2 (819.3) | 0 |
| 2 | 57,705 | 200.0 (889.6) | 14,696 |
| 3 | 68,109 | 181.0 (804.9) | 72,401 |
| 4 | 65,975 | 185.8 (826.7) | 140,510 |
| 5 | 25,989 | 185.3 (824.1) | 206,485 |
| 6 | 72,499 | 185.3 (824.3) | 232,474 |
| 7 | 72,162 | 185.5 (825.0) | 304,973 |
| 8 | 66,739 | 185.3 (824.4) | 377,135 |
| 9 | 65,137 | 185.3 (824.3) | 443,874 |
| Total Number of Cycles | | | 509,011 |

limits placed on trucks, occasionally they exceed the limits. The overloads were included in the loading program to account for the overweight trucks. The overloads potentially inflict more damage on the girders and reduce their fatigue life below to that assumed in design codes (AASHTO 1998). The magnitude of overload was selected so that the maximum tensile fiber stress was $12\sqrt{f_c'}$ psi ($1\sqrt{f_c'}$ MPa) using the measured concrete compressive strength of 7.89 ksi (54 MPa). This stress is twice the permissible code limit on tensile fiber stress. No particular field data were used to arrive at the overload. Table 1 shows the magnitude of the overloads. Note that on day 2 the load exceeded the target value by 20 percent due to an equipment malfunction. The maximum tensile fiber stress for the load on that day was $14.4\sqrt{f_c'}$ psi ($1.2\sqrt{f_c'}$ MPa).

The objective of the test of Girder 5 was to evaluate the shear strength of the girder after it had been subjected to repeated service fatigue loads and occasional overloads. The maximum fatigue load was chosen to correspond to a calculated maximum tensile fiber stress of $6\sqrt{f_c'}$ ($0.5\sqrt{f_c'}$ MPa) set by the AASHTO code (AASHTO 1992). The fatigue loading was applied at a frequency of 1 to 1.2 Hz. This rate was considerably lower than the calculated fundamental frequency of 19.4 Hz for the girder. As a result the dynamic amplification factor was 1.004, which had a negligible effect on the response of the beam. Table 1 shows the number of loading cycles for different days. The total number of fatigue load cycles was 509,011. The number of loading cycles was approximately the same as that used in a previous fatigue study of similar girders subjected to flexural loading to facilitate comparison of the results (Saiidi *et al.* 2000). The number of loading cycles was not based on the service life expectancy of the bridge. Following the fatigue tests, Girder 5 was subjected to a monotonic load test to failure.

## 5. Test results

The most revealing measured data in the tests that were conducted were the load-deflection and load-strain relationships. In addition, the formation and propagation of the cracks were important indicators of the behavior of the girders.

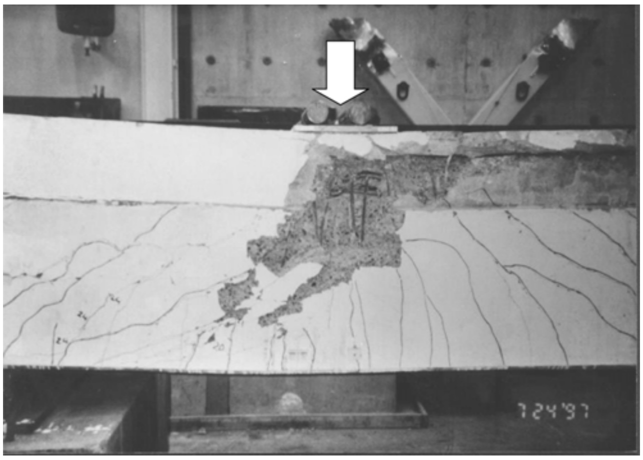


Fig. 4 Failure of Girder 4

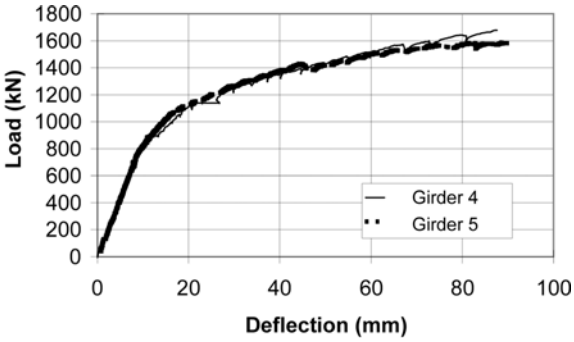


Fig. 5 Load-deflection relationships under ultimate loading

### 5.1 Performance of Girder 4

Girder 4 was subjected to monotonic loads until it failed. Cracks were marked up to a load equal to 80 percent of the estimated girder capacity. Marking of the cracks did not continue beyond this load because of safety concerns. Vertical flexural cracks were initially developed at the bottom of the girder under the load. As the test progressed, more flexural cracks were formed away from the load, but became inclined toward the load at higher forces and became essentially diagonal tension cracks as shown in Fig. 4. The angle of the inclined cracks varied from approximately 25 to 45 degrees. As loading continued more cracks formed and the existing cracks widened. The girder failed due to a combination of shear failure of the webs and crushing of concrete in the compression zone below the point load at a load of 377 kips (1,677 kN) and deflection of 3.45 in (87.6 mm). At this load the topping slab also delaminated explosively. The brittle failure of the girder was expected because it was cut in half to increase the shear level. The exposed stirrups and the delaminated slab can be seen in Fig. 4. The measured load-deflection response of Girder 4 is plotted in Fig. 5. The kink at the load of 180 to 190 kips (800 to 845 kN) is due to cracking. The girder exhibited a moderate level of ductility of 5.5 based on elasto-plastic idealization of the measured

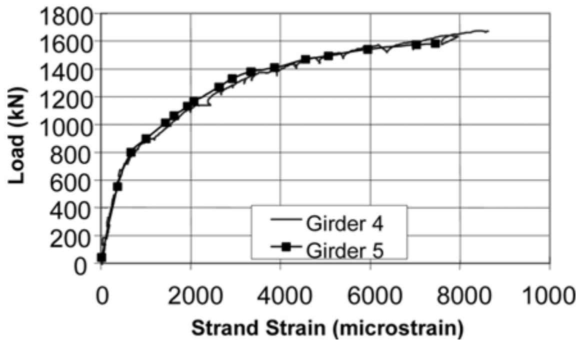


Fig. 6 Load strand strain relationships under ultimate loading

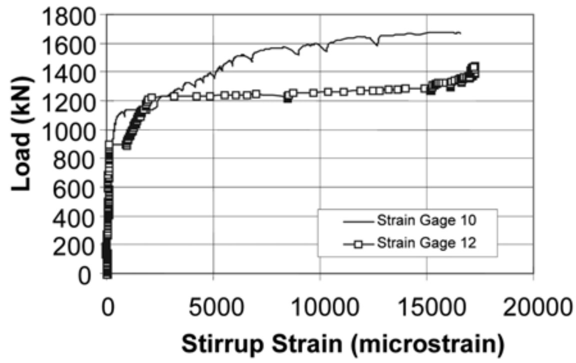


Fig. 7 Stirrup strain for Girder 4

curve. Fig. 6 shows the average measured strand strains under the load. The estimated initial strain due to the existing prestress and the dead load of the girder was 5000 microstrains, leading to a total strain at failure in excess of 13,000 microstrains. The total strain exceeded the estimated yield strain of 10,000 by a significant margin. It is hence concluded that the failure was associated with limited yielding of the strands.

The stirrup strains were well past the yield strain in the vicinity of the load. Fig. 7 shows the measured load-strain relationship for the stirrups. The measured yield strain of the stirrups was 1,510 microstrains, whereas the maximum measured strain exceeded 17,000 microstrains. The stirrups began to yield at a load of 247 kips (1,100 kN). At failure no visible sign of stirrup fracture or debonding was noted.

Following the failure test, the girder was cut into two segments and the reinforcement details were examined. All the strands and the bars had been cast according to the shop drawings with two exceptions. The size of the stirrups was smaller than that indicated on the drawings (discussed in previous sections). In addition, no dowels had been provided for the topping slab. The lack of dowels led to the explosive failure of the topping slab when the beam failed.

### *5.2 Performance of Girder 5*

Girder 5 was initially subjected to a load that cracked the beam. The loading was increased and the formation of cracks was monitored. At a load of 192 kips (854 kN) the first diagonal crack was observed as shown in Fig. 3. No flexural cracks were observed. Two LVDT's were installed across the crack to monitor its behavior under subsequent loading. The measured stirrup strains during the cracking test are shown in Fig. 8. Gages 17 and 18 were on the stirrups that crossed the diagonal crack and registered strains of over 700 microstrains. Gage 19 was on a stirrup that did not intersect the diagonal crack and showed very small strains. The measured strand strain during the cracking test peaked at 649 microstrains and dropped to 20 after unloading.

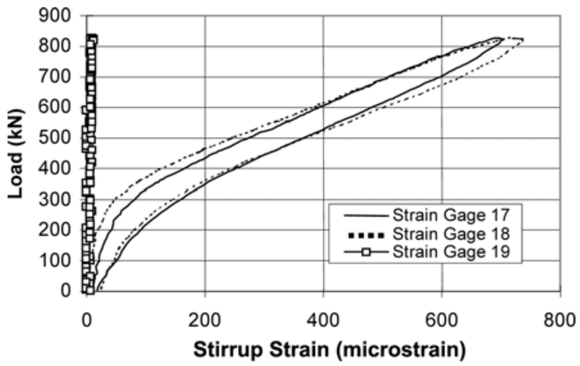


Fig. 8 Stirrup strain during cracking test of Girder 5

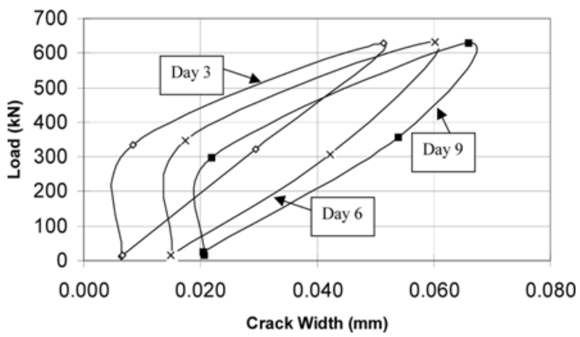


Fig. 9 Diagonal crack width variation during fatigue loading

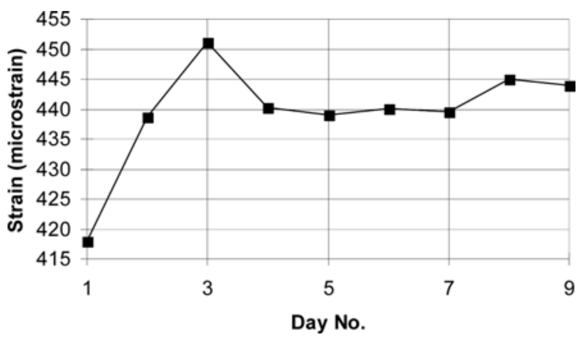


Fig. 10 Average strand strain range during fatigue loading

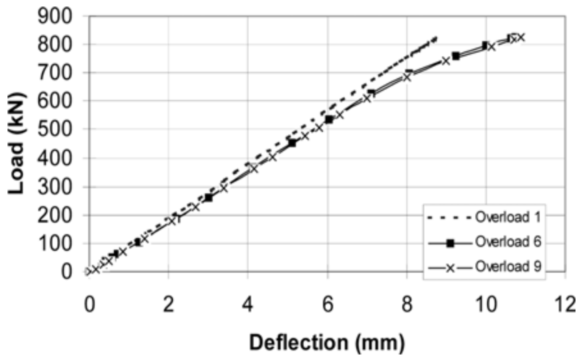


Fig. 11 Load-deflection relationships for overloads 1, 6 and 9

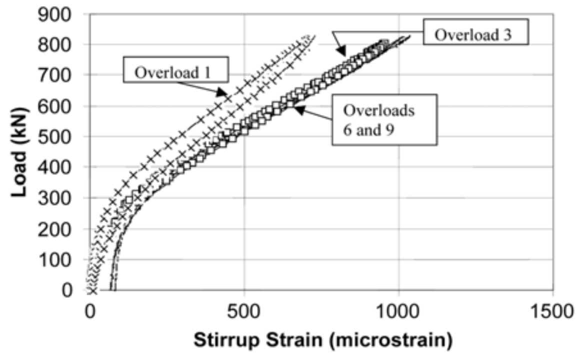


Fig. 12 Stirrup strain during overloads in Girder 5

Several flexural cracks were developed during the second static overload test. The number of flexural cracks increased during the second day of fatigue loading. The effect of fatigue loading on the width of the diagonal crack is shown in Fig. 9. The data show that there was a gradual but slight increase in the crack width in the course of fatigue loading.

The average strand strain range increased by 7 percent during each of the first three days of fatigue tests, dropped by 2 percent the next day, but remained steady afterward. This can be observed in Fig. 10. Fig. 11 shows that this trend is consistent with the load-deflection responses during different overloads. It can be noted in Fig. 11 that the stiffness remained unchanged after the initial drop.

Because shear was the primary focus of the study, the effect of fatigue loads on stirrup strains was evaluated. Fig. 12 shows the strains in the stirrups crossing the initial diagonal crack. It is evident that the strains increased from day 1 to day 3. However, fatigue loading did not lead to any appreciable changes in the peak strains afterward. A typical curve showing the variation in stirrup strains in the shear span (Fig. 13) demonstrates that fatigue loading had negligible effect on the stirrup strains in Girder 5.

At the conclusion of fatigue loading the girder was subjected to monotonically applied failure load. New diagonal cracks were developed at a load of 191 kips (850 kN). This followed by several new flexural cracks at 210 kips (934 kN). Similar to Girder 4, the girder failed in an explosive shear/flexure mode at a load of 357 kips (1,584 kN) and a deflection of 3.53 in. (89.7 mm). Fig. 14

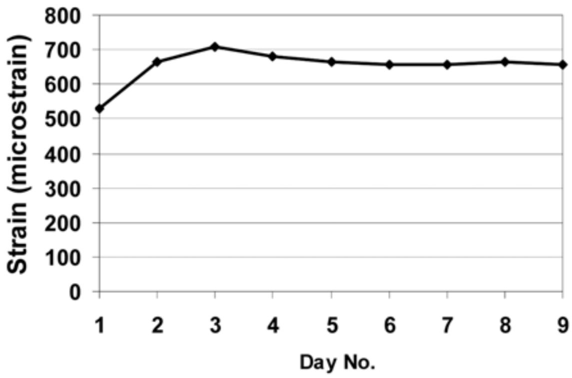


Fig. 13 Average peak stirrup strains during overloads in Girder 5

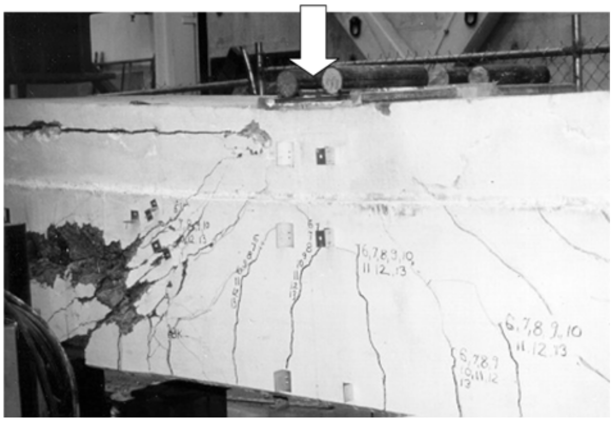

Fig. 14 Failure of Girder 5

shows the girder after failure. Note the severe shear damage. The compression failure zone on the other side of the girder (not shown) was similar to that of Girder 4 as shown in Fig. 4.

The measured load-deflection relationship for Girder 5 is shown in Fig. 5. It can be seen that girder was relatively ductile and its ultimate load behavior was very similar to that of Girder 4. The difference between the load capacities of the two girders was less than six percent and the difference between the failure displacements was less than three percent. Fig. 6 shows that the average strand strains in the two girders were also essentially the same.

## 6. Effect of fatigue loading

The data presented in the previous sections showed that more than one-half million cycles of fatigue loads in combination with overloads had negligible effect on the shear performance of the girders that were studied. The failure load in the girder was mainly controlled by shear. This conclusion does not agree with the finding of a previous study of girders of the same cross section in which one-half million cycles of fatigue loading reduced the shear strength and ductility of the

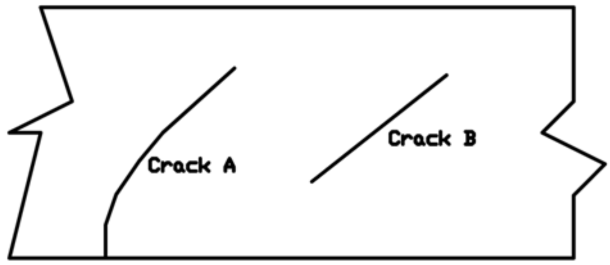


Fig. 15 Captured (Crack B) vs Uncaptured (Crack A) cracks

girders (Saiidi *et al.* 2000). As a result of fatigue loading, the girder strength in that study dropped by 14 percent and its deflection capacity dropped by 35 percent. The failure load in those girders was controlled mainly by flexure. Previous studies on the effect of fatigue on prestressed girders also have shown mixed results with no clear explanation.

The present study and the study reported in Saiidi *et al.* (2000), however, pointed toward a plausible explanation based on the data that were collected as follows: Fatigue loading can have a detrimental effect on the shear strength of concrete beams in which the primary cracks are due to flexure. Flexural cracks in these girders start at the extreme tensile fiber and become inclined towards the load. Crack A in Fig. 15 is a typical flexural crack. However, in a shear-dominated beam, mainly web shear cracks, similar to Crack B in Fig. 15, are formed. Crack A is relatively free to open at the bottom and is "uncaptured". However, Crack B is restrained by the concrete at its ends and is "captured". Under fatigue loads, crack A tends to open and close leading to some grinding of the aggregate in the crack. The repeated grinding can reduce the aggregate interlock and reduce the shear capacity. Crack B, however, is less susceptible to opening under fatigue loads and is subjected to little grinding. Because the grinding of the aggregate in the captured shear cracks is minimal, there is little shear strength loss under fatigue loads.

## 7. Analysis of shear strength

The shear capacity of the test girders was calculated using several methods and compared with the measured data. The following analytical methods and theories were used: (1) the modified truss model (Ramirez and Breen 1991), (2) the compression field theory (CFT) (Collins and Mitchell 1991), (3) the modified compression field theory (MCFT) (Collins and Mitchell 1991), and (4) the American Concrete Institute (ACI) 318 method (ACI 318-02 2002). Note that the Load Resistant Factor Design (LRFD) method of the American Association of State Highway and Transportation Officials (AASHTO 1998) uses MCFT. The measured material properties and existing prestress force were used in the analysis. Detailed background information about the analytical studies is provided in Buzick and Saiidi (1999).

Under downward loads, the shear resistance mechanism of reinforced concrete beams in the truss model is assumed to consist of a top concrete compressive chord, a bottom steel tensile chord, vertical steel stirrup elements, and inclined concrete compressive struts formed in between diagonal cracks. The latter two act as vertical and inclined truss members, respectively. In the original truss model the stresses carried by concrete across the cracks were ignored. Several models developed

Table 2 Measured and calculated shear capacities

| Analysis method | Girder 4 [Measured capacity: 217 kips (965 kN)] | | Girder 5 [Measured capacity: 205 kips (912 kN)] | |
|---|---|---|---|---|
| | Calculated Kips (kN) | Measured/ Calculated | Calculated Kips (kN) | Measured/ Calculated |
| ACI | 150 (668) | 1.44 | 150 (668) | 1.37 |
| Truss – 45 deg. | 98.7 (439) | 2.20 | 98.7 (439) | 2.08 |
| Truss – 37 deg. | 117 (520) | 1.86 | 117 (520) | 1.75 |
| CFT | 162 (722) | 1.34 | 162 (722) | 1.26 |
| MCFT | 174 (773) | 1.25 | 174 (773) | 1.18 |

subsequently accounted for the stress transfer across the cracks. The model by Ramirez and Breen (1991) were used in this study with two crack angles of 37 (the measured angle) and 45 degrees.

A more accurate estimation is included in CFT by calculating the crack angles based on principal stresses in the web. The shear capacity is determined using equilibrium, compatibility, an elasto-plastic stress-strain relationship for steel, and a parabolic stress-strain relationship for concrete (Collins and Mitchell 1991). Similar to the earlier truss models, the stress transfer across the diagonal cracks is not included in CFT. A modified version of CFT, MCFT, has been developed that includes the tensile stress transfer (Collins and Mitchell 1991). The ACI code (ACI 318-02 2002) considers two possible modes of failure in prestressed concrete beams, one assuming web shear cracks and the other flexure/shear cracks. The shear capacity for each of these modes is found using empirical equations and the smaller of the two is combined with the shear strength provided by the stirrups to obtain the total shear capacity. The shear capacity of concrete includes the effect of the prestress force. The calculated shear capacities of the girders along with the measured values are listed in Table 2. No strength reduction factors were applied to the results. Because the fatigue loading had little effect on the shear capacity of the test girders, the table includes data for both Girders 4 and 5. The results for the truss model are based on 45-degree and 37-degree crack angles, with the latter being the measured crack angle in the failure zone.

It is clear from the data that the actual shear capacity exceeded all the calculated values, and that all the methods were conservative. The truss model with a 45-degree crack angle led to perhaps an overly conservative estimate of the shear capacity, with the actual capacity being 2.2 and 2.08 times the estimated shear strength for Girders 4 and 5, respectively. The other three methods, the ACI, CFT, and MCFT led to close yet conservative correlation with the test data. The actual strength was from 18 to 44% larger than the values estimated by these methods.

## 8. Conclusions

The experimental and analytical results presented in this article led to the following conclusions:

(1) In shear-dominated prestressed precast concrete girders, fatigue loading in combination with occasional overloads does not appear to reduce the shear capacity significantly. This conclusion was reached based on 509,000 cycles of loading. However, the trend in stiffness changes, shear crack widening, and strain changes in the strands and stirrups showed that after approximately 200,000 load cycles these parameters were essentially stabilized and that

subsequent fatigue loading did not alter them significantly.

(2) The reason for insensitivity of the shear strength to fatigue loading is that web shear cracks are restrained cracks and do not open significantly under repeated loads. In contrast, flexure/shear cracks are unrestrained at the extreme tensile fiber and open under the loads. The stability of the web shear crack keeps the shear strength of concrete unchanged whereas the repeated opening and closing of a flexure/shear crack reduces the concrete shear strength.

(3) All the five analysis methods that were used provided a conservative estimate of the shear capacity of the girders, although the truss model results might be considered overly conservative. The ACI code method, the compression field theory, and the modified compression field theory all led to reasonable estimates of the shear strength.

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