BEHAVIORAL IMPROVEMENTS IN
SEGMENTAL CONCRETE BRIDGE JOINTS
THROUGH THE USE OF STEEL FIBERS

by

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B.S., Civil Engineering
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Submitted to the Department of Civil Engineering
in Partial Fulfillment of
the Requirements of the Degree of
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ABSTRACT

Joints between segments in precast concrete segmental bridges
require special attention in design and construction. These joints
introduce discontinuity in the bridge yet they must transmit large
compressive and shear stresses. Under present design practice no
continuous mild steel reinforcement is provided across the joint,
and failure of the joint could be brittle as a consequence of this
procedure.

Tests were carried out to assess the shear strength and
deformation behavior of precast segmental bridge joints utilizing
steel fiber reinforced concrete (SFRC). The key joint models were
tested in a push-off arrangement with and without epoxy (dry) under
various confinements. The confinements represented bridge
prestress levels. Straight and crimped-end fibers were
incorporated at 1% and 2% volume fractions, and both monotonic and
cyclic tests were performed. Plain concrete key joints were cast
with each mix for comparison purposes.

Although these early test results do not provide definitive
conclusions due to the limitations of the tests and test
parameters, evidence presented in this report clearly indicates
that the addition of fibers creates a stronger and more ductile
concrete which could be used advantageously in the construction of
precast concrete segmental bridges. Crimped end fibers generally
produced better strength and toughness values than the straight
fibers, but in several instances the deformed fibers did not give
a ductile behavior. Both fiber types consistently produced a
cement with better strength and ductility than plain concrete.
It was found that the shear strength increased with the volume
fraction, but the relationship between toughness and volume
fraction did not exhibit a definite relationship. In some cases
a rapid post-peak strength decline was observed for the 2% Vf
specimens. Fiber contributions were more substantial at lower
confinement levels.

Thesis supervisor: Professor Oral Buyukozturk
Title: Professor of Civil Engineering
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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND AND IMPORTANCE OF THE RESEARCH

In recent decades the precast segmental method of bridge construction has become increasingly popular. Increased speed of erection, improved aesthetics, and mitigation of environmental disturbance are all positive qualities contributing to the expanded use of this method. Additionally, the various techniques within segmental construction are adaptable to a variety of span lengths, and quality control is improved while deflections from creep and shrinkage are reduced.

Although this method provides many advantages, drawbacks arise as well. For instance, joints between the precast segments require special attention in design and construction. These joints introduce a structural discontinuity in the bridge, yet they must transmit the large compressive and shear stresses in the joint locations. Another concern is that no continuous mild reinforcement crosses the joint in precast concrete segmental bridges. This lack of steel across the joint could lead to brittle failure in the joint regions under overloads or earthquake loadings.

Despite their importance, at present, no established methodology exists for the design of joints in precast segmental bridges. The existing body of research in this area is limited, and the behavior of the joints under shear loading is not fully
understood. The Post Tensioning Institute (PTI) [56], (1988) has recently published recommendations for the design and construction of segmental concrete bridges, but the authors emphasize the need for added research in many areas including the joint regions.

Hence it is evident that continued research is necessary to evaluate the ultimate strength of the bridge joints and to understand their deformation behavior under service and ultimate loadings.

1.2 OBJECTIVES OF THE RESEARCH PROGRAM

In light of the above mentioned concerns, an extensive research project has been undertaken at the Massachusetts Institute of Technology to study the behavior of segmental bridges with particular emphasis placed on investigation of the joint regions. One important aspect of the research has been the attempt to improve joint performance through the use of steel fiber reinforced concrete.

The main objectives of the overall program were (1) to develop a better understanding of the joint behavior as affected by different design parameters, and (2) to develop design aids for evaluating the strength and stiffness of the joints [14].

The first objective was achieved largely through experimental investigation. Models of the joint region were tested under different combinations of pertinent parameters. The parameters
of primary importance were bridge prestress level and thickness or absence of epoxy bonding agent. From these experiments a comprehensive understanding of the physical behavior of the joints under shear loading was developed.

Empirical equations were proposed for the plain concrete joints which can be used to evaluate the shear strength of a section under a given normal stress distribution. These equations were based on the results of the experimental work in objective (1).

From the tests it was observed that one of the principal characteristics of the joint behavior was a brittle failure; after attaining maximum load, the capacity of the joint would quickly drop off. This observation ties in with earlier mentioned concerns that no mild reinforcement crosses the joint region. Once the concrete has cracked, only residual forces remain to carry load.

To improve bridge safety, it is desirable to provide the joints with a higher post-peak load carrying capacity. The addition of discrete fibers to the concrete mix enhances the concrete mechanical properties. These fibers limit the progression of cracking and help to transmit load across cracked surfaces.

As part of the experimental program, steel fibers were added to the concrete in an attempt to improve the joint strength and ductility. Two fiber types were investigated, and the volume fraction of these fibers was varied along 0%, 1%, and 2%. Dry
and epoxied joints were tested at various levels of confining pressure. Several cyclic tests were also performed to evaluate the properties of fiber reinforced bridge joints under repeated loadings.

1.3 ORGANIZATION

This report summarizes the experimental work that has been performed to investigate ultimate strength and deformation of precast concrete segmental bridge joints, and particular emphasis will be placed on the assessment of behavioral improvements in the bridge joints through the use of fiber reinforcement.

Report organization is as follows: following this introductory chapter, Chapter 2 will review existing literature concerning the state-of-the-art in segmental bridges and fiber reinforcement. The review will cover current segmental bridge construction technology, research on joints in the bridges, and tests on entire segmental structures. Additional topics will be tests on related structures and design recommendations of the joints as set forth by PTI. Having described segmental bridge mechanics, a description of fiber reinforced concrete will be presented so that improvements in behavior of the joints as a result of fiber addition can be appreciated. Regarding fiber reinforcement, the review will cover shear tests on beams, direct shear tests, and various modeling techniques applied to
steel fiber reinforced concrete (SFRC).

The testing program will be described in detail in Chapter 3. Outlined in this chapter will be material parameters, specimen preparation, and testing procedure.

Presentation and discussion of results will follow in Chapter 4. First the results of tests on plain concrete specimens studied under the larger research program will be outlined. Next the effects of individual fiber types will be considered followed by a comparison of their results. Additional sections describe modeling concepts which can be applied and propose methods for the application of SFRC in industry.

Chapter 5 will summarize the important results and present conclusions and directions for future research.
Chapter 2

REVIEW OF PREVIOUS WORK

2.1 PRECAST CONCRETE SEGMENTAL BRIDGES

The segmental method of concrete bridge construction originated in post War World II Europe in response to the shortages of supplies and manpower. Eugene Freyssinet was the first to use precast segmental construction for prestressed concrete bridges in France in the late 1940's. Segmental construction was also being utilized in Germany in this period; Ulrich Finsterwalder applied cast-in-place construction in the balanced cantilever method. It was not until 1962 that precast segmental construction was applied in the Choisy-le-Roi Bridge over the Seine River in Paris [29],[48],[55],[77].

Although a simple precast short span I-beam bridge utilizing longitudinal segments was constructed in New York in 1952, full-scale use of the segmental concrete construction method did not arrive in North America until 1964 with the completion of a cast-in-place structure in Quebec. The first precast segmental bridge in North America was constructed in Quebec in 1967 [29]. The first segmental concrete bridge in the United States was the JFK Memorial Causeway in Corpus Christi, TX in 1973 [31]. The use of segmental bridges in the US has expanded dramatically since, and major bridges built using the method include the
Sunshine Skyway, Houston Ship Channel Bridge, Pasco-Kennewick, and the I-205 Columbia River Bridge.

Starting from a pier or abutment, construction of segmental concrete bridges is accomplished through the placement of short, transverse sections of the box girder. After each segment has been positioned, it is secured to the existing superstructure through post-tensioning. Once the segment is securely in place, it becomes an integral part of the structure, and work on the next segment can proceed from the one that has just been placed. In this manner construction proceeds from an abutment or pier along the entire span length. Different variations exist for the provision of structural support until the span is completed. The four primary methods within segmental concrete construction are the balanced cantilever, span by span, progressive placing, and incremental launching methods, and, following a description of casting methods, these techniques will be presented.

Segments can be produced by either cast-in-place or precast procedures. Falsework or travelers are used to support forms directly on the superstructure in the cast-in-place fabrication process. After the concrete has gained sufficient strength, the segment is post-tensioned, and the forms are moved forward so that the next segment may be cast. A principal advantage derived from this method is that continuous mild reinforcement may be provided across the joint. The primary disadvantage is that casting of the next segment cannot commence until the previous segment has reached a sufficient strength [29],[55].
Precast segments are fabricated away from the bridge site in a casting yard or factory. Rapid production of high quality segments is achieved under the controlled factory manufacturing environment [48]. Completed segments are transported to the bridge site via truck or barge, and they are assembled to the bridge deck with cranes or launching gantries. Segment widths vary from 8 to 20 feet depending on the capacity of the available construction equipment.

A process known as match casting is used in current precast segmental practice. In this method the faces of adjoining segments are cast against one another at the factory, and when the segments are later reassembled at the bridge site a nearly perfect fit is obtained. Precasting is further broken down into long line and short line segment casting.

Long line casting takes place along a half or entire span length. The segments are cast along a preadjusted soffit profile, and the forms travel along the length of the casting bed. This method is not as technically demanding as short line casting, but it requires more space at the casting yard [29]. (See Figure 2.1a).

The short line casting method is shown in Figure 2.1b. In this method the formwork is stationary, and only two segments are on the casting bed at one time. The new segment is cast between a bulkhead on one side and the segment to which it will adjoin in the completed structure on the other. Horizontal and vertical curvature of the structure may be obtained through
Figure 2.1 Precast segment fabrication processes [77]
alignment of the neighboring segment before casting against it [29],[54].

In early segmental bridge construction the gap between segments was filled with mortar or concrete. Through improvements such as the match-casting process, these mortar joints are only required at the closure of a span. The bonding agent currently used between segments is usually a thin layer of epoxy. The benefits provided by epoxy are: during the construction phase it (a) facilitates placement of segments by lubricating joint regions; (b) eliminates unevenness of the joint surfaces. In the completed bridge it (c) contributes to the strength of the structure by transmitting shear stresses across the joint; (d) prevents corrosion of the tendons by forming a waterproof seal [13],[14].

Several recent bridges erected in warmer climates have used dry joints in which epoxy is absent, and some designers have questioned the value of epoxy in any climate [6],[61]. One of the concerns cited by these sources is that the transfer of stresses in epoxied sections may not be uniform because field conditions do not allow for precise application of the epoxy layer. It is also suggested that the epoxy may not in fact be effective in sealing the joint against the elements.

Construction Methods

Most common among the concrete segmental construction methods, the balanced cantilever method is typically utilized on longer
spans [35]. Construction begins at a central pier, and segments are alternately placed on either side of the pier in order to balance the moment at the pier created by the cantilevered sections (See Figure 2.2). The columns and pier caps must be designed to resist large moments during construction resulting from unbalanced cantilevers. Cast-in place or precast segments may be used, but cast-in place segments have been used for the longer spans. The Houston Ship Channel Bridge and Columbia River Crossing were built using cast in place segments in conjunction with the balanced cantilever method [25],[57]. Span lengths for this technique range from 300 to 860 ft. Examples of balanced cantilever construction which utilized precast segments are the JFK Memorial Causeway, Kishwaukee River Bridges, Twelve Mile Creek Bridges, and the Zilwaukee Bridge [9],[15],[31],[38].

In the span by span method of segmental construction, a temporary support system is erected between piers to support the concrete segments until the entire span is completed. Once the span has been completed, the support truss is moved to the next span, and construction of that span proceeds in a similar manner. This method is economical for longer, multispans bridges whose span lengths range between 100 and 300 feet. The span by span method primarily utilizes precast segments, as this combination provides for rapid span completion [6],[20],[35],[48],[55]. The implementation of the method is shown in Figure 2.3. First, segments must be constructed over
Figure 2.2  Balanced contilever construction method [77]
the piers at either end of the span. Next, the erection truss is assembled with the truss ends fixed either to the pier segments or the pier itself. The individual segments are then transferred to the bridge site, placed in the proper position, and prestressed to the existing structure. Transportation of the segments may be via truck along the existing structure, or they may be carried by barge or truck to an area beneath the span and lifted into place. Several bridges utilizing this method are the Seven Mile and Long Key bridges in Florida and the I-10 interchange in San Antonio, Texas [49],[66].

The progressive placement method is similar to the balanced cantilevered method in that the segments are cantilevered out from the pier, but in this method work progresses in only one direction. Temporary construction piers may be erected to reduce the negative moments created before the span is completed (See Figure 2.4). Due to these large moments, feasible span lengths for this method lie between 150 - 300 feet. This method is effective in areas of difficult abutment conditions as is evidenced by the Linn Cove Viaduct project in North Carolina [29].

The incremental launching method has only been utilized twice in the United States to date [35], but it has found many applications in European countries. In this method the segments are cast in forms which are usually located behind or over one of the abutments. After the segment has cured, the forms are retracted, and the entire structure is moved forward. Casting
Figure 2.3  Span by span construction method [77]

Figure 2.4  Progressive placing construction method [77]
of the next segment can begin against the face of the previously cast segment in the same forms and at the same location. A steel launching nose, which covers 30-50% of the span length, is fixed to the lead segment to reduce the free cantilever length. A schematic diagram of this method is shown in Figure 2.5. Typical span lengths with this method are from 30-50 m, but longer spans may be achieved if intermediate supports are utilized. This method is advantageous in that all of the casting work takes place in the abutment area, and a repetitive scheme is used so that peak efficiency may be obtained in casting operations. Additionally, equipment costs are typically lower for incremental launching than the other segmental methods [42],[77].

2.2 TESTS ON CONCRETE SEGMENTAL BRIDGE STRUCTURES

Results from tests on entire concrete segmental bridge structures will be presented in the following section. This review, while not exhaustive, will acquaint the reader with the behavior of a structure which is constructed entirely from precast segments. The emphasis in this review will be placed on the performance of the joints in the structures.

Bishara and Mahmoud (1972)

In reference [12] Bishara and Mahmoud tested a 110 ft. two span continuous girder. This girder was composed of three
Figure 2.5  Incremental launching construction method [29]
precast segments which were joined near the inflection points by keyed scarf connections. Scarf connections are characterized by an inclined rather than a vertical section at the joint. Prestressed keys, epoxy-sand mortar, and high tension bolts were used to secure the scarf connections. Under a loading pattern that produced moment and high shear in the scarf connections, the beam behaved as a monolithic prestressed continuous beam. Cracking in the scarf connection initiated in the concrete rather than in the epoxy mortar.

Kashima and Breen (1975)

Kashima and Breen performed an ultimate load test on a model of the JFK Memorial Bridge in Texas [31]. This "direct" model was built to one-sixth scale. It was the first model of a segmental bridge to be built and loaded to failure which accurately reflected the construction procedure. The authors conclude that the epoxy joints did not reduce the design shear strength. At ultimate load levels, however, large cracks formed along several of the joints. Under one loading configuration this led to the formation of a plastic hinge, and in another case this cracking led to the rupture of several of the prestressing tendons.

McClure and West (1984)

The findings of service and ultimate load test on a precast concrete segmental box girder bridge are reported in [1] and
The two independent girders have a simply supported span length of 121 feet, and they were tested to failure in a predominately flexural mode. The precast segments were joined with epoxy, and steel shear dowels were used for alignment and shear transfer. Apparently the joints performed well under ultimate loads as the prestressing tendons snapped at advanced loading stages. Behavior of the epoxy joints is not described in detail in this article.

Specht (1986)

The effect of the tendon bond on prestressed beam behavior was investigated in tests at the Technical University of Berlin (TUB) [67]. Seven precast segmental I beams, 4.2 m in length, were tested in 4 point bending. These tests, using grouted and ungrouted tendons, were designed to study the joint regions of segmental girders, and heavy shear reinforcement was placed in the segments' interiors to force failure at the joint regions. The critical conclusion drawn from the tests at TUB that is pertinent to this report is that for the epoxied joints, cracks at the joints were caused by the discontinuity of the steel and not by the presence of the epoxied surface. Cracks develop in the concrete rather than in the epoxy.

Rabbat and Sowlat (1987)

Rabbat and Sowlat performed ultimate bond tests on three segmental concrete girders to examine the effect of external
placement of tendons [58]. The three T-shaped girders were each composed of 22 segments which covered a 31-0 ft. span, and dry keyed joint connections (without epoxy) were provided to transfer shear forces at the joints. The bonded and modified unbonded (tendons embedded in subsequent concrete pour) tendons attained similar strengths in the elastic and early inelastic ranges, but the unbonded tendon girder was only able to achieve 75% of the moment of the other two girders. All three of the girders had a higher load bearing capacity than was predicted by AASHTO [2] and ACI formulas [3]. Under ultimate loading the bonded tendon girder displayed a much more ductile behavior; this difference was due largely to the fact that the two girders with external tendons suffered loss of bonding in the anchorage zones. It is interesting to note that, for the two girders with external tendons, the ultimate failure mode was largely due to shear, and several of the shear keys broke off at the final failure stages.

Assessment of Tests on Bridge Structures

The experiments listed in the preceding section suggest that the epoxy is effective in joining together concrete sections, and that failure of the joint usually initiates in the concrete rather than in the epoxy or the epoxy concrete interface. For the tests which utilized dry joints [58], it was observed that several of the shear keys broke off in the final failure stages. It is important to note that the tests in [31],[44],[58], and
[67] had no mild reinforcement across the joint regions, and cracking predominated in these areas.

In spite of these results which suggest that the epoxy restores the strength of a concrete section, several researchers have proposed that epoxy may give rise to stress concentrations in the joint regions [1], [16], [61]. Several failures have been reported in industry as a result of epoxy problems. The epoxy used in the Kishwaukee Bridges did not cure properly, and cracking occurred in the joint regions as a result [37]. Additionally, it is important to understand the shear capacity of the individual shear keys; they provide strength during construction before the epoxy has cured, and many of the newer bridges are being constructed with dry joints (no epoxy). It becomes even more critical to quantify the strength of shear keys if the keys have been damaged or broken off during construction as reported in [9], [59], and [74].

2.3 TESTS ON JOINTS IN CONCRETE SEGMENTAL BRIDGES

In the following two sections tests on dry and epoxied flat concrete surfaces will be presented. This information is important because it will help to evaluate the strengths contributed from each of the various components in the actual bridge joint. Tests on shear keys in concrete segmental bridges will be presented in Section 2.3.3. These tests investigate the strength of shear keys located along the height of the web.
2.3.1 SHEAR TESTS ON PLAIN CONCRETE SURFACES

Franz (1959)

Franz investigated the shear resistance between two concrete surfaces with specimens which were made of five 200 mm precast concrete cubes [21]. External prestressing was applied to the cubes, and the shear force was applied at one of the joints (See Figure 2.6). The shear strength was found to be approximately 0.7 of the applied confining force. It was also found that the shear strength was independent of the eccentricity of the normal force.

Jones (1959)

Jones performed shear tests on post-tensioned rectangular concrete beams having different types of joints such as plain butt joints, mortar joints and precracked joints [30]. For the plain butt joints, axial load was applied, and the shear load was increased until the joint slipped (see Figure 2.7). The same specimen was tested numerous times at increasing prestress levels. The average value for the coefficient of friction was 0.525 with a standard deviation of 0.066. The mortared joints were tested in a similar configuration. Load was applied until
Figure 2.6  Testing arrangement of Franz [21]

Figure 2.7  Testing arrangement of Jones [30]
initial slippage occurred, and then the test was continued to examine the frictional resistance of the joint after the bond had been broken. Coefficients of friction were then determined for both cases. For the case of the surface after the bond was broken, the average coefficient of friction was 0.691 for the mortar surface which had been cast against a concrete face. This situation more closely resembled the conditions in a match cast concrete bridge.

Gaston and Kriz (1971)

Gaston and Kriz tested the shear strength of bolted concrete connections in the arrangement shown in Figure 2.8 [22]. From the tests on dry joints they concluded that the shear resistance is a linear function of the applied normal stress, and the size of the contact area does not affect the coefficient of friction.

2.3.2 SHEAR TESTS ON EPOXYED SURFACES

Base (1963)

Base tested precast concrete beams to determine the shear strength in epoxy joints [8]. Beams were formed by epoxying three blocks together as shown in Figure 2.9. The loading was configured such that the joints were located at points of contraflexure. From the tests it appeared that failure was not initiated by the epoxy joint. The plane of failure was partly along the epoxy-concrete boundary but cracking of the concrete
Figure 2.8  Testing arrangement of Gaston and Kriz [22]

Figure 2.9  Testing arrangement of Base [8]
appeared to initiate collapse. In the specimens where mold oil had been used, the failure was generally immediately adjacent to the epoxy. This led to the conclusion that the oil penetrated the concrete, preventing the epoxy from properly bonding to the concrete. Under no prestress, the joint withstood a shear stress up to 680 psi (4.69 MPa). The application of prestresses generally increased the ultimate shearing stress by an amount equal to the prestress, but there were too few results to permit an accurate assessment.

Sims and Woodhead (1968)

Sims and Woodhead [65] performed a set of shear tests on epoxied joints similar to those tested by Base in connection with the design of the Rawcliffe Bridge in Yorkshire, England. Eight beams, each made up of three 6 inch (152 mm) cubes epoxied together, were tested to investigated the effects of surface preparation. It was found that wire brushing the contact surface to expose the aggregates increased the strength of the joint, and chiseling the joint surface to expose the aggregates increased the capacity of the joint even more. Sims and Woodhead also reported that the failures occurred through the epoxy, the first time such a failure was noted.

Moustafa (1974)

As part of his investigation into the strength of a segmentally constructed concrete I-beam Moustafa [46] performed
tests to examine the behavior of the epoxied concrete surfaces. He used 6 inch cubes to make up beams of three cubes each and tested them in set-up similar to that used by Base [8]. Three different normal stress levels were used, and in each case the concrete failed adjacent to the epoxy joint. Since the ultimate shearing strengths were much higher than stresses which would occur in the larger test beams, he concluded that the joints would not have a significant influence on the strength of segmental concrete beams.

**Nakazawa (1984)**

Nakazawa [51] performed a series of direct shear tests on epoxy-jointed concrete specimens. Two short cylinders, each 4 inch (102 mm) in diameter and 2-5/8 inch (67 mm) were epoxied together and then sheared apart. A shear box, used at M.I.T. for testing the shear behavior of rocks and soils, was adapted for Nakazawa's experiments. The normal force was extended with the use Belleville washer spring system. Among the parameters tested were surface roughness, epoxy thickness, and normal load (prestressing). As was discovered by earlier researchers, Nakazawa found that the rough surfaced specimens had a higher shear capacity that the smooth surfaced specimens.

**2.3.3 SHEAR TESTS ON KEYED JOINTS**

In contrast to the tests of dry and epoxied flat joints, very
few tests have been performed on keyed joints; four such investigations are reviewed here.

**Kupfer et al. (1982)**

Kupfer et al. [36] performed a series of experimental investigations to determine the structural behavior of segmental precast prestressed joints. In a set-up which simulated the shear and normal stresses in a segmental bridge joint, concrete prisms with joints oriented obliquely to the direction of compression were tested. The prisms were 800 mm high and had a square cross-section 200 mm wide. The joints were inclined at an angle of 50 degrees with the horizontal, and they were bonded by cement mortar or epoxy. Comparison tests on the prisms with cement mortar joints provided with multiple keys yielded a joint strength ranging from 78% to 91% of that of comparable monolithic joints.

**Koseki and Breen (1983)**

Koseki and Breen [33] studied the shear strength of segmental concrete bridge joints by testing specimens having a cross-section of 3 inches (76.2 mm) by 20 inches (508 mm) which is approximately one fourth the size of a web in a typical segmental bridge. The types of joints considered included no keys, single large keys, and multiple keys. Both dry and epoxy joints were tested. Results of these limited exploratory tests showed that designing keys as corbels using either ACI or PCI
specifications is conservative. The specifications predicted a value of only 60% of the actual test results for both the single key and multiple key dry joints. The epoxied joints tested were 60 - 80% stronger than the comparable dry joints. They also found that joints bonded with epoxy developed essentially the same strength as monolithic specimens. Examination of the stress-slip curves indicates a brittle behavior for the epoxied and monolithic sections, but the dry joints, while still exhibiting a brittle behavior, were able to carry a more substantial load following the peak stress.

Ramakko and Sadler (1981)

During casting and erection, a "significant" number of shear keys had been broken on the construction of the Highway 406 bridges over the Twelve Mile Creek [9],[59]. As a result, the Ontario MTC carried out a number of tests to estimate the shear capacities of the keys. Several types of specimens were cast in an effort to determine the shear load required to shear off the keys. In Type I tests (see Figure 2.10a) a single match-cast key was provided on the shear plane. In an effort to discern the contribution of concrete, one half of the specimens had spacer blocks inserted in the end regions of the key so that no compression was transmitted through the key. In a further effort to distinguish frictional resistance from concrete shear strength, grease was applied to the contact surfaces in 13 of the 18 specimens. Type II tests, which were also match-cast,
Figure 2.10 Testing arrangement at Ontario MTC [59]
carried two keys on the shear plane, and a program similar to that for Type I was followed (see Figure 2.10b). A 30 psi longitudinal (or confining) stress was provided in all of the tests.

Actual "shear off" of the keys was never achieved in the series because the loading head was not directly above the shear key (See Figures 2.10a and b).

The authors estimated shear key strength by taking a ratio of the key area to the area along the failure line, and the average value of shear strength for the keys was 5.8 $f'_c$. The authors also noted that the presence of lubricant displayed no effect on the shear strength. Had the placement of the load been closer to the shear plane then it is likely that shear-off of the keys would have occurred. In the case of key shear-off, the grease may have had a more noticeable effect.

**Cholewicki (1971)**

Cholewicki studied the shear transfer of precast panels used in the construction of shear walls used in multi-story buildings [17]. One of the common methods to enhance the rigidity of the walls is to use numerous shear keys throughout the story height, and this paper describes the method of shear transfer based on analytical and experimental reasoning. This report is included here because the shear keys in panel walls and bridge joints are similar in size and function.

In panel buildings the keyed surface will be cast into the
face of a precast panel. After adjoining panels have been correctly placed, a filler concrete or mortar is poured into the gap. This process closely resembles the closure pour required at midspan in many segmental bridges.

The testing program of Cholewicki utilized push-off specimens to place the joints under shear loading. Six different joint geometries were examined, and these are shown in Figure 2.11. Type 3 joints contained multiple shear keys along the specimen height, and no reinforcement was provided between the panels and the infill mortar. There was sufficient bond strength to prevent destruction of the bond, and failure occurred in the mortar. The $\tau_{\text{mean}}$ value was 7.0 kg/cm$^2$, but the infill mortar in this group was relatively weak. The failure mode is described as follows: diagonal cracks form which extend through the infill mortar layer. After cracking of the infill concrete, compressive members form which carry the load in a manner similar to that predicted by the truss models of shear transfer in concrete.

The remaining three test series examine the effects of bond strength between the mortar and the panel and the effects of reinforcement extending from the panel into the mortar layer. The researchers reported a sizable increase in deformation for the joints with lesser bond. The joints with the superior bond carried loads twice as high as those with the damaged bonds. It was observed that those joints with reinforcement carried a forty percent higher ultimate load than unreinforced joints, and
Figure 2.11 Testing arrangement of Cholewicki [17]
the reinforced joints maintained an ability to carry load at much higher displacements, ie. they were more ductile than the unreinforced joints.

Assessment of Keyed Joint Tests

From the preceding discussions it is observed that the body of research in the area of shear keys used in segmental bridge construction is very limited. The referenced authors have performed tests which must be regarded as preliminary in investigating the phenomenon of shear-off of the keys. From the results which have been presented it can be seen that joint failure is usually brittle, and the presence of steel in the joints improves the strength and ductility. Epoxied joints tested in [33] were stronger than the dry joints in single and multiple key configurations.

Design Recommendations

Recently, the Post-Tensioning Institute [56] has prepared specifications for design and construction of segmental bridges. For the design of joints in precast segmental bridges, they proposed using the same equations as in the AASHTO specifications [2] for cast-in-place, wet concrete, or epoxy joints which are specified by PTI as Type A joints. For Type B joints (dry joints) a lower value for the strength reduction factor (for flexure \( \phi_f = 0.90 \) compared to \( \phi_f = 0.95 \) for Type A joints, and for shear \( \phi_f = 0.70 \) compared to 0.85 for Type A
joints) is recommended to account for the larger slip that might occur in these joints. If unbonded tendons are used, these values are further reduced for both types of joints.

This design practice is aimed at the prediction of web cracking which results from diagonal tension cracks or from the extension of flexural cracks. This method does not consider the possibility of shear-off of the keys which was observed in the tests of Koseki and Breen [33] and in the current experiments.

Regarding the shear design proposals, the PTI [56] states "the values of \( \phi_f \) and \( \phi_v \) ... are based on consideration of relatively limited test results and are considered interim provisions until further comprehensive tests, analyses, and experience with completed structures are obtained."

2.4 FIBER REINFORCED CONCRETE

In Chapter 1 it was stated that a more ductile behavior of the joint would be desirable. One possible method to improve the post-peak behavior of the joints would be through the inclusion of discrete fibers in the concrete in the joint regions. The properties of SFRC will be presented in the following section, and the mechanical behavior of this material will be discussed in a variety of failure modes.

Concrete has been shown to be an economical and versatile construction material. The primary deficiency associated with
concrete is its low tensile strength. Traditionally, steel reinforcing bars have been used to compensate for the low tensile strength of concrete, and this can be a major cost factor from a materials and a labor viewpoint.

In recent decades the addition of discrete fibers to the concrete mix has received increased attention. Fiber addition transforms a brittle material to one which may exhibit an increased tensile capacity through the delay and control of cracking of the concrete matrix. In this manner the onset of flexural and shear cracking is delayed and a substantial post-cracking capability is provided as well [45],[50],[69].

Although many fiber materials have been added to the concrete matrix to improve its behavior, only steel fiber reinforced concrete (SFRC) will be presented in this discussion. This decision was based on the observation that steel fibers have been used more frequently in heavy construction applications [27],[45].

2.4.1 Direct Shear Tests

The results of several experimental programs investigating the shear strength of fiber reinforced concrete are presented below. Direct shear or shear with limited bending stresses is addressed in this section. This research is especially applicable to the shear keys because the stress configuration for the keys is one of a nearly direct shear mode.
Kohno et al. (1983)

Kohno et al. conducted an experimental program to investigate the shear strength of SFRC as affected by fiber type, aspect ratio, and aggregate size [32]. 10 x 10 x 40 cm specimens were tested in direct double shear, compression and flexure. Results from the two fiber types [cut fiber (♂) and sheared fiber (♀)] showed slightly higher strengths for the sheared fiber in shear and flexure. This improvement is caused by the larger unit surface area provided by the rectangular cross section. Shear and flexural strengths were lower for larger aggregates. The authors recommend a 10-15 mm maximum aggregate size. Aspect ratio comparisons were performed on the sheared fibers only. Aspect ratios of 40, 60, and 80 were examined, and in these tests the slump was held constant through water addition. Based on shear strength, the optimal aspect ratio is approximately 60, but the optimal for flexural tests lies above 80. Comparisons of flexural and shear tests reveal that relative strength increases are much higher in the direct shear tests. When slump is held constant, it appears that the optimal fiber content is 1.0 to 1.5% by volume. A linear relationship was observed between direct shear and flexural strength. The authors report

\[ f_s = 1.93f_b - 60.6 \]

where \( f_s \) is the shear stress and \( f_b \) is bending stress in kg/cm\(^2\). The correlation coefficient for this equation is 0.833, indicating a fair agreement of data.
Hara (1984)

Hara [24] used push-off specimens to study the capacity of SFRC under combined shear and compressive loading. The failure plane angle was varied along 0°, 10° and 20° to achieve different confinement levels. These tests utilized crimped-end fibers at volume fractions of 0, 0.5, 1 and 1.5%. Significant strength gains were observed only for the 1 and 1.5% Vf specimens.

Another testing series performed by Hara examined the strength improvements of SFRC when used in conjunction with standard reinforcement. In these tests all of the volume fraction levels provide significant ultimate strength gains, and the relationship between strength improvement and fiber content appears to be linear. Individual fiber pullout tests by Hara report significantly higher strength for the crimped fiber over the plain fiber. Embedment lengths of L/2 and L/4 were investigated. For the former embedment length, crimped-end fibers' bond strength was 4.6 times that for the flat fiber, and at an embedment of L/4 the crimped-end fibers were a full 14 times stronger.

Van de Loock (1987)

Van de Loock [73] performed shear tests on precracked fiber reinforced concrete push-off specimens. External bars normal to the cracking plane provided a passive confining stress i.e one which increased only under joint expansion. An important result from this program is the observation that fiber influence
decreased as the normal force was increased. This result is extremely relevant to the SFRC shear key program.

Fattuhi (1987)

Fattuhi conducted tests on 22 steel fiber reinforced corbels in Reference [19]. The corbel dimensions and loading configuration are shown in Figure 2.12. Two fiber types were examined in this program in addition to plain concrete specimens. The two fiber types were Duoform 0.40 x 40mm (0.016 x 1.6 in), and doubly indented 0.65 x 60mm (0.026 x 2.4 in) fibers. Additionally, the shear span-to-depth (a/h) ratio was varied in several tests. Concrete cubes did not show significant strength increases in compressive strength as a result of fiber addition, but the shearing capacity of the corbels was increased considerably.

First crack loads for the 0.4 x 40 mm corbels increased significantly for V_f of 1% and 1.5%. Fracture toughness, computed as area under the load-deflection curve up to an arbitrary deflection of 6.55 mm, was increased by 13.4, 23.5, and 22.1 times over plain concrete for V_f = 0.5%, 1.0%, and 1.5%.

The author derives best fit linear equations for the test groups in which the a/h ratio is varied. These equations are given for plain concrete

\[
\text{Load(kN)} = 38.95 - 29.79a/h
\]

and for SFRC with 1% V_f of 0.65 x 60 indented fibers

\[
\text{Load(kN)} = 120.80 - 89.85a/h.
\]

From the coefficients on the a/h terms, it is evident that the
fibers have a greater effect under predominantly shear loadings. Based on these experimental results, an attempt is made to estimate fiber efficiencies by the shear friction theory which gives the shear strength as

\[ V_n = A_{vf} \cdot f_y \cdot \mu \]

where \( A_{vf} \) is the area of reinforcement across the potential crack, \( f_y \) is the steel yield strength, and \( \mu \) is the coefficient of friction between materials along the crack (assumed to be 1.4 for monolithic concrete). An additional factor is necessary for
fiber reinforced concrete to allow for pullout and orientation.

\[ V_n = A_v f_y \mu n \]

Here \( n \) is an overall fiber efficiency factor. \( n \) values from Fattuhi's tests ranged from 0.057 to 0.186, and these increased with a decrease in fiber content and \( a/h \) ratio.

**Assessment of SFRC Direct Shear Tests**

From the results presented above it can be seen that the addition of steel fibers to the concrete matrix significantly improves the shear strength of the concrete above the strength for an unreinforced matrix. In view of the reported results, it appears that the optimal aspect ratio lies between 60 and 80, and the optimal fiber volume fraction may be 1.25 to 1.5%. It also appears that the strength gains are more substantial for loadings in which the \( a/d \) ratio is lower; SFRC improves strength more in pure shear than in flexure.

2.4.2 **Combined Shear and Flexure Tests**

Many researchers have investigated the mechanical improvements provided by SFRC in flexural loading configurations. The reports of several experimental programs will be presented in this section. Testing configurations which were designed to examine shear behavior in beams are selected for this literature review. The interested reader can find results of other flexure tests in the following references:[10],[11],[18],[40],[41],[62],[64],
Shanmugam and Swaddiwudhipong (1984)

Shanmugam and Swaddiwudhipong examine the ultimate load behavior of fiber reinforced deep beams in Reference [63]. Three test series were performed at span/depth (a/d) ratios of 1.0, 1.5, 2.0, 2.5, 3.0, and 3.5. Series P specimens were cast of plain concrete; series F specimens were made of fiber reinforced concrete, and series FS utilized FRC and conventional bar reinforcement in the tension zone. Dramia (ZP 50/0.5) hooked-end fibers with aspect ratio=60 were used in the FRC at 1% fraction by volume. Test results showed that strength gains from fiber reinforcement were more substantial at lower a/d ratios. F1.0 (Series F at a/d ratio = 1.0) achieved a 95% higher load than P1.0. A more ductile type of failure was observed in the F series while the P series specimens failed suddenly. Deflection at ultimate load was greater than 4 mm for F1.0 and F1.5, but P1.0 and P1.5 fractured completely at a deflection less than 0.8 mm. Hence a significantly higher toughness and ductility is provided by fiber addition. The addition of tensile reinforcement provides larger strength gains for the beams which fail in flexure than for those failing in shear. The presence of fibers did cause flexural/shear failure modes which exhibited a more ductile behavior than the pure shear failure which would have been observed with plain concrete.
Swamy and Bahia (1985)

In an attempt to distinguish the effects of steel fibers on shear behavior of reinforced concrete beams, Swamy and Bahia performed tests on 11 reinforced concrete girders with varying reinforcement arrangements and cross sections [69]. The experiments employed 9 T-shaped and 2 rectangular sections which were simply supported over a 2.8 m (9.1 ft.) span. The loading was applied such that the moment to shear ration was 4.5. From these tests it was observed that the fibers substantially increase the shear capacity of the beam over the section with no shear reinforcement. An equation was fitted for the T-sections which related ultimate shear stress to the flexural strength of 4 x 4 x 20 in. control beams rather than basing the shear strength on a compressive value.

Narayanan and Darwish (1987)

Narayanan and Darwish conducted a testing program which examined several aspects of SFRC behavior [53]. Crimped steel fibers with aspect ratios of 100 or 133 were used. One group of tests compared the behavior of longitudinally reinforced beams with plain and fiber reinforced concrete. Increases of up to 170 percent in ultimate shear strength were observed, and strength gains were higher at the lower a/d ratios. For beams with higher volume fractions, several diagonal cracks were observed thus indicating a redistribution of stresses after cracking. At volume fractions as low as 1%, the fibers produced
flexural-shear and flexural failures rather than shear failures. The longitudinal reinforcement ratio, $\rho$, was 2% in these beams.

Another group of tests investigated the possibility of replacement of conventional stirrups with steel fibers. Ten beams were cast with equal percentages of either traditional reinforcement or fibers. Load-deflection characteristics of the beams were very similar. At lower volume fractions the fibers yielded higher ultimate strength values while at higher volumes (1.5 and 2.0%) the stirrups were stronger. In all cases fiber reinforcement led to a higher cracking strength and a more extensive cracking pattern. Here again is evidence of the stress redistribution which is provided by the uniform distribution of reinforcement.

Assessment of SFRC Combined Shear and Flexure Tests

From the above reports it is seen that SFRC induces a better mechanical behavior in beams which experience shear and flexural loading. In addition to gaining higher ultimate strengths, a much more ductile behavior is observed. As was reported in the previous section, the fibers tend to yield higher strength gains under predominantly shear loading conditions. Although previous discussions have focused on the local behavior of the joint regions in segmental bridges, it should be noted that several researchers reported that SFRC could complement or completely replace stirrups [11],[39]. This prospect may be considered in the global scheme of segmental bridge construction.
2.4.3 **Modeling of SFRC Tensile and Shear Behavior**

**Model of SFRC Tensile Behavior**

In ref [39] Lim et al. propose a model for the tensile behavior of SFRC. Several areas are discussed in this article including pre- and post-cracking composite behavior and fiber bond-slip relationships. Regarding fiber pullout, it was found that twice as much force is required to dislodge a deformed (hooked-end fiber) from the matrix as is required for a straight fiber.

A trilinear model is used to describe the tensile behavior of SFRC, and this curve is shown in Figure 2.13. The elastic deformation is given by

\[
\Delta L = PL / AE_{ct}
\]

where \( P \) is the loading, \( L \) is the gauge length, \( A \) is the cross sectional area, and \( E_{ct} \) is the composite modulus of elasticity. The latter variable can be calculated as

\[
E_{ct} = E_{mt}V_m + \eta_l \cdot \eta_o \cdot E_f \cdot V_f
\]

where \( V_m \) and \( V_f \) are the matrix and fiber volumes, \( \eta_l \) is defined as the length efficiency factor, and \( \eta_o \) is the orientation factor which may be taken as 0.14 for a specimen with cross sectional dimension greater than the fiber length. If the elastic shortening of the test grips is neglected, by compatibility of extension the load will drop to the point marked (A) after the section has cracked. Point (A) corresponds to the load level at
Figure 2.13 Analytical load extension curves proposed by Lim et al. [39]

Figure 2.14 Analytical shear model proposal by Narayanan and Darwish [53]
cracking strain which would be carried by a pre-cracked section. An increase in load level is observed until the ultimate fiber bond stress is reached at which point the load declines linearly.

Although this model is based on tensile behavior of SFRC, it is relevant to the current research because shear failure is dominated by tensile stresses.

**Model of SFRC Shear Behavior**

Several researchers have proposed models which estimate the ultimate shear capacity of fiber reinforced beams. Similar models were developed by Uomoto and Weeraratne [72] and Narayanan and Darwish [53], and the latter will be described here. The free body diagram of part of the shear span of a simply supported fiber reinforced concrete beam is shown in Figure 2.14. The various contributions to the shear resistance of the beam are (1) the shear forces across the compression zone, $V_c$; (2) the vertical component of aggregate interlock, $V_a$; (3) the vertical force from dowel action of the main reinforcement, $V_d$; and (4) the vertical component of the tensile force provided by fibers along the cracked surface, $V_b$. The authors note that all four of the shear forces may not be additive when failure is imminent, and to assure a conservative prediction of ultimate load, the $V_a$ term is ignored under the present model.

The vertical component of the fiber contribution is evaluated
through the summation of individual fiber forces across the cracked surface. The number of fibers at a cross section is given as

\[ n_w = \frac{1.64 \rho_f}{\pi D^2} \]

where \( \rho_f \) and \( D \) are the fiber volume fraction and diameter, respectively. The number of fibers across the inclined cracked section is given as

\[ n = n_w b \frac{jd}{\sin \alpha} \]

where \( b \) is the beam width and \( \frac{jd}{\sin \alpha} \) is the inclined crack length. From this point the authors go on to calculate the total bond area of fibers across the inclined cracked section based on an average embedment depth of \( L/4 \). The total force developed by the fibers is then given as

\[ F_b = A_b \tau \]

where \( A_b \) is the total bond area of fibers and \( \tau \) is the average fiber matrix interfacial bond stress. Assuming that the angle of the cracked surface to the horizontal \( (\alpha) \) is 45°, the vertical fiber pullout stresses can be given as

\[ v_b = 0.41 d_f \frac{L}{D} \rho_f \tau \]

\[ = 0.41 F \tau \]

where \( d_f \) is the fiber bond factor defined as 0.5 for straight fibers, 0.75 for crimped fibers, and 1.0 for indented fibers. \( L/D \) is the aspect ratio, and \( F \) is the fiber factor defined as the product of the fiber aspect ratio, volume fraction, and bond
factor. Based on several other researchers' results, a value of 4.15 N/mm² is assumed for the ultimate bond stress.

An ultimate shear equation for design is proposed which considers the contributions from $V_c$, $V_d$, and $V_b$. This equation is given as

$$V_u = e[A' \cdot f_{spfc} + B' \cdot d/a] + V_b$$

(all these units are in N and mm) where $e$ is a non-dimensional factor which takes into account the effect of arch action.

$$e = 1.0 \quad \text{for } a/d > 2.8$$

$$= 2.8 \cdot d/a \quad \text{for } a/d \leq 2.8$$

$f_{spfc}$ is the calculated split cylinder strength, $A'$ is a non-dimensional constant and $B'$ is a dimensional constant. Their values were derived through regression analysis and are given as 0.24 and 80 N/mm².

The shearing strengths predicted by this equation were compared with the results of tests on 91 beam tests. The mean value of observed/predicted ultimate shear was 1.09 with a coefficient of variation of 14.45 percent. This rather large variational measure may result from the fact that size effects are not considered in the equation. It has been reported that shear strength exhibits dependence on the member size [76].

Thus it can be seen that several models have been proposed which describe the shear or tensile behavior of SFRC. These models will later be used in the development of a model which characterizes the effects of SFRC when applied in keyed joints.
CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 SCOPE

It is hoped that the addition of discrete steel fibers to concrete in the joint regions of precast concrete segmental bridges will improve the strength and safety of these bridges. A series of tests on joint models which utilized SFRC were performed as part of a larger research project.

This larger research effort closely examined the behavior of bridge joints under variation in epoxy thickness and prestress levels. The basic mechanics of shear transfer between flat epoxied and dry concrete surfaces were examined. Tests were performed on a 1:1 model of a shear key to understand the key behavior on an elemental level. Monotonic loading tests were performed to evaluate strength and deformation of the specimens, and cyclic tests were aimed at understanding the degradation of specimen strength under repeated loadings.

The thrust of the SFRC keyed joint tests was to evaluate improvements in strength and ductility of the keys as compared to the joints with plain concrete. To facilitate the comparison with existing results, the testing methods for the SFRC specimens followed along the template of the previous research. Although some modifications to the concrete were necessary, the basic content of the mix remained the same.
Two types of commercially available steel fibers were utilized in the course of the research program; these were straight and crimped-end fibers. The fiber effects were examined in dry joints at 100 and 500 psi confinement levels and in epoxied joints at a 500 psi confinement level, and specimens with fiber volumes of 0%, 1%, and 2% were tested. Two other testing series provided a preliminary examination of the ability of straight fiber SFRC to withstand cyclic loadings. Plain concrete and SFRC control cylinders were cast with each testing series to estimate the concrete compressive strength for normalization of the shear tests.

Three specimens were cast from each concrete batch, and this group of specimens was defined as a series. For the monotonic tests, a series consists of three specimens which were tested under the same confining stress and the same epoxy thickness (or dry condition). The first specimen was cast of plain concrete while the second and third specimens were cast of 1% and 2% $V_f$ SFRC. Three series deviate from this schedule, and each of the specimens in these series was cast of SFRC with the same volume fraction. Two of these series investigated SFRC under cyclic loading, and the other examined strength for 1% $V_f$ under three different confinement levels.

A summary of the testing program is provided in Table 3.1. Note that in the table peak shear stress is defined as total load carried by the specimen divided by the area of the shearing plane (18 sq. in.). The next column lists $\tau / f'_c$; here the shear
**TABLE 3.1 SFRC KEYED JOINT TESTING SUMMARY**

<table>
<thead>
<tr>
<th>Series</th>
<th>Test</th>
<th>Confining stress psi</th>
<th>Epoxy thickness mm</th>
<th>Fiber type</th>
<th>Fiber Vol. %</th>
<th>$f'_c$ psi</th>
<th>Peak shear stress, $\tau$ psi</th>
<th>$\tau / \sqrt{f'_c}$</th>
<th>Toughness psi*in</th>
</tr>
</thead>
<tbody>
<tr>
<td>KF1</td>
<td>A</td>
<td>500</td>
<td>0</td>
<td>FT</td>
<td>&quot;</td>
<td>0</td>
<td>6560</td>
<td>1220.5</td>
<td>15.069 0.765</td>
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<tr>
<td></td>
<td>B</td>
<td>500</td>
<td>0</td>
<td>&quot;</td>
<td>1</td>
<td>&quot;</td>
<td>1363.1</td>
<td>16.830 1.118</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>500</td>
<td>0</td>
<td>&quot;</td>
<td>2</td>
<td>&quot;</td>
<td>1461.1</td>
<td>18.040 1.242</td>
<td></td>
</tr>
<tr>
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<td>0</td>
<td>FT</td>
<td>&quot;</td>
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<td>1388.3</td>
<td>16.396 0.933</td>
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<td>&quot;</td>
<td>1</td>
<td>&quot;</td>
<td>1472.3</td>
<td>17.387 1.122</td>
<td></td>
</tr>
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<td>0</td>
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<td>&quot;</td>
<td>1571.3</td>
<td>18.557 1.332</td>
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<td>2</td>
<td>FT</td>
<td>&quot;</td>
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<td>1596.9</td>
<td>17.700 1.004</td>
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<tr>
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<td>1533.9 Cyclic</td>
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<td>&quot;</td>
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<td>&quot;</td>
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<td>14.720 0.821</td>
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<td>1496.8</td>
<td>17.113 0.989</td>
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* FT = FlexTen straight fiber  
DM = Dramix hooked-end fiber
## TABLE 3.1 SPFC KEYED JOINT TESTING SUMMARY - CONT'D

<table>
<thead>
<tr>
<th>Series</th>
<th>Test</th>
<th>Confining stress psi</th>
<th>Epoxy thickness mm</th>
<th>Fiber type</th>
<th>Fiber Vol. %</th>
<th>$f'_c$ psi</th>
<th>Peak shear stress, $\tau$ psi</th>
<th>$\tau / \sqrt{f'_c}$</th>
<th>Toughness psi*in</th>
</tr>
</thead>
<tbody>
<tr>
<td>KF8</td>
<td>A</td>
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<td>&quot;</td>
<td>Cyclic</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
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<td>C</td>
<td>500</td>
<td>0</td>
<td>&quot;</td>
<td>2</td>
<td>&quot;</td>
<td>Cyclic</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>KF9</td>
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<td>2</td>
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<td>0</td>
<td>&quot;</td>
<td>1</td>
<td>&quot;</td>
<td>1040.3</td>
<td>11.509</td>
<td>0.600</td>
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<tr>
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<td>0</td>
<td>&quot;</td>
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<td>&quot;</td>
<td>1411.3</td>
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<td>DM</td>
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<td>1418.9</td>
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<td>&quot;</td>
<td>1</td>
<td>&quot;</td>
<td>1859.3</td>
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<td>1.299</td>
</tr>
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<td>&quot;</td>
<td>2</td>
<td>&quot;</td>
<td>1775.9</td>
<td>20.134</td>
<td>1.324</td>
</tr>
</tbody>
</table>
stress has been normalized by the square root of the cylinder compressive strength. The toughness is calculated as the area under the normalized shear stress-slip curve up to a displacement of 2.5 mm.

3.2 SPECIMEN DESCRIPTION

The shear capacity of keys in segmental bridge joints was examined through the use of push-off specimens similar to those used by Walraven [75] and Hsu [28]. The specimens were 10 in. along the base, 15 1/2 in. along the outside edge, and 3 in. deep (See Figure 3.1).

The shear key itself is a 1:1 reproduction of a key which was used in the construction of an actual segmental bridge project which utilized multiple keys along the entire height of the web. The bases of the trapezoidal shaped key are 3 7/8 and 2 5/8 in., and the altitude is 1 1/4 in. The overall depth of the shearing contact area is 6 in. which yields a projected contact area of 18 sq. in.

To ensure that the specimen did fail in the intended shearing mode, it was necessary to guard against bearing, splitting, and flexural failures. Mild reinforcement was placed in the specimens to accomplish this end. The outside face was prone to flexural failure, and the base was prone to bearing failure. Two bent #5 bars were placed in the specimen as shown in Section B of Figure 3.1 to prevent such failures. These Grade 60 "I" shaped bars were 8 and 13 in. on the longer and shorter edges,
Figure 3.1 Keyed joint specimen dimensions
respectively. Two straight #3 bars, also Grade 60, were placed as shown in Figure 3.1 to prevent a splitting failure at the inside corner of the specimen.

3.3 MATERIALS

The three primary material parameters within the testing program were concrete, steel fibers, and epoxy. Each of these materials will be described in detail in the following sections.

3.3.1 Concrete

The first SFRC keyed joint test utilized the same concrete mix which had been used in tests on plain concrete keyed joints. This mix had a 0.45 w/c ratio. By using the same mix in the SFRC and plain concrete key joints, comparisons could easily be made between the two concrete types. However, it was evident from the first test that many of the fibers were pulling out before yielding and were therefore not realizing full strength and ductility gains, and so it became necessary to modify the mix in order to prevent early fiber pull-out.

Following the example of Fattuhi [19], superplasticizer was added to the mix in an effort to reduce the w/c ratio. By reducing the w/c ratio it was hoped that the improved matrix would develop a higher bond stress. As per recommendations of W. R. Grace engineering consultants, the w/c ratio was decreased to 0.38 to allow better utilization of the superplasticizer. Due to the lower w/c ratio, the concrete strength was roughly
15% higher. Provision for this discrepancy in matrix strength is made through the normalization of the shear stress by the square root of the concrete cylinder strength. Cylinder strengths for each of the testing series are listed in Table 3.1.

Even after superplasticizer was added to the mix it was suspected that the fibers were pulling out prematurely. Again an attempt was made to prevent early fiber pullout. Another variation in the concrete mix which was considered was the replacement of a fraction of the cement with fly ash. Several researchers have reported that the addition or substitution of fly ash for cement may improve the bond between the matrix and the fiber [4],[69]. It was decided not to use fly ash but to resort instead to the use of deformed fibers to prevent premature pullout. In this manner the matrix parameters would be held constant and comparisons to plain concrete tests would still be valid.

The quantities for the plain concrete and SFRC mixes are given in Table 3.2. From this table it can be seen that the mix quantity was increased slightly for the SFRC so that several additional control cylinders could be cast.

Gradation of the coarse aggregate was controlled by weighing out specified quantities remaining on the 1/4, 3/8 and 1/2 in. sieves. It became necessary during the SFRC testing to use two gravel types for the quantities specified in the mix design. The 1/2 in. large aggregate came directly from Boston Sand and
### TABLE 3.2 CONCRETE MIX SUMMARY

<table>
<thead>
<tr>
<th>MIX</th>
<th>Water (lb)</th>
<th>Water ratio</th>
<th>Cement (lb)</th>
<th>Cement ratio</th>
<th>Sand (lb)</th>
<th>Sand ratio</th>
<th>Coarse Aggregate</th>
<th>Coarse Aggregate</th>
<th>Coarse Aggregate</th>
<th>Admixture (ml)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1/4&quot;</td>
<td>3/8&quot;</td>
<td>1/2&quot;</td>
<td></td>
</tr>
<tr>
<td>Plain Conc.</td>
<td>11.52</td>
<td>0.45</td>
<td>25.6</td>
<td>1.0</td>
<td>29.3</td>
<td>1.14</td>
<td>4.0</td>
<td>0.16</td>
<td>28.0</td>
<td>1.09</td>
</tr>
<tr>
<td>SFRC</td>
<td>10.81</td>
<td>0.38</td>
<td>28.16</td>
<td>1.0</td>
<td>32.25</td>
<td>1.14</td>
<td>4.38</td>
<td>0.16</td>
<td>30.81</td>
<td>1.09</td>
</tr>
</tbody>
</table>

**Fiber quantities:**

- Initial weight of concrete mix: 112.2 #
- Cast plain specimen, 3 cylinders: 37.9 #
- Remaining concrete: 74.3 # \( \times 0.01 \times \frac{500}{150} = 2.475 \) Fiber for 1% \( V_f \).
- Cast 1% \( V_f \) specimen, 2 cylinders: 37.2 #
- Remaining concrete: 37.1 # \( \times 0.01 \times \frac{500}{150} = 1.238 \) Add'l Fiber for 2% \( V_f \).

\( \dagger \) 500 pcf = assumed steel unit wt.

150 pcf = assumed concrete unit weight
Gravel stockpiles. This aggregate was a crushed rock composed mainly of basaltic minerals. Although specified as 3/8 in. gravel, the supply was deficient in the lower sizes. To satisfy the requirements of the mix design, it became necessary to use a smaller size aggregate in addition to the crushed stone. This smaller rock was a pea gravel which was supplied in 80 lb. sacks by the Waldo Brothers Company.

Type III high early strength cement was employed in the mix so that the specimens could be tested at 9 days after casting. In order to ensure consistent test results, the quality of the cement was carefully controlled by sifting before use. In this manner, any hydrated cement was left on the sieve, and only very fine cement was used in the mix.

Standard mortar sand, provided in 70 lb. bags by a local sand and gravel company, was used in all of the mixes. The steel used to reinforce the sections was Grade 60 mild steel supplied by the Barker Steel Company, Boston, MA.

WRDA-19 Superplasticizer was provided by W.R. Grace Company, Cambridge, MA. This compound conforms to ASTM specification C-494 [5] for admixture of Types A and F.

3.2.2 Fibers

Two types of fibers were utilized during the course of the testing program: FlexTen and Dramix fibers.

FlexTen is a carbon steel fiber made from steel sheet with a tensile strength of approximately 50 ksi. These were straight,
rectangular fibers 0.011 x 0.022 x 1.0 in. Converting the cross-sectional dimensions to an equivalent diameter yields an aspect ratio of 60.

Observations from the initial SFRC key tests indicated that many of the fibers were pulling out of the matrix prematurely. In order to improve the strength and ductility of the keys, an effort was made to prevent early fiber pullout. Several approaches could be followed to accomplish this goal. First, the matrix could be further modified to improve bonding characteristics. Superplasticizer was added to the matrix in an effort to achieve this end. Even with the superplasticized concrete matrix, it was suspected that full strength and ductility gains were not being extracted from the straight fibers. Again it became necessary to develop a better bond between the fiber and matrix. As discussed in the previous section, however, it was not desirable to further change the concrete mix so that comparisons to the existing research work would be valid. Therefore, the fiber itself required modification to prevent slippage. Two possible options exist to achieve this objective. Either a fiber with a higher aspect ratio could be used or deformations could be introduced to the fiber while holding the aspect ratio constant. Several fiber distributors were contacted to discuss the issues of mechanical qualities and availability of the different fiber types.

A critical factor in the fiber selection was the key dimension. Since the key indentation had dimensions of 4 x 1.25
in., a 2 or 3 in. fiber might not be effective due to nonuniform distribution. None of the fiber companies which the author contacted were able to provide an aspect ratio greater the 60 in a 1 in. length. As a result, deformed fibers were chosen to improve fiber efficiency.

Dramix ZL 30/.50 hooked-end fibers were finally selected. These fibers are 30 mm long with a 0.5 mm diameter and a resulting aspect ratio of 60. The fibers are manufactured from a low carbon, cold drawn steel wire, with a minimum tensile yield strength of 150 ksi. Dramix fibers meet or exceed the requirements of ASTM A820-85 for Type 1, deformed fibers [5].

3.3.2 Epoxy

Two of the straight fiber series and one of the crimped-end series were tested with epoxied key joints. Dural 100 Type II, a two component epoxy provided by the Dural International Corporation was used in the tests. This epoxy type was employed on major bridge projects such as the Linn Cove Viaduct in North Carolina, Sunshine Skyway in Tampa, Florida, and the Kishwaukee Bridges in Rockford, Illinois [54]. Manufacturers' specifications for the epoxy parameters are listed in Table 3.3. Compressive tests were performed on small epoxy cubes on several occasions to provide a check against the manufacturers' specifications. The average peak strength of these tests was 10,600 psi; this value is only slightly lower than reported in the manufacturers specifications.
Table 3.3 EPOXY SPECIFICATIONS

The epoxy used in these experiments was Dural 100 Type II, manufactured by Dural International Corporation. The specifications (provided by Dural) of the epoxy are as follows:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>temperature range</td>
<td>55 to 80°F</td>
</tr>
<tr>
<td>viscosity of mixed material</td>
<td>non-sag</td>
</tr>
<tr>
<td>gel time (at high temperature)</td>
<td>32 min.</td>
</tr>
<tr>
<td>open time (at high temperature)</td>
<td>80 min.</td>
</tr>
<tr>
<td>24 hr. compressive strength (at low temperature)</td>
<td>11,000 psi</td>
</tr>
<tr>
<td>48 hr. compressive strength (at low temperature)</td>
<td>12,000 psi</td>
</tr>
<tr>
<td>compressive shear strength (cured at low temperature, tested at 77°F)</td>
<td>3170 psi</td>
</tr>
<tr>
<td>total load on slant cylinder</td>
<td>179,260 lbs</td>
</tr>
<tr>
<td>load on compression cylinder</td>
<td>192,760 lbs.</td>
</tr>
<tr>
<td>percentage load on slant cylinder</td>
<td>93.0%</td>
</tr>
<tr>
<td>resin : hardener ratio</td>
<td>100:31.24</td>
</tr>
</tbody>
</table>
3.4 SPECIMEN PREPARATION

3.4.1 Concrete Mixing and Casting

Casting of the keyed specimens took place on two successive days. Prior to casting, it was necessary to prepare the molds and reinforcing steel. The concrete casting sequence, molds and steel are described in the following section.

The reinforcing steel was prepared by arranging the bars in two planes spaced approximately 2 in. apart. Using 1/4 in. steel dowel as a framework, the reinforcing "cage" was secured with 6 in. steel reinforcement ties. The completed reinforcement is shown in Figure 3.2.

The molds used to cast the keyed specimens are shown in Figure 3.3. The various pieces of the molds were fabricated from 1/2 in. plexiglas sheets. The pieces were held together by 1 in. 10-32 screws which could be removed to expedite demolding. These molds, along with the 3 in. diameter X 6 in. cylinder molds, were lubricated with a light mold oil to facilitate removal of the concrete from the molds.

A removable key wall, which can be seen in the center of the mold in Figure 3.3, provided the geometric surface against which the female portion of the joint was cast. One day after casting of the female part of the specimen, the key wall was removed, and the male block was cast directly against the existing concrete surface. A light coating of mold oil facilitated later separation of the two blocks. This casting procedure is shown
Figure 3.2  Mild steel reinforcement

Figure 3.3  Molds for the keyed specimens
in Figure 3.4; it emulates the industry process known as match casting.

The coarse aggregate used in the mix was rinsed prior to mixing to remove any silt or other fine particles adhering to the aggregate. Following rinsing, the aggregates were weighed once again. The water retained on the aggregates after rinsing was calculated as the difference between the two weights, and this quantity was subtracted out from the mix water.

After all of the coarse aggregate sizes had been washed, they were placed in the mixer and rotated for approximately one minute to ensure uniform gradation. At this point the superplasticizer was blended in with the mix water, and the remaining components of water, sand, and cement were added to the mixer incrementally. Sand and cement were added in roughly equal quantities, and water was added only as necessary to maintain a moist consistency. After all of the dry ingredients had been combined, the remaining mix water was added.

In accordance with ASTM C-192 [5] provisions for mixing concrete in the laboratory, the concrete was mixed for 3 minutes; allowed to sit while covered for another 3 minutes; and mixed a final 2 minutes before placement.

Fiber addition took place after the plain concrete specimens and/or plain concrete control cylinders had been cast. Based on the weight of concrete remaining in the mixer after casting of the plain specimens, the weight of fibers corresponding to a 1% volume fraction were added to the mix. The fibers were
Figure 3.4  Match-casting process
sprinkled gradually into the concrete mass as the mixer rotated, and care was taken to ensure that balling of the fibers did not occur. After one additional minute of mixing, the specimen and cylinders with 1% \( V_f \) were cast. A similar procedure was followed to cast the 2% \( V_f \) specimens.

Before placement of the concrete, the molds were fixed to a vibrating table with 2 "C" clamps. Concrete was placed in two equal lifts, and the reinforcement was set in place following the first lift. Each lift was compacted by a combination of rodding and external vibration. The rods used were as specified in ASTM C-192 [5], and the vibration was applied for approximately one minute. Longer periods of vibration were frequently required for the SFRC with higher volume fractions. The specimen surface was smoothed by troweling.

The 3 in. X 6 in. control cylinders were cast as per ASTM C-192 specifications [5]. The concrete was placed in three equal lifts, and each lift was rodded 25 times. Supplemental light vibration was also provided as allowed under the ASTM provisions.

A thick plastic sheet was placed on the specimens following casting, and they were allowed to cure in the molds for 24 hours. Following this initial curing period, the specimens were demolded and cured underwater for another four days.
3.4.2 Epoxying Procedure

The concrete specimens were epoxied on the fifth day after casting according to the procedures described in the following section. The objective of the epoxying operations was to distribute an epoxy layer of even thickness over a projected contact area of 18 sq. in. The keyed joint specimen curing configuration can be seen in Figure 3.5, and the epoxying procedure is described in the following paragraphs.

The first step in the epoxying sequence was to prepare the concrete surface. This was accomplished by first cleaning any heavy residue with a stiff wire brush which was followed by sanding with a coarse grained sandpaper. High pressure air was used to clear minute particles from the void spaces on the surface. At this point the epoxy boundaries were marked on the concrete.

Uniform thickness of the epoxy was achieved by placing two aluminum bars on the shear plane with their axes oriented at right angles to the loading direction. These bars were 1 in. x 5 in. (25 mm x 76 mm) and their thickness varied according to the desired thickness of epoxy for the individual test. Oil was distributed on either side of the bars to facilitate their removal after the epoxy had cured. This oil did not affect the epoxy bond as it never came in contact with areas of epoxy application.

Restriction of the epoxy to the intended area was accomplished through the use of thin (0.5 mm) aluminum strips placed at the
Figure 3.5  Specimen configuration for curing of epoxy
limits of the region to be epoxied. To fix the strips in place, scotch tape was placed on the specimen at the epoxying boundaries, and very small amounts of superglue were used to secure the strips to the tape and, consequently, to the specimen. After the epoxy had been applied and correct specimen alignment was achieved, the strips were gently broken free from the tape and slid out. The sliding out of the strips removed most excess epoxy from the open region between the two faces. Any epoxy remaining beyond the specified limits induced minimal effect since it was bonded to the tape rather than to the concrete.

Scotch tape was also placed on the outside faces of the specimen near the epoxied regions so that excess epoxy could be easily removed after the curing period by scraping the surface with a putty knife. By removing excess epoxy from the specimen face, cracking patterns in the concrete could be better observed as the test progressed.

Once the surface preparation, marking, taping, and bar placement had been completed, the specimen was ready for the epoxy. Using a 500 g balance, epoxy quantities were weighed out in a small (8 oz.) paint can. Hardener amounts were first weighed, and then the resin quantity was added. The epoxy was mixed for 4 to 5 min. with a Jiffy brand epoxy mixing paddle powered by a 3/8 in. electric drill.

Following thorough mixing, the epoxy was "painted" on the specimen with a 1 in. putty knife. Part A of the specimen
(Figure 3.1) was placed on small blocks so that the clamp would have clearance underneath the specimen. Both sections A and B were painted with epoxy, but the male block received a heavier coating since it was on the bottom (See Figure 3.5). Sufficient epoxy was used on the specimens to prevent the formation of any air voids in the epoxy layer.

At this point Part B was placed on Part A. Extreme care was taken to achieve nearly correct alignment because repositioning of the two blocks could smear the epoxy outside of the desired areas or disturb the bar arrangement. Final alignment was checked by placing a straight edge between the two blocks at the upper and lower corners of the specimen.

A pressure was applied to the epoxied surfaces during the curing period. This pressure is provided by a 24 in. cabinet clamp, and the estimated applied pressure was 50 psi. Excess epoxy was extruded from the surface by the application of a pressure with cabinet clamps.

After three days curing time, the clamp and spacer strips were removed. In several cases it was not possible to remove both of the spacer strips, and the specimen was tested with these in place. Examination of ultimate strength values, load-slip and load-horizontal expansion curves indicated that the presence of the bars did not have a significant effect.
3.5 TESTING EQUIPMENT

The shearing force used in the experiments was provided by a Material Testing System (MTS) model number 916.04-30 machine with a 50 metric ton capacity (See Figure 3.6). Loading patterns are generated at the system controller which can be seen in Figure 3.7. Through a feedback loop control configuration, the MTS machine is capable of load, strain, and displacement controlled tests under ramp, sine, and haver-sine loading patterns. Axial force exerted by the machine is measured using a calibrated MTS load cell. Crosshead displacement or stroke was measured using an internal LVDT (Linear Variable Differential Transformer).

Confining stress was provided by a hydraulic pump and confining grid. The latter consisted of two rectangular tubular sections prevented from moving outward by nuts on two 11/16 in. diameter prestressing rods. Two 4 in. x 3 in. x 1/2 in. steel plates were welded to the tubular sections to provide a uniformly distributed normal stress. Force for the confinement was provided by an Enerpac cylinder and Enerpac hand pump.

Digital testing information is collected with the data acquisition system shown in Figure 3.8. This system is composed of a Fluke 2400B Analog to Digital (A/D) converter and an IBM XT personal computer. As per instructions from the computer, the A/D converter receives continuous input from a set number of channels and then discretizes the data at specified time intervals.
Figure 3.6  MTS testing frame
Figure 3.7  MTS system controller
Figure 3.8 Data acquisition system
Specimen deformations were recorded by LVDT's placed directly on the specimen. These LVDT's used in this capacity are alternating current Schaevitz Model 100 Meir-1991. These small instruments (3/8 in. diameter X 1 in. long; 5 oz.) have an accuracy of 1.0E-5 in/mV.

The control cylinders were broken on a 200 kip Baldwin Testing Machine. This testing frame was run on a feedback loop, and the feedback loop was controlled from software on an IBM AT.

3.6 TESTING PROCEDURE

The specimens were tested on the ninth day after casting according to the testing procedure described below.

Pretest preparation included the removal of any residue from the specimen surface and marking of the specimen. The markings were placed on the specimen to ensure that (1) it would be centered between the loading heads and (2) so that the confining grid would apply an even pressure over the shearing area. After the specimen had been lifted onto the loading head the confining grid was held in place, and pressure was then provided from the hydraulic pump to secure the grid (See Figure 3.9).

The next step in the testing procedure involved placement of the displacement measurement devices (See Figures 3.9 and 3.10). These devices were placed so that vertical slip between the two blocks could be measured. On several occasions horizontal expansion of the joint was measured as well.

In addition to providing information about the expansion of
Figure 3.9  Testing configuration
Figure 3.10 Testing diagram
the joint under ultimate loads, the horizontal displacement readings were also used to check that significant rotation of the specimen did not occur during the test. On another occasion LVDT's were placed on either side of the specimen (i.e. front and back) to ensure that out-of-plane bending did not occur. From these tests it was judged that the loading did not develop substantial bending forces in- or out-of-plane, and hence a direct shear loading was approximated.

Small metal cubes (1/2 x 1/2 x 1/2 in) with cylindrical openings fixed the LVDT's to the specimen surface. The core for the LVDT rested on a target which was situated on the other block. Small springs provided compressive force to keep the cores in place for the horizontal LVDT's. The holding blocks and targets were fixed to the specimen with superglue, and scotch tape was placed on the metal pieces to facilitate removal when the test was completed. Correct alignment of the metal blocks was ensured by (1) drawing parallel lines on the specimen with a T square and (2) through alignment pins and holes drilled in the pieces.

In final preparation for testing, the electronic inputs were checked with a voltmeter and the data acquisition system; the settings on the MTS machine were confirmed; and the plotter paper and connections were checked.

To begin the test, the upper crosshead was lowered until a small force was exerted on the specimen, and the system controller was activated. As the test progressed, cracking
patterns were observed and recorded, and these patterns were correlated with progress on the load-slip graph.

The test was concluded when it was judged that the post-peak shear resistance had leveled out. At the finish of the test the data acquisition program was terminated and data was transferred to floppy diskettes for later processing.

The monotonic tests were displacement controlled at 0.125 mm/min. This rather slow loading rate was selected so that cracking patterns could be clearly observed and recorded as the test progressed.

The two cyclic series were conducted as follows: Specimen A of the series was tested in monotonic loading to estimate the ultimate load of the joint. Specimens B and C were cycled at 90 and 95% of the ultimate load. Thirty cycles at $1.7 \times 10^{-2}$ Hz were applied under load controlled tests, and if the specimen did not fail before the 30th cycle then it was tested monotonically to obtain the reserve strength.
CHAPTER 4

DISCUSSION OF RESULTS

Tests were undertaken to investigate the possibility that the use of steel fiber reinforced concrete in precast bridge joints would improve the strength and safety of the bridges. The findings from these few tests may only be regarded as preliminary, and these results will be presented in this chapter.

As outlined earlier, the SFRC keyed joint tests were one segment of a larger research program which examined the behavior of precast concrete segmental bridge joints. In this chapter, the results of this main testing effort will be presented first to acquaint the reader with strength and deformation behaviors of the joints as influenced by the bridge prestress level and epoxy thickness. Sections 4.2 and 4.3 will present and discuss the test results for straight and crimped-end fibers. A comparison between the behavior for the fiber types will then be presented, followed by sections dealing with modeling considerations and industry applications.

Before proceeding to these sections, a brief discussion concerning the presentation of results will be given. Data is presented by plots (Figure 4.1) in which the abscissa gives the values of the vertical slip and the ordinate represents the shear stress of the joint normalized with \( /f'_c \). Shear stress is defined here as the load divided by a projected area of 18 sq.
Figure 4.1 Sample shear stress-slip curve
in. (116.1 cm²). This is a similar normalization scheme to the one used by ACI [3] and AASHTO [2] which assumes that the shear stress varies with the square of the cylinder compressive strength which is a measure of concrete tensile strength. The ultimate shear strength shall be defined as the maximum shear stress that the specimen carries; this quantity is shown graphically in Figure 4.1.

One of the primary parameters associated with SFRC is the fracture toughness which quantifies the energy absorption of the material or the ability to withstand loads after the peak stress has been achieved. It is usually taken as the area under the stress-strain curve up to a specified point. It may also be given as a ratio of areas under the load-deflection curve before and after the onset of cracking up to a given limit. This limit is usually defined as a multiple of the first-crack deflection [4],[7]. ACI Committee 544 recommends that toughness indices be calculated from flexure tests on 4 x 4 x 14 in. beams [4].

The toughness parameter used in this testing program is defined as the area under the load deflection curve up to a 2.5 mm vertical slip. This level of slip was chosen as the boundary because the post-peak curve typically has leveled off at this point. Since it is not expressed as a ratio of areas under the curve, this toughness index is not nondimensional. The units as presented in this thesis are (normalized psi) * in. Here normalized psi refers to the average shear stress divided by the square of the cylinder compressive strength. The digital data
for several specimens was discontinued before a slip of 2.5 mm; in these cases it became necessary to extrapolate the curve to calculate the toughness.

4.1 KEYED JOINTS UTILIZING PLAIN CONCRETE

Under the main research program, three specimens were tested for each combination of the following parameters: Epoxy thicknesses of 0 mm (dry joint), 1 mm, 2 mm, and 3 mm and normal stresses of 100 psi, 200 psi, 300 psi and 500 psi. Additionally, several monolithic specimens were tested for comparison purposes. These monolithic specimens had no key or joint, and reinforcement was not provided across the failure plane. A more thorough description of these results is presented in [14].

Dry Keyed Joints

The shear stress-slip curve for a dry key joint with a confining pressure, $\sigma_c$, of 100 psi is shown in Figure 4.2. It can be seen that the load increased continuously up to almost 70% of the maximum load, then a drop occurred in the curve. This drop was accompanied by the formation of a crack at the bottom corner or root of the key which propagated away from the shear plane at a 45° angle (Figure 4.3a). Up to this point, the load was resisted by friction along the straight surfaces of the joint and by bearing at the bottom corner of the key. After this crack had formed, most of the load was transferred through a truss mechanism of shear transfer as described in [28]. Shear
Figure 4.2  Stress-slip curves for dry keyed joints at various confinements $\sigma_c$
Crack closes after failure

Figure 4.3 Cracking patterns for dry joints

(a)

(b)

(c)
forces were transmitted largely through compression struts which had formed parallel to the cracks in Figure 4.3b. The stress trajectories then extended so that the load was carried by bearing on the lower face of the key. The frictional resistance from the flat portion below the key was greatly reduced at this stage in the experiment. As the load increased, cracks started to join along the root of the key, some of them extending to the lower face of the key. Final failure occurred when the concrete in the compressive struts failed in compression. This was an extremely brittle failure characterized by a large and sudden slip between the two parts of the specimen (Figure 4.3c). The initial crack closed after the creation of a distinct failure plane since the load was now carried through frictional forces.

Since aggregates protruded from the sheared-off surfaces, the specimen could still resist a reduced load through aggregate interlock. Thus the shear stress-slip curve became horizontal at a slowly decreasing rate as the aggregates on the shear plane were smoothed out.

A similar behavior was observed for confinements of 300 and 500 psi, but, at the higher confining stresses, the drop in the load due to cracking at the bottom corner of the key (Figure 4.3a) was partially overcome. Additionally, the initial crack at the root of the key was not as large, suggesting that the higher confining stresses reorient the principle stress planes thereby reducing the tendency for a compression strut to
from. As a result, a more linear behavior was observed.

From regression of the test results the following equation is proposed for the shear strength of the keyed dry joints.

\[ \tau = 7.80 f'_{c} + 1.36\sigma_{c} \]

This equation, which considers both the area of the actual key and the area of the flat surface, quantifies the strength for a keyed section.

**Epoxied Key Joints**

Typical shear stress-slip curves for keyed epoxied joints are given Figure 4.4. The curves are linear prior to failure, and all specimens failed in a brittle manner. Failure was accompanied by a sudden slip between the two parts of the specimen thus indicating the brittle failure mode.

The cracking sequence for the epoxied joints is shown in Figure 4.4. The initial crack would form either in the root of the key or in the center of the shear plane at a load level close to the maximum load (Figure 4.5a). Immediately before reaching the peak stress, other diagonal cracks would form along the shear plane and rapidly interconnect. Load carrying capability would drop off quickly following formation of the cracked plane. Further deterioration of the concrete was evident in the post-peak region; typical post-peak cracking patterns are shown in Figures 4.5b and 4.5c. Notice that a crack progressed through the concrete behind the epoxy layer rather than through the interface. This behavior was noted in a significant number of tests, and it agrees with the
Figure 4.4 Stress-slip curves for epoxied key joints at various confinement $\sigma_c$
Figure 4.5  Cracking patterns for epoxied joints
observation of several other researchers who have found that the failure initiates in the concrete rather than in the epoxy or in the epoxy-concrete interface.

In Figure 4.4 it can be seen that as the normal stress increases, the failure load also increases. Test results indicate that the shear strength is linearly proportional to the confining stress. A solid relationship could not be established between the epoxy thickness and strength nor could a relationship be observed between the epoxy thickness and the joint stiffness. Here the stiffness is defined as the secant from the origin to the deformation at peak load.

From regression analysis, the following equation is proposed for the ultimate strength of keyed epoxied joints:

$$\tau = 11.1 f'_c + 1.20\sigma_c$$

Dry vs. Epoxied Joints

The dry joints tested were significantly weaker than the epoxied joints. The differences ranged from a 25% lower shear strength at a confinement of 100 psi to a 13% difference at a confinement of 500 psi. This strength differential can be explained by the fact that the dry key slipped while being tested, so that stresses were not distributed as evenly as in the epoxied keys. Consequently, there was a concentration of shear and compressive stresses on the lower face of the key. At higher normal stresses, the frictional forces were higher resulting in less slip. Since less slip occurred, the stress concentration was not as prevalent along the lower face, and the
joint acted more like a monolithic section.

Final failure for both dry and epoxied joints was brittle, but the dry joints exhibited more cracking behavior prior to failure.

The epoxied joints consistently yielded strengths which were greater than or equal to those for monolithically cast specimens. This improvement in strength is caused by the interlock provided by the shear key. In the monolithic specimen, load is transferred through shear stresses, and failure occurs when the maximum principal stresses along the failure plane reach the tensile strength of the concrete. In the keyed, epoxied section mechanical interlock provided by the key enhances the shear transfer mechanism, and the epoxy is able to develop a large contribution from the concrete on each of the glued surfaces.

The dry keyed joints were not as strong as the monolithic specimens. Although the interlock effect is still present, the contributions of the flat sections above and below the key are only a fraction of those in an epoxied section.

4.2 STRAIGHT FIBERS

Based on the brittle behavior which was exhibited in the unreinforced keys, fiber addition to the concrete was considered as a remedy to provide a more ductile behavior. Another possible consequence of fiber addition could be a higher
ultimate strength resulting from a more cohesive matrix. The findings from tests on fiber reinforced key joints will be presented in the following sections. The fiber testing groups are defined as series; Series KF1 through KF8 utilize straight fibers, and Series KF9 through KF11 incorporate hooked-end fibers. Results from straight fiber addition will be presented in this section, and the crimped-end fiber results will be given in Section 4.3.

Workability of the mixes was recorded only from a qualitative viewpoint. Slump tests are not representative because placement of fiber reinforced concrete typically requires vibration. Even at low volume fractions, SFRC usually will have no-slump until it is vibrated [4],[5]. To estimate the usefulness in construction practice, then, a description of the behavior of the mix under vibration is critical.

For the straight fibers, workability of the 1% $V_f$ mix was satisfactory, and placement was achieved with minimal vibration. Workability observed in the 2% $V_f$ mix was very difficult; it is doubtful that this mix could be applied in construction. Poor compaction was discovered in several mixes even though excessive vibration had been applied.

Dry Keyed Joints at 500 psi confinement

For the series under 500 psi confinement and no superplasticizer (KF1) the effect of fibers is manifested in slightly higher ultimate strength and significantly better
ductility (See Figure 4.6). The improved post-peak behavior is evidenced through a 47.9% toughness increase in conjunction with a 11.5% strength increase for the 1% $V_f$ specimen over corresponding values for the plain concrete. Similarly, the 2% $V_f$ specimen showed a 19.7 increase in strength and a 64.3% increase in toughness. From the curves it can be seen that the SFRC keys were able to maintain significantly higher post-peak load levels. Another indication of the improved ductility provided by SFRC lies in examination of the cracking sequences, and the cracking sequence for the 1% $V_f$ specimen is shown in Figure 4.7. For the KFl series fiber reinforced specimens cracking behavior was more extensive in the pre-peak loading curve than was seen in the plain concrete specimens. For the 1% $V_f$ specimen the first crack occurred at 89% of the ultimate load, and this crack formed at the root of the key and propagated at a 30° to the direction of loading (Figure 4.7a). A nonlinear trend in the load-displacement curves was present at an earlier stage as a result of extended cracking behavior. Additional inclined cracks formed in the key body at an angle of 20° to 40° to the direction of loading (Figure 4.7b). Final failure of the section occurred when these cracks coalesced as in Figure 4.7c.

The 2% $V_f$ key first crack load was 83% of ultimate while the plain concrete specimen did not develop the root crack until it had reached 96% of the ultimate load.

In general cracking patterns for the SFRC keys were similar to those for plain concrete; the pattern in Figure 4.7(a) would
Figure 4.6  Shear stress-slip for Series KF1: Dry key joints at 500 psi confinement, non-superplastized concrete, straight fibers.
4.7 Cracking sequence for Specimen KF1-B: Dry key joint at 500 psi confinement, 1% $V_f$ straight fibers
be observed at 85-90% of the ultimate load, and Figure 4.7(b) was typical of cracking at roughly 95% of the ultimate load. The primary difference between the plain and fiber reinforced concrete was that the crack widths were typically smaller for the SFRC specimens, and the nonlinear stress slip behavior was observed at an earlier proportion of the ultimate strength.

Following the testing series KF1, the remainder of the tests utilized superplasticizer in the concrete mix. As stated in Section 3.3, it was hoped that the reduced w/c ratio resulting from the use of superplasticizer would provide a matrix with improved bonding ability.

Test series KF1 and KF2 provide the only direct comparison between concretes with and without superplasticizer. Both of these series employ dry keyed joints at a 500 psi confinement. Series KF1 had no superplasticizer in the mix while Series KF2 specimens (Figure 4.8) contained superplasticizer. A comparison of the stress-slip curves for the 1% $V_f$ specimens shows that the non-superplasticized concrete provided higher strength and ductility gains. The 2% $V_f$ keys showed similar trends, but the differences were reduced. Obviously the results utilizing superplasticizer were somewhat disappointing, but poor alignment was suspected for the 1% $V_f$ superplasticized specimen. This error in alignment may have led to eccentricity of loading which in turn would have reduced the ultimate strength value. It should also be pointed out that the superplasticized SFRC produced higher ultimate strength values. Member size
Figure 4.8  Shear stress-slip for Series KF2: Dry key joints at 500 psi confinement, superplastized concrete, straight fibers
reductions in segmental bridges may be realized through the strength gains, and cost savings may be achieved as a result of the lighter sections. Based on these considerations it was decided to continue in the use of superplasticizer in the mix. In retrospect this may have been a poor decision. Even though the superplasticized concrete did provide higher absolute strength levels, the non-superplasticized concrete resulted in greater relative strength gains without the added cost of the admixture.

The superplasticized SFRC did exhibit good ductility when compared to plain concrete. The onset of cracking for the 1% $V_f$ occurred at 75% of the ultimate load, and non-linear stress-slip behavior initiated well before the peak load. The cracking sequence for this specimen mirrored that for the 1% $V_f$ specimen in series KF1 shown in Figure 4.7.

KF2 specimen C (2% $V_f$) began to develop the level of ductility which had been sought through the use of SFRC. Even though the onset of cracking did not occur until 88% of the ultimate shear strength, the post-peak behavior of this specimen was far superior to that for the plain concrete specimen. Examination of the stress-slip curve reveals the high reserve strength of the specimen following ultimate load. Notice that the slope of the descending curve is very gradual, and the post-peak inflection point occurs at a slip of 0.8 mm following the ultimate load.
Dry Keyed Joints at 100 psi Confinement

Series KF4 varied the fiber content for dry keyed joints at 100 psi confinement. At this lower confinement level the benefits from SFRC become even more apparent.

The 1% $V_f$ specimen provided a 15% increase in shear capacity and a 46.8% increase in toughness over the plain concrete key. The dramatic increase in ductility is evidenced by the similarity of this specimen's stress-slip curve to a steel-like behavior (Figure 4.9). The maximum load was sustained over a slip of 0.4 mm. The 2% $V_f$ key achieved a 32.4% increase in shear strength but only a 36.6% improvement in toughness.

The cracking sequence for the 1% $V_f$ specimen is shown in Figure 4.10. As is the case in plain concrete specimens, the initial crack forms at the root of the key. When this occurs, the frictional contribution from the flat surface is reduced as the joint expands, and the load drops off slightly (Figure 4.10a). The load increases once again as it is transferred primarily through compression struts which form between the inclined cracks orientated at roughly a 30° angle to the shear plane (Figure 4.10b). When the compressive strength of the struts in the plain concrete are exceeded, they fail in an extremely brittle manner; load carrying capacity of the plain concrete specimen was reduced by 76% in a period of less than 1 second at final failure. In the 1% $V_f$ specimen, this load drop occurred over a period of roughly 18 sec. This gradual behavior
Figure 4.9  Shear stress-slip for Series KF4: Dry key joints at 100 psi confinement, straight fibers
4.10 Cracking sequence for Specimen KF4-B: Dry key joint at 100 psi confinement, 1% $V_f$ straight fibers
occurs as individual fibers pull out of the matrix at random time and location intervals.

The initial root crack in the 2% $V_f$ specimen formed at a stress level which was 88% of the ultimate strength. Cracking patterns for this specimen were similar to those for the 1% $V_f$ specimen.

As reported previously, the 2% $V_f$ specimen attained lower toughness increases than were observed in the 1% $V_f$ specimen. This lower toughness value was not expected, and it may be explained in two possible manners.

First, the compaction of the concrete was most likely deficient. The presence of air voids would reduce the bonding area on the fiber surfaces, and consequently, the confinement provided by the fibers following the maximum load would also decrease. Degradation of the matrix would increase, and the strength contributions from aggregate interlock would be lowered.

Another possible reason for the steeper slope of the curve in the post-peak region relates to energy stored in the specimen and the loading frame during testing. Since higher strength values are obtained in the 2% $V_f$ specimen, more elastic energy is stored in the loading frame during the test. When a certain percentage of the fibers have pulled, the remaining fibers are no longer able to contain the strain energy stored in the testing frame, and a rapid failure mode is observed as the stored energy is released.
It may also be theorized that the larger numbers of fibers present at the higher volume fraction created many minute stress concentrations. The greater number of stress concentrations could have led to a more brittle failure as compared to the 1% $V_f$ specimen.

It must be pointed out that a more ductile behavior was still observed for the 2% $V_f$ specimen as compared to the plain concrete specimen; the loss of load carrying capacity at final failure for the 2% $V_f$ SFRC occurred over a time span roughly 4 times greater than that observed for the plain concrete section.

**Variation in Confinement at Constant Fiber Volume Fraction**

Fiber content was held constant at 1% and confinement levels were varied along 100, 300, and 500 psi in Series KF7 (Figure 4.11). Although there is no reference plain concrete keyed specimen with which to compare values, observations of the stress-slip curves reveals a more effective fiber contribution at lower confinement levels. This improved fiber contribution is evidenced in the post-peak stress-slip behavior. The slope of curve in this region is increasingly steep as confinement increases. The 100 psi specimen shows an ability to maintain the peak load over a slip of 0.4 mm, but no such plateau is evident at the 300 and 500 psi confinements. The descending portion of the 300 psi specimen occurred over a longer period than was observed for the 500 psi confinement.
Figure 4.11 Shear stress-slip for Series KF7: Dry key joints at various confinements $\sigma_c$, 1\% $V_f$ straight fibers
Assessment of Contributions from Fibers and Confinement

Referring to Series KF1, KF2, KF4, and KF7, it appears that there is a complex interaction between the ultimate strength and toughness as affected by confining stress and fiber volume fraction.

The results suggest that fiber addition is less effective at higher confinement levels; this observation was also made by Van de Loock [73]. Two primary explanations may be offered for this trend. First, the effect of fibers may be masked by the larger contribution of aggregate interlock at higher confinements. A Mohr-Coulomb type relationship may be assumed for the plain concrete specimens, and the fiber effectiveness most likely does not improve linearly with the confinement as does the shear force in the plain concrete. Secondly, since the ultimate strengths are greater at higher confinements, the effect of stored energy in the specimen and loading frame may cause the more rapid failures.

Although it appears that ultimate strength does increase with fiber volume fraction, a clear relationship was not observed for toughness gains when compared to fiber volume fraction. Regarding cases where the 2% $V_f$ scored lower toughness gains than the 1% $V_f$ specimens, the lesser toughness improvements may be a result of stored energy considerations at the higher ultimate strength values. It is also possible that the more brittle failure was caused by additional stress concentrations present at the 2% $V_f$ level.
**Epoxied Keyed Joints at 500 psi Confinement**

Straight fibers were used in conjunction with epoxy in testing series KF3 and KF6, and both were tested at a 500 psi confinement level. These series are shown in Figures 4.12 and 4.13, respectively. Specimen C (2% \( V_f \)) from the former series could not be tested at 9 days because of an equipment failure. This specimen was finally tested at 28 days, but its results are questionable because the concrete strength was not known exactly and normalization may be inaccurate.

These testing series suggest that shear strength and ductility gains are not as substantial when fibers are used in conjunction with epoxy. As mentioned in Section 4.1, epoxy reduces the stress concentrations at the root of the key and consequently reduces cracking behavior. Improvements from the use of SFRC would not be as large since benefits from fibers are largely manifested in mitigation of cracking behavior.

The cracking sequence for the 1% \( V_f \) specimen from Series KF6 is shown in Figure 4.14. It can be seen that this cracking sequence follows a similar pattern to that seen for the plain concrete specimens (Figure 4.5). The initial crack forms at a stress level close to the ultimate strength. In this case the early cracking occurred in the body of the key (Figure 4.14a). There is a minor plateau for this specimen as fibers are briefly able to maintain the load level as additional cracks form (Figure 4.14b). Eventually the cracks interconnect leading to
Figure 4.12 Shear stress-slip for Series KF3: Epoxied key joints at 500 psi confinement, straight fibers
Figure 4.13 Shear stress-slip for Series KF6: Epoxied key joints at 500 psi confinement, straight fibers
4.14 Cracking sequence for Specimen KF6-B: Epoxied key joint at 100 psi confinement, 1% Vf straight fibers
final failure of the specimen. (Figure 4.14c).

On the flat regions of the specimen above and below the key the effect of fibers is probably not significant. The fibers' contribution to the strength of flat epoxied concrete is limited because the failure surface typically does not extend very deep into the concrete mass, but instead it closely parallels or occurs in the concrete-epoxy interface. When the joint is epoxied the flat areas contribute substantially more strength to the overall capacity. In dry joints the contribution is from frictional resistance, but the flat epoxied surface strength may be estimated as

\[ \tau = 10.25/f'_c + 0.98\sigma_c \]

where \( \tau \) is the average shear stress and \( \sigma_c \) is the confining pressure [14]. Thus the epoxy provides strength gains over the dry specimen which are unaffected by fiber addition.

In many of the epoxied, keyed joints the ultimate load follows soon after the first crack appears. When cracks form in the key body, the remaining areas of the joint cannot withstand the additional load, and a rapid failure of the flat epoxied regions typically results. Once the flat epoxied surface has failed, its contribution is reduced to a frictional force. A larger portion of the load must then be carried by the SFRC in the key body. Thus, it seems that a higher demand is placed on the fibers at the onset of cracking in epoxied joints, and earlier fiber pullout results when fiber bond strength is inadequate. A steeper decline in post-peak strength, and hence a more brittle
behavior, is observed when failure occurs in this manner.

Cyclic Testing Series

Series KF5 and KF8 investigated the cyclic capabilities of dry SFRC joints under a 500 psi confinement. These two series had 1 and 2% fiber $V_f$, respectively, and Specimen A from each series was tested to estimate the ultimate strength of the joint at the specified fiber volume fraction (See Figure 4.15). The remaining two specimens in series were cycled at 90% and 95% of the ultimate load to analyze the ability of SFRC keyed joints to withstand repeated loadings. Figures 4.16, 4.17, and 4.18 show the stress-slip relationships for the cyclic specimens. Three of the four fiber reinforced specimens survived the full 30 cycles, but Specimen KF8-C (95% ultimate) failed on the first cycle, so it must be discarded.

The cyclic behavior of plain concrete keys was examined thoroughly in the main research project, and the results of these two preliminary test groups will be compared with this existing database. Result from the comparison of bridge joint cyclic tests are summarized in Table 4.1.

From this table it can be seen that the plain concrete dry joints were prone to failure before reaching the 30th cycle. For plain concrete specimens cycled at 95% of ultimate strength, cracking would typically initiate at the third or fourth cycle. The initial cracks propagated, and other cracks appeared in ensuing cycles until the specimen would finally fail
Figure 4.15  Shear stress-slip for Series KF5 and KF8: Dry key joints at 500 psi confinement, straight fibers
Figure 4.16 Shear stress-slip for Specimen KF5-B: Dry key joint at 500 psi confinement, 1% $V_f$ straight fibers, Cycled at 90% of ultimate load.
Figure 4.17 Shear stress-slip for Specimen KF5-C: Dry key joint at 500 psi confinement, 1% V, straight fibers, Cycled at 95% of ultimate load
Figure 4.18 Shear stress-slip for Specimen KF8-B: Dry key joint at 500 psi confinement, 2% $V_f$ straight fibers, Cycled at 90% of ultimate load
catastrophically. The specimens tested at 90% of the maximum load would also crack in the early cycles. Crack propagation was not as extensive for these specimens, and several were able to withstand the full 30 cycles.

Specimen KF5-C was tested at 95% of the estimated section capacity. A small crack formed at the root of the key after the third cycle, and by the 15th cycle three additional small diagonal cracks were observed along the key body in a pattern similar to that in Figure 4.7a. Extensive diagonal cracking was present at the 30th cycle. The stiffness of the key was progressively reduced during the cycling. Even as the stiffness decreased, the small

### Table 4.1 Cyclic Testing Summary

Comparison of plain concrete and concrete reinforced with straight fibers in dry key joints at 500 psi confinement

<table>
<thead>
<tr>
<th>Concrete Type</th>
<th>Cycles to Failure</th>
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<tbody>
<tr>
<td></td>
<td>90% Ult.</td>
</tr>
<tr>
<td>Plain Conc.</td>
<td>&gt;30</td>
</tr>
<tr>
<td>SFRC 1% $V_f$</td>
<td>&gt;30</td>
</tr>
<tr>
<td>SFRC 2% $V_f$</td>
<td>&gt;30</td>
</tr>
</tbody>
</table>
residual deformation accrued with each cycle remained fairly constant. Specimen KF5-B (1% $V_f$), cycled at 90% of ultimate shear strength, also formed an initial crack during the third cycle. At this reduced load, however, cracking did not propagate as extensively as in the previous specimen. Also, the key stiffness did not degrade as quickly.

2% $V_f$ SFRC was used in specimen KF8-B, and it was tested at 90% of the estimated shear strength. The first crack occurred in the body of the key during the second cycle, and a small crack formed at the root of the key during the fourth cycle. By the 18th cycle several diagonal cracks had formed across the shear plane; smaller, interconnecting cracks formed in subsequent cycles. Still, the crack arresting mechanism of SFRC was able to maintain the joint integrity through the full 30 cycles.

SFRC specimens displayed better ductility under repeated loadings when compared to plain concrete specimens. All three of the valid SFRC cyclic test were able to withstand 30 loading cycles at 90 or 95% of the estimated ultimate load. The plain specimens were not always able to sustain a full 30 cycles. It must be emphasized that this is only a preliminary step into the examination of the behavior of fiber reinforced keys under repeated loadings.
4.3 CRIMPED-END FIBERS

Series KF9, KF10 and KF11 utilized crimped-end fibers and the specimens in each of these series were loaded monotonically to failure. Crimped-end fibers were employed in an effort to prevent fiber pull-out through better interlock.

Workability of the 1% $V_f$ mix was satisfactory, and placement was achieved with minimal vibration. Workability observed in the 2% $V_f$ mix was very difficult; it is doubtful that this mix could be applied in construction. Poor compaction was discovered in several mixes even though excessive vibration had been applied. It is estimated that the workability of the crimped-end fibers is slightly more difficult than that for the straight fibers.

DRY KEYED JOINTS AT 100 PSI CONFINEMENT

Beginning with series KF10, dry joints at 100 psi confinement, it can be seen that the crimped-end fibers produced substantial strength and ductility gains (Figure 4.19). Compared to the plain concrete specimen, the 1% $V_f$ key carried 23.5% more shear stress and had 71.9% higher toughness. Following formation of the root crack (as in Figure 4.10a), the 1% $V_f$ specimen does not experience a drop in load due to separation of the lower flat surface. Here the crimped-end fibers display the ability to control crack growth where the straight fibers were unable to do so. The continued deformation evident at ultimate strength levels is further testimony to the improved ductility provided
Figure 4.19 Shear displacement for Series KF10: Dry key joints at 100 psi confinement, crimped-end fibers
by the crimped-end fibers. The first crack load for this specimen is at 77% of the ultimate strength.

Specimen KF10-C (2% $V_f$) shows the highest strength and ductility gains from all of the SFRC tests. The ultimate shear stress of 1411 psi is 67.5% greater than in the control specimen, and toughness is increased by 200%. The first crack appeared at a stress level of 720 psi, only 50% of the ultimate load. This first-crack stress is only 20% higher than that of the plain concrete specimen. This key does not sustain maximum loading for the extended displacement that is visible in KF9-B (1% $V_f$). This may occur due to the early cracking manifested in the 2% $V_f$ specimen. After the matrix cracked, the fibers carried higher proportions of the load, and the shear planes had slipped 0.7 mm between first crack and ultimate load. During this deformation, the fibers were increasingly stressed, and fiber efficiency apparently decreased. Near ultimate levels the load was carried primarily through truss action along the compression struts. This judgement is based on the observation that all of the cast surfaces except for the back of the key and the lower key face had separated from Section A. Thus it appears that the crack arresting mechanism of the fibers extended the capacity of the compressive struts significantly.

**Dry Keyed Joints at 500 psi Confinement**

Series KF11 utilized crimped-end fibers in dry keys tested at 500 psi confinement (Figure 4.20). This series yielded
Figure 4.20 Shear displacement for Series KF11: Dry key joints at 500 psi confinement, crimped-end fibers
surprising results in that the strength gains were roughly equal to the toughness gains. The 1% $V_f$ specimen carried a 31.0% higher ultimate load than the plain concrete, yet it yielded only a 28.5% increase in toughness. Similarly for the 2% $V_f$ specimen the strength gain was 25.2% as compared to a 31.0% improvement in toughness.

First cracking occurred in the 1% $V_f$ specimen at 75% of the ultimate load. It is theorized that the disappointing toughness values for this specimen are due to a straightening out of the fiber. Figure 4.21 shows crimped-end SFRC keys which have been separated from the specimen. From this figure it can be seen that the deformations on several of the fibers have been straightened out. It is postulated that the improved anchorage from these crimped-end fibers was maintaining the integrity of the matrix at high stress levels. As the fibers were loaded eventually a fraction begin to straighten out. When some of the fibers pulled out, the confinement on the matrix was decreased, and the load carrying capacity rapidly decreased. Since the matrix had deteriorated less force was required to pull out the remaining fibers. This brittle type of failure could possibly be improved through the use of higher volume fractions. The 2% $V_f$ specimen in this series must be disregarded, however. The strength of this specimen is lower than that for the 1% $V_f$ specimen. Inadequate compaction is the suspected cause of this poor performance.
Figure 4.21  Body of key from crimped-end SFRC specimens
Epoxied Keyed Joints at 500 psi Confinement

Series KF9, shown in Figure 2.22, consisted of epoxied joints under 500 psi confinement. Observation of these curves yields several interesting results. First, note that the SFRC specimens display higher stiffness values under the increased loading. This is an indication that the steel fibers control the growth of smaller cracks and thus preserve the integrity of the matrix. Also note that strength gains were only 7.9 and 20.2% for the 1% $V_f$ and 2% $V_f$ specimens, respectively. As discussed in Section 4.2, the fibers are not able to improve the strength as in the dry joints because of the higher strength contribution provided by the flat regions above and below the key. For the 1% $V_f$ specimen note that the fibers were unable to resist the additional load released upon failure of the bonding agent. Because the fibers pulled out quickly after the peak, the toughness for the FRC key was only 8.4% higher than that for the plain concrete section. The 2% $V_f$ specimen, however, is fully able to compensate for the loss load carrying ability of the flat surfaces, and a 56.3% higher toughness value is obtained.

For specimens A (plain concrete) and B (1% $V_f$) in this series, decline of the load carrying capability quickly followed the formation of the first crack. In specimen C, however, a significant load carrying capacity is present even after extensive cracking has occurred.
Figure 4.22 Shear stress-slip for Series KF9: Epoxyed key joints at 500 psi confinement, crimped-end fibers
4.4 COMPARISON BETWEEN FIBER TYPES

From the discussions provided in the previous two sections it is evident that the addition of steel fibers to the concrete matrix provides behavioral improvements in segmental bridge shear keys. These improvements are manifested in several areas. First, the ultimate shear strength of the section is increased by as much as 67% compared to the plain concrete control specimen. Secondly, significant toughness gains are shown, and based on percentages, these gains are usually higher than the strength gains. The value of greater toughness levels is evident in several areas. First, the bridge would be better able to withstand heavy repeated loadings such as earthquake loadings. Serviceability will improve in that the fibers inhibit the propagation of microcracks. During the construction phase, damage of keys from careless handling will be prevented. Thus fiber addition does provide an improved material behavior through either straight or crimped-end fibers. This section will address the differences between the performance of the two fibers.

Although an economic analysis will not be presented in this thesis, fiber costs are an important consideration, and they will be mentioned here. The FlexTen straight fibers cost $0.35 per pound when purchased in bulk. The Dramix hooked-end fibers cost $0.69 per pound. Thus it is calculated that the hooked-end fibers are roughly twice as expensive.

The crimped-end fiber reinforced key joints generally showed
better results than those utilizing straight fibers. Gains in strength and ductility were delivered as a result of the crimped-end fiber's better resistance to pull-out. The deformation in the fiber does create a stress concentration, however, and in one series of tests a less ductile post-peak behavior was observed for the crimped-end fiber. In general, though, the strength and post-peak capacity of concrete reinforced with crimped fibers was superior to concrete with straight fibers added.

As mentioned in the literature review, several other researchers have performed pullout tests in which an individual fiber is embedded in a mortar matrix. Hara [24] reports the bond strength of the deformed fiber to be 38.2 kg/cm² while the corresponding value for the straight fibers was 2.7 kg/cm² at a L/4 (7.5 mm)

Figure 4.23 Experimental fiber bond-slip relationships [39]
embedment. Load-displacement curves for the pullout tests reveal that extremely brittle failure mode for the crimped fiber, but the flat fiber is able to maintain high load levels for a full 3 mm.

The results of pullout tests by Lim et.al. [39] are shown in Figure 4.23. In these experiments the strength differences were not as dramatic between the deformed and straight fibers. Additionally, the crimped-end fiber displays a more ductile behavior. The fiber factor proposed in [52] includes a bond factor, $d_f$. This factor is specified as 0.5 for undeformed fibers and 0.75 for crimped fibers.

Thus it can be seen that the pullout force for the crimped end fiber is greater than that for the straight fiber. Ratios of the individual fiber pullout strengths are much greater than the ratio of the fiber bond factors, so it is likely that difference in matrix strength gains will not be proportional to the disparity in pullout loads for the two fiber types.

Dry Keyed Joints at 100 psi Confinement

At 100 psi confinement and dry joint condition, strength gains from the crimped-end fibers were significantly higher than those from the straight fibers. The strength improvements for the deformed fibers are 23.5% and 67.5% for 1% and 2% $V_f$, respectively. The straight fibers scored 15.0% and 32.4% gains at the same volume fractions.

The toughness was also considerably higher in specimens
utilizing deformed fibers. The 1% and 2% $V_f$ toughness indices were 0.600 and 1.044 psi*in. for the crimped fibers and 0.512 and 0.477 psi*in. for the straight fibers. Compaction is the suspected cause of the unexpectedly low toughness value for the 2% $V_f$ straight fiber specimen. Figure 4.24 shows the shear stress-slip curves for crimped-end and straight fibers at 1% $V_f$ and 100 psi confinement. The plain concrete curve is shown on the same figure as well. Several interesting observations can be made. First, the stiffnesses are roughly equal. This is to be expected because the steel volume in the matrix rather than the fiber type is influential on the composite modulus, $E_{ct}$ before cracking [53].

Note that following initial crack formation (Figure 4.10), the straight fiber specimen displayed a slight load drop as was observed in the plain concrete specimen. The crimped-end fiber specimen maintained a smooth curve throughout the onset of cracking. When the root crack opens, load drops as a result of the loss of frictional force along the lower flat surface. Since the crimped-end specimen did not observe a load loss here, it is theorized that the improved bond from the crimped-end retarded crack growth much more effectively than the straight fiber.

It can be seen that the crimped-end specimen maintains a 15% higher load level than the straight fiber specimen through the plateau region in the figure. Here both tests show improved ductility, but the deformed fiber is better able to maintain the
Figure 4.24  Stress-slip curves comparing fiber types for dry key joints at 100 psi and 1% fiber $V_f$
matrix integrity. In the post-peak region the strength difference is even greater. It is likely that substantial slip has occurred in both fiber types at this point. The higher resistance provided by the crimp improves strength through (1) better direct tensile force from inclined fibers; (2) greater dowel action; and (3) improved aggregate interlock from a matrix with lesser crack widths. See Section 4.5 for a more thorough discussion of these factors.

Dry Keyed Joints at 500 psi Confinement

Three series of dry keys were tested at 500 psi. KF1 and KF2 utilized straight fibers, but superplasticizer was not added to the first mix. KF11 utilized crimped-end fibers. Only KF2 and KF11 will be compared here since their matrixes are similar, that is, of superplasticized concrete. The crimped-end SFRC recorded substantially higher strengths, but toughness values were closer in these series. It must be pointed out, however, that poor compaction of the 2% V fiber crimped-end specimen may have prevented it from realizing full potential strength and ductility improvements. Nonetheless, a valuable comparison of the 1% V fiber specimens can be made, and these curves are shown along with that of the control keyed joint specimen in Figure 4.25.

From this figure it can be seen that the crimped-end fiber key achieved a significantly higher ultimate shear stress than the straight fiber key, but it failed in a rather sudden manner. This non-ductile behavior could be a result of the brittle
Figure 4.25  Stress-slip curves comparing fiber types for dry key joints at 500 psi and 1% fiber $V_f$
pullout behavior reported by Hara [24]. As the crimped-end fibers pulled out of the matrix suddenly, the degradation of the matrix may have led to the progressive pullout of other fibers at an accelerated rate. It is postulated that 2% $V_f$ may have shown a more ductile behavior if properly compacted. If more fibers are contributing to the integrity of the matrix, then the effect of individual fiber pullout might not be catastrophic. This theory is supported by the ductile behavior displayed by the epoxied crimped-end SFRC specimen at 500 psi confinement and a 2% $V_f$ (Figure 4.22).

A more even behavior was observed for the straight fibers. Note in Figure 4.8 the ductile behavior of the 2% $V_f$ specimen which may have resulted from gradual pullout of the straight fibers.

Epoxied Keyed Joints at 500 psi Confinement

Three testing series were conducted on epoxied specimens at 500 psi confinement. Series KF3 and KF6 utilized straight fibers while crimped-end fibers were used in series KF9. With the exception of the crimped-end 2% $V_f$ specimen, all of these tests showed disappointing gains in shear strength and toughness. From the examination of the stress-slip curves in Figures 4.12, 4.13, and 4.22, it is evident that the slopes of the post-peak curves for the fiber reinforced specimens are roughly equal to those for plain concrete keys. Only the hooked-end 2% $V_f$ key shows strength increases and the ability to
sustain maximum load following the peak.

The brittle behavior exhibited by the majority of SFRC keys may have resulted from the large transfer of stress which occurs when the flat regions crack. As discussed in previous sections, the epoxied specimens exhibit limited pre-peak cracking behavior. When cracks form across the flat regions above and below the keys, the load carrying mechanism becomes one of friction, and the fibers do not contribute significantly in these areas. Following the formation of a definite cracked shear plane, the body of the key must transmit a larger proportion of the load. In the series KF3, KF6 and KF9(B), the fibers were unable to resist the energy released from the testing frame after cracks had formed. Rapid pullout of fibers followed which was indicated by steep post-peak stress strain curves. At a volume fraction of 2% the crimped-end fibers were able to withstand the load transfer at cracking. Apparently the level of reinforcement required to resist the post-cracking energy release in epoxied joints is greater than 1% of crimped-end fibers, and straight fibers were not able to develop this strength at the volume fractions tested.

4.5 MODELING OF FIBER CONTRIBUTION

In this section a theoretical examination of the failure process of the keyed joint will be presented. Basic modeling will be used to describe stress transfer at various stages
during the tests, and qualitative estimation of the joint strength will be made. This discussion will focus primarily on the dry keyed joints, and concepts described for the dry joints will be applied briefly for the epoxied joints.

Shear transfer during the test may be divided into three distinct stages: (1) uncracked concrete; (2) Diagonal cracking; (3) Formation of a distinct cracked failure plane in concrete.

Idealization of the shear transfer mechanism for stage 1 is shown in Figure 4.26. The two principal mechanisms of shear transfer are friction and a combination of shear and compressive stress trajectories. Frictional forces are present along the flat surfaces above and below the key and at the vertical face at the back of the key. The magnitude of the frictional force is dependent upon the confinement level, and the coefficient of friction, $\mu$, may be conservatively taken as 0.5 [14],[30],[47]. Fiber addition does not significantly effect the frictional force because troweling produces a finish similar to that obtained in plain concrete. Remaining load is transferred via shear forces through the key body. As the stress trajectories near the lower face of the key, they are transmitted in a compression mode. Fiber addition does improve the performance in this aspect of load transmission. First, a more stiff loading behavior is observed in several tests, thus indicating that the fibers limit microcracking in the matrix thereby producing a higher composite modulus of elasticity, $E_{ct}$.
Figure 4.26 Stage 1: Uncracked specimen
Additionally, the fibers produce a higher first-crack load so that the structure may be designed for greater service loads.

In Stage 2, large diagonal cracks have formed at an angle of roughly 30° to the shear plane (See Figure 4.27). This angle decreases slightly with confining pressure, due to the reorientation of the principle stress planes at the higher stresses [14]. Especially at lower confinements, a slight expansion of the joint occurs at this stage, and, as a result, the contributions from frictional forces along the upper and lower flat faces are reduced.

The mechanism of shear transfer at this stage can be described by the truss model described in References [28] and [75]. This theory states that shear forces are transferred through a truss mechanism. The compression is carried in the concrete through struts which form between the diagonal cracks. In conventionally reinforced concrete, tensile forces are carried by mild steel reinforcement across the shearing plane. When this theory is applied to a keyed joint, the compression struts are observed, but, since conventional reinforcement is absent, the tensile forces exist elsewhere. It is theorized that the tensile component is provided by the external prestressing. According to the truss theory as outlined by Hsu [28], final failure of the concrete occurs when the compressive strength of concrete in the strut is exceeded. However, it was found that compressive strength of the diagonal struts can be much lower than the standard cylinder strength. The benefits of fiber
Figure 4.27 Stage 2: Diagonal cracking
reinforcement are realized in contributions to the integrity of the truss system. The reduction in concrete compressive strength in the strut may result largely from loss of biaxial confinement when the diagonal cracking occurs. The crack arresting action of the fibers helps preserve some degree of the confinement present in the uncracked matrix. The fibers also contribute directly to the compressive behavior of the concrete. Although the fibers do not significantly increase the concrete compressive strength, they do provide a much higher ductility [18]. This post-peak load carrying capacity provides for a more gradual destruction of the strut mechanism at ultimate failure loads.

Stage 3, the final failure stage, is the formation of a distinct cracked plane parallel to the direction of loading. Frictional forces may still be present along the flat surfaces, but these will be greatly reduced due to expansion of the joint following cracking. The greatest portion of the load is transferred through the key body to the lower key face. The total shear force across the cracked plane is assumed to be the sum of three components: (1) Aggregate interlock, $V_a$; (2) tensile forces from inclined fibers, $V_{ft}$; and (3) dowel action provided from fibers oriented roughly transversely to the shear plane, $V_{fd}$ (Figure 4.28). Aggregate interlock is effected primarily by crack width and confinement level. Hence it is intuitive that the crack arresting mechanism of the fibers will limit crack width and thereby provide a higher $V_a$. A direct
Figure 4.28 Stage 3: Final cracking
tensile contribution to shear stress will be realized for fibers orientated at an angle, $\alpha$, between 30 and 70 degrees with the horizontal. The unit strength of this component is a function of fiber density, orientation, embedment length, and bond stress. From [53] the quantity may be calculated as

$$v_b = 0.41 \cdot \tau \cdot F \cdot R'$$

where $\tau$ is the fiber bond stress, $F$ is the fiber factor, and $R'$ is constant which accounts for the limited range of orientation angles that will be effective in this case. The final component of shear force, $V_{fd}$, is the dowel action from fibers orientated transversely to the failure plane. The extent of this contribution depends upon matrix splitting strength and fiber density [26].

The three stages are also exhibited in the epoxied joints, but the duration of stage 2 is typically much shorter.

During stage 1 the contribution from the flat surfaces is much greater than in the case of a dry joint. The ultimate shear strength of the flat epoxied sections may be calculated as

$$\tau = 10.28 f'_c + 0.19 \sigma_c$$

which is considerably greater than the frictional force [14]. As outlined in Chapter 2 and Section 4.1, the use of epoxy nearly restores the full strength that would be obtained for a monolithic concrete section. The effects of fiber reinforcement are mainly realized in the higher cracking strength and increased stiffness.

The flat surface contributions are greatly reduced when
failure occurs in these regions, and as a result, the body of the key is required to absorb large amounts of energy. It is possible that the compression struts are unable to resist this massive transfer of stress, and hence the observation of stage 2 is short lived.

The load transfer mechanisms at stage 3 are very similar to those outlined for the dry joints. One slight difference may be an increase in frictional force on the flat faces due to the uneven cracked failure plane.

4.6 **INDUSTRY APPLICATION**

Although SFRC has not been utilized extensively in structural members to date, its use has been exploited in many other construction applications. In this section several areas of SFRC utilization will be outlined. Following these discussions, a description of preliminary structural uses will be presented. To conclude this section several possible schemes for the introduction of SFRC to precast concrete segmental bridges will be proposed.

The most common application of SFRC has been in slabs and pavements [27]. The added toughness from SFRC provides various benefits for different applications. Airport runways are exposed to high impact loadings from aircraft, and SFRC is typically applied in pavement overlays whose depth varies from 5 to 10 in. [27],[23]. Deterioration of bridge decks may also be mitigated through the use of SFRC. This deterioration of the
decks is especially prevalent in colder regions where studded tires and snow chains provide additional impact forces. SFRC overlays or deck replacements have been used on many bridges to extend the replacement life [70]. The benefits of SFRC are utilized in several aspects of applications in industrial warehouse floors. Impact loadings from forklifts or other equipment can spall and crack traditional R/C slabs. Thermal cycles resulting from furnace operations will likewise deteriorate slabs on grade. SFRC has been used in these areas to extend slab replacement time and thus limit down time of factory operation [34].

Fibers are added to mortar mixes frequently for use in steel fiber reinforced shotcrete (SFRS). SFRS has found its largest application is mining operations where it is used for forming linings in tunnel walls. This material has also been utilized in rock slope stabilization work, brick bridge arch strengthening, dome structures and canal linings [60].

Some military structures utilize SFRC. Improved toughness from the fibers provides added resistance to blast forces [34].

SFRC is receiving increased attention in a structural applications such as thin shells and precast elements. Many researchers have investigated the expanded usage of SFRC in larger structural elements. Tests have been performed to understand the mechanical behavior of SFRC in the various stress modes: compression, tension, flexure and shear.

The application of SFRC in combined shear and flexure looks
especially promising when used in conjunction with conventional reinforcement [18], [68]. Consensus among investigators seems to be that the use of fibers without longitudinal reinforcement does not provide enough ductility. Several researchers have proposed that fibers can replace the secondary reinforcement (stirrups). In sufficient quantity, fibers in beams with only longitudinal reinforcement were able to prevent shear failures and thus promote full flexural failures [11],[39],[41],[53]. Other test have indicated that fibers may be more beneficial as a replacement for some fraction of the secondary reinforcement [10].

One leading-edge example of SFRC in structural members is in the Ribtec offices in Gainsville, Ohio. This structure was constructed almost entirely of SFRC, and conventional reinforcement was used in only a few locations in the structure.

Two proposals will be made here for the implementation of SFRC in precast concrete segmental bridges. Under the first plan, the use of SFRC would be limited to the shear key regions. In this demand-specific application, SFRC would only be used in regions where the placement of conventional reinforcement is not practical, is in the shear keys. This plan is feasible because of the controlled fabrication process which exists at the precast yard. While plain concrete is being placed in the segment middle regions, a 6 to 12 in. width of SFRC could be placed in the web end regions, thereby providing a more ductile material in the joint regions.
The second plan calls for the use of SFRC throughout the entire segment. The added expense of the fibers could be justified in two areas. First, fibers could replace some of the secondary reinforcement, and cross-sectional dimensions could be reduced as well. Also, since the bridge deck is cast monolithically with the box girder, the advantages from fibers could be realized in an improved wearing surface. As outlined in [48] and [55] several box girder bridges have not utilized any additional wearing courses.

Hence, it appears that SFRC will see increased employment in structural applications. The benefits from SFRC may be realized in segmental bridge construction in one of the two plans outlined in this section.
Chapter 5

SUMMARY AND CONCLUSIONS

5.1 SUMMARY OF IMPORTANT FINDINGS

In an effort to improve the strength and safety of joint regions in precast concrete segmental bridges, an exploratory experimental program was conducted which utilized steel fibers in the concrete mix. Shear tests were conducted on an elemental level; a 1:1 model of a shear key used in an actual bridge was incorporated in a push-off testing arrangement. Epoxied and dry joint conditions were examined under different bridge prestress levels, and the fiber content in the keys was varied along 0%, 1%, and 2% by volume. Straight and crimped-end fibers were tested monotonically, and two other testing series looked at the cyclic behavior of the straight SFRC. Results from these experiments are summarized in Table 5.1.

Crimped-End Fiber SFRC

Crimped-end fibers produced consistently higher strengths than the straight fibers. For the dry joints, crimped-end fibers produced strength gains as high as 31.0% at 1% $V_f$ and 67.5% for the 2% $V_f$ fraction. Improvements in toughness were even more substantial; the 1% $V_f$ produced gains up to 71.9% and the 2% $V_f$ resulted in toughness 200% higher than in the plain concrete control specimen in one instance. Improvements listed here are
**TABLE 5.1 SUMMARY OF RESULTS**

Strength and Toughness Increases for SFRC Specimens as Compared to Plain Concrete. Based on Normalized Shear Stress

Percent Increases: Listed as strength / toughness

<table>
<thead>
<tr>
<th>Key Parameters</th>
<th>Straight Fibers</th>
<th>Crimped-End Fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1% $V_f$</td>
<td>2% $V_f$</td>
</tr>
<tr>
<td>Dry, 100 psi</td>
<td>15.0/46.8</td>
<td>32.4/36.6</td>
</tr>
<tr>
<td>Dry, 500 psi</td>
<td>6.0/20.3</td>
<td>13.2/42.8</td>
</tr>
<tr>
<td>Dry, 500 psi *</td>
<td>11.5/47.9</td>
<td>19.7/64.3</td>
</tr>
<tr>
<td>Epoxied, 500 psi</td>
<td>4.6/46.8</td>
<td>12.8/36.6</td>
</tr>
</tbody>
</table>

* This series utilized non-superplasticized concrete, all other series used superplasticizer in the mix.
based on comparisons with plain concrete specimens which were cast with each mix. It should be noted that several of the crimped-end fiber tests did not exhibit a particularly ductile behavior; the post-peak load carrying capacity declined at a rate similar to that found in plain concrete. One possible explanation for this observation could be that the specimens failed in a brittle manner because of the higher energy stored in the system at the higher ultimate strength values. Another possibility is that the higher fiber volume fractions (2%) produced many more stress concentrations. While the fibers were still able to provide a greater ultimate strength, once cracking began to progress, a rapid degradation of the matrix ensued.

It would appear that toughness gains as compared to plain concrete decrease with the prestress level. The results do not present a clear relationship between confining stress and ultimate strength. The reduced toughness gains are likely attributable to the rapid release of energy stored in the matrix at the higher confining stress. Also, a masking effect of fiber contribution could result from the greater shear strengths at the 500 psi.

When used in conjunction with epoxy, fibers were generally less effective. Here again release of energy at higher ultimate loads may have led to the rapid failure. Since epoxied joints develop greater strengths than dry joints and since there is less cracking behavior before ultimate load, the fibers may pull-out rapidly when energy is released at ultimate load.
There apparently exists a fiber efficiency threshold which lies between 1 and 2% $V_f$ of the crimped fibers. The former percentage was unable to maintain a ductile post-peak behavior while the 2% $V_f$ produced an extended load carrying capacity even after extensive cracking had occurred.

**Straight Fiber SFRC**

Straight fibers also reported strength and toughness increases, but these were not as impressive as in the case of deformed fibers. Maximum strength gains were 15% and 32.4% for the 1 and 2% $V_f$, respectively. Toughness values were increased by as much as 47.9% and 64.3% for the 1% and 2% volume fractions. A decrease in fiber efficiency was observed as the bridge prestress level increased. This effect may be caused by two different factors. Since the joint strength increases with confining pressure, the energy levels are higher at the larger ultimate strengths, and energy release may lead to a more sudden failure. Masking of the fiber contribution by increased concrete shear strength may also occur.

Straight fibers were fairly ineffective when used in conjunction with epoxy. At practical volume fractions, the straight fibers apparently could not achieve the critical fiber effectiveness mentioned in the preceding paragraph. The epoxied specimens utilizing straight fibers were unable to withstand the load transfer following cracking of the epoxied surfaces; all of these specimens failed in a brittle manner.
Two fiber series were tested under cyclic loadings, and these initial tests showed a greater ability to withstand repeated loadings when compared to plain concrete.

5.2 CONCLUSIONS

Although these early tests do not provide definitive conclusions due to the limitations of the tests and test parameters, the following conclusions regarding fiber reinforcement will be given:

A) Improvements from SFRC as compared to plain concrete
   i) Evidence reported in previous chapters clearly indicates that the addition of fibers creates a stronger and more ductile concrete which can be used advantageously in the construction of precast concrete segmental bridges.

B) Comparisons of the Effects of Different Fiber Types
   i) Joints reinforced with crimped-end fibers showed higher strength and better ductility in most instances as compared to behaviors exhibited by straight fibers, but a more brittle behavior was observed in several specimens which failed at higher ultimate strength levels.
   ii) Workability for the straight SFRC was slightly better than that for concrete with the crimped-end fibers, and the former was also half as expensive per pound.
C) Conclusions Regarding the Effects of Fibers in General

i) It appears that fiber reinforcement is not as effective at higher confinement levels.

ii) Ultimate strength of the joints increased with increasing fiber content under the range of fiber contents studied in this program (0%, 1%, 2%).

iii) Toughness did not display a clearly defined relationship with fiber content, but it appears that ductility increases begin to level off at a volume fraction greater than 1%.

iv) The dry joints generally realized better results from fiber addition than did the epoxied joints.

v) When workability and strength are considered simultaneously the optimal volume fraction would appear to lie between 1% and 1 1/2%. This judgement is based on the observation that the 2% $V_f$ mix is too stiff for construction purposes, but the 1% $V_f$ composite experienced rapid fiber pullout in some instances.

vi) It is recommended that the use of a more liquid mix be investigated. In this manner, a higher volume fraction could be utilized while adequate workability was maintained.

vii) Limited results from this testing series based on increases relative to plain concrete indicate that the use of superplasticizer does not contribute any
significant increase in matrix bond strength.

To conclude, it appears that the use of SFRC in precast concrete segmental bridges is indeed plausible. The optimum mix might incorporate deformed fibers at a volume fraction on the order of 1 1/2\%. Toughness gains from the fibers would allow reductions in web and deck thicknesses, and the life of the deck would be increased through fiber addition.

5.3 DIRECTIONS FOR FUTURE RESEARCH

The experimental program described in this report may only be categorized as preliminary. Several additional parameters should be investigated, and slight changes are necessary within the existing program.

a) Matrix.

The concrete mix used in this program clearly is not optimal. A more workable mix is needed which yields improved bond in the hardened matrix. The effects of greater amounts of superplasticizer should be examined in a limited number of tests to see if that will improve bond. More likely, Fly Ash addition or substitution for cement will result in better matrix characteristics. Increases in workability due to the use of a higher w/c versus an increase in superplasticizer should be investigated.
b) Control Specimens.

In addition to plain concrete, SFRC control cylinders were cast with each testing series. Fiber addition generally does not improve compressive strength, and a tensile failure mode is more desirable for the control specimens. Split cylinder, direct tension, or a modulus of rupture from 4 x 4 x 14 in. beams should be considered.

c) Fiber Type and Quantity.

Both straight and crimped end fibers should be examined in further tests. Although the latter yielded a better mechanical behavior, the cost savings in the straight fibers merit continued examinations. A greater number of specimens per cast is desirable so that more volume fractious can be examined. If four specimens are tested, volume fractions of 0, 1, 1.33, and 1.67% could be used.

d) Cyclic Loading.

The effectiveness of crimped-end SFRC should be examined under repeated loading. In addition to the near ultimate loadings which were cycled under the current program, several test series should examine the cyclic behavior at load levels of roughly 60-70% of ultimate.

e) Modeling of Shear Behavior

Analytical work in the future should include the development of a material model for the SFRC used in the bridge keys. This work could be complemented by in-depth
finite element analyses; these analyses could predict crack widths at various stages of failure and the contribution from the fibers could be superimposed across the cracked surfaces.
REFERENCES

Note: References are arranged alphabetically and then in order of date according to the first author.


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