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Analytical Modeling of Fiber Reinforced Post-Tensioned Concrete Anchorage Zones

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ANALYTICAL MODELING OF FIBER
REINFORCED POST-TENSIONED
CONCRETE ANCHORAGE ZONES

By

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ABSTRACT

The use of post-tensioning in bridge girders causes tensile bursting stresses to occur some distance ahead of the anchorage device in a region known as the general zone. Large amounts of mild steel reinforcement are placed in this area of the bridge girder in order to resist these highly tensile stresses. This causes congestion in the area of the steel and poses difficulty during concrete placement. The objectives of this study were to determine the feasibility of reducing the mild steel reinforcement by adding fibers to the general zone and to determine the impacts of doing so. Fiber reinforced concrete (FRC) improves the mechanical properties of non-fibrous concrete. So it is expected to support the proposed reduction of mild steel reinforcement in the post-tensioned anchorage zone. The first phase of the study involved researching past studies on the use of FRC in order to determine the material and mechanical properties pertaining to the fibers. Steel fiber was deemed to be the most useful for enhancement of non-fibrous concrete properties. The second phase of the study was to determine a realistic and reasonable specimen for FRC application. The pier segment of a currently used bridge in Florida was chosen. This selection was based on having common and less complex geometry. After selection, information was gathered about the segment such as the volume of concrete, mild steel reinforcement details, and post-tensioning system details. In the final phase of this study, a finite element model was developed for the segment using design required mild steel reinforcement. Using the initial model, duplicate models were analyzed with varying steel fiber volumes. The theoretical results indicate that a maximum reduction of 65% of the design mild steel reinforcement can occur when replaced by 0.50% steel fiber to the concrete volume of the general zone. However, it is recommended that a mild steel reinforcement reduction of 50% be replaced by 0.50% steel fiber in order to stay conservative and safe. It was also observed that higher volumes of steel fiber could increase stresses in the general zone. Therefore, it is recommended that experimental testing of these procedures be done for complete verification.
INTRODUCTION

1.1 Post-Tensioned Concrete

The use of post-tensioned concrete in civil engineering structures has become more and more mainstream, especially in bridges. This method of construction threads high strength steel tendons through ducts and anchorages that are placed in the section before the concrete is poured. Once the concrete has reached sufficient strength, the tendons are jacked against the section creating tension through the tendons and compression through the concrete section. Since concrete performs well under compression and steel performs well under tension, an optimal capacity is reached. This solution allows for longer spans and smaller cross-sections in bridges. Figure 1.1 shows a diagram of post-tensioning being used in a segmental bridge. Figure 1.2 shows Victory Bridge over the Raritan River in New Jersey with a record-breaking 440-foot main span, accomplished through the use of post-tensioning.

![Figure 1.1: Example of Post-Tensioning used in Segmental Construction](image-url)
The main component of this method is the tensile force created in the tendons. Special anchorage devices, as shown in Figure 1.3, are used to transmit the compressive force to the concrete. Upon reaching the concrete, the force spreads out to eventually reach a nearly linear stress distribution as seen in Figure 1.4. The region of concrete that is affected by the introduction of this force is known as the anchorage zone.
1.2 Anchorage Zone Details

There are three critical regions found within the anchorage zone (Breen et al 1994). The first exists directly ahead of the concentrated force where the concrete is subjected to high bearing and compressive stresses. The second extends a certain distance ahead of the anchorage device where the concrete is subjected to lateral tensile stresses. These tensile stresses are usually referred to as “bursting stresses” and are due to the deviation of the compressive stresses parallel to the force. The third is found along the edge of the member that is being loaded where the concrete is subjected to local tensile stresses. These tensile stresses are known as “spalling stresses” even though they do not cause spalling of the concrete.

There are also three principal modes of failure of the anchorage zone in precast prestressed concrete bridges (Burdet 1990). In order to best explain these failures, a general understanding of the division of the anchorage zone is needed. The anchorage zone is divided into two regions: the local zone and the general zone. The local zone is located immediately

![Stress Distribution in the Anchorage Zone](image)

Figure 1.4: Stress Distribution in the Anchorage Zone (Breen et al 1994)
ahead of and surrounding the anchorage device where the largest compressive stresses occur. The general zone is located ahead of and behind the anchorage device where the stresses become more linearly distributed.

Failure of the anchorage zone is described as occurring when equilibrium cannot be developed to resist the tendon force. The first mode of failure happens in the local zone due to a lack of concrete compressive strength or confining reinforcement. This failure usually occurs in the shape of a cone during application of the post-tensioning force. The second mode of failure happens in the general zone due to bursting stresses exceeding the capacity of the provided transverse reinforcement. It is induced by large cracks that run parallel to the post-tensioning duct from the anchorage device and usually occurs at first cracking or during loading. The third and final mode of failure occurs at the interface of the local and general zones. It is similar to the first mode of failure in that it is a compressive failure and produces a similar shape. However, it occurs at a greater distance from the anchorage device and is preceded by cracking around the anchorage device and bulging of concrete cover. All three modes of failure are brittle, explosive, and, of course, undesirable.

1.3 Anchorage Zone Reinforcement

In order to prevent the previously discussed failures, the anchorage zone must be designed to resist the high compressive stresses in the local zone and the bursting stresses in the general zone. The use of special anchorage devices with bearing plates helps to distribute the force to the concrete more evenly, thus improving the local compressive strength of the concrete. In addition to failure, bursting stresses can cause excessive cracking if not properly restrained. Passive reinforcement is used to carry the tensile stresses in the general zone after cracking has occurred. If enough reinforcement is placed in the area of cracking, then the propagation of cracks will be stopped and the tension will be transferred to the steel. Cracking is generally noticed during jacking of the post-tensioning tendons (Breen et al 1994). However, it can also occur at later stages due to creep, settlement, and temperature effects. In order to prevent this cracking, the anchorage zone is confined with secondary closed stirrups and/or spirals, as shown in Figure 1.3.
1.4 Problems with Anchorage Zones in Bridges

As previously mentioned, post-tensioning allows for smaller cross-sections in bridges. Smaller cross-sections can mean limited space for the placement of anchorage devices and confinement, shear, and general zone reinforcement. The anchorage zone quickly becomes overly congested, which in turn complicates the placement of concrete. It can also negatively affect the bond between concrete and reinforcing steel. If the anchorage zone is too congested to fit an adequate amount of reinforcing steel or if the bond between concrete and reinforcing steel is broken, failure of the anchorage zone can occur. Figure 1.5 shows a working anchorage device. Figure 1.6 shows a similar anchorage device after failure of the anchorage zone.

Figure 1.5: Working Anchorage Device
In addition, since bridges are typically located over and around bodies of water, they tend to be exposed to highly corrosive elements. A crack of sufficient size can allow these elements to permeate the structure and eventually damage the reinforcing steel. The ends of bridge girders, where anchorage zones are found, are especially susceptible to this problem. Corrosion in this area can eventually disrupt the bond between post-tensioning tendons and concrete, which slowly eliminates the strength being provided by the post-tensioning tendons. The best solution to preventing corrosion seems to be limiting the amount of cracking in the concrete. This effectively reduces corrosion possibilities and thereby protects the reinforcement and prevents extensive damage to the concrete.

1.5 Fiber Reinforced Concrete

In general, it is difficult to prevent concrete from cracking because it is a nonductile and brittle material. The formation of cracks, even micro cracks, from loading and environmental effects, has been shown to lead to deterioration and in some cases, failure. Micro cracks usually form at the interface of coarse aggregates due to thermal and moisture activity in the cement
paste even before loading occurs. Upon loading, these micro cracks propagate and group to form cracks. Short fibers evenly dispersed throughout the concrete increase the durability of concrete by preventing the micro cracks from widening and spreading into larger cracks.

For more than twenty years now, steel fiber reinforced concrete (SFRC) has been used in shotcrete, precast concrete, slabs, concrete floors, and concrete repairs. It has gained acceptance in the construction industry by continually exhibiting increased dynamic force resistance and effective crack reduction. Background review on the effective application of SFRC is presented in Chapter 2.

1.6 Objectives of the Study

In recent years, some larger post-tensioned bridge girders have shown extensive cracking mainly in the web areas, near the girder supports. The cracks have appeared soon after the transfer of prestressing forces onto the girders, and before any application of external loads. Such cracking may lead to ingress of corrosion causing agents, leading to long-term durability and ensuing strength problems for the girders.

SFRC has been effectively used in shotcrete, precast concrete, slabs, pavements, metal decks/concrete floors, seismic structures and concrete repairs. SFRC has been found to have significantly better crack resistance, ductility, modulus of rupture, shear strength, torsional strength, fatigue endurance, abrasion resistance, and energy absorption capabilities compared to plain concrete.

The objective of this study is to theoretically determine the applicability of steel fiber reinforced concrete in enhancing the general post-tensioned anchorage zones of bridge girders, together with the expected reduction of mild steel reinforcement in these zones due to SFRC application.

1.7 Scope of the Study

This study theoretically explores the feasibility of using steel fiber reinforced concrete in the general zones of post-tensioned bridge girders. This research follows a previous study performed by Yazdani et al (FDOT Report #1902-145-11) on the use of fiber reinforced concrete
in the local zones of AASHTO bridge girders. Small-scale girders were loaded to failure and the structural properties of SFRC were evaluated. It was determined that the use of SFRC enhanced the local zone behavior and provided a reduction in secondary reinforcement. Therefore, a study on the use of SFRC in the general zone is valid and useful in design.
LITERATURE REVIEW

2.1 Prestressed Concrete

Concrete is strong in compression, but weak in tension allowing the formation of flexural cracks at early load stages. A longitudinal force can be imposed to the cross-section of the concrete element, through the use of high-strength steel tendons, in order to reduce the tensile stresses that cause these cracks to develop. This longitudinal force is called a prestressing force, which is a compressive force that prestresses the sections along the span of the structural element prior to the application of dead and live loads. This concept is not new, dating back to 1872 when P.H. Jackson patented a prestressing system using a tie rod to construct beams or arches from individual blocks (Nawy 2003). The industry of prestressed concrete has come a long way through the years. Today it is used in buildings, underground structures, floating and offshore structures, power stations, and numerous bridge systems including segmental and cable-stayed bridges (Figures 2.1 and 2.2).

Figure 2.1: S.R. 600 Broadway Bridge, Daytona, FL (FIGG)
The prestressing force that is applied is dependent on the geometry and loading of a specific element and is determined with the principles of mechanics and stress-strain relationships. The prestressing force (P) can be either concentric (along the axis of the member) or eccentric (a distance away from and parallel to the axis of the member). For the concentric case, the compressive stress on the beam cross-section is uniform and equal to

\[ f = -\frac{P}{A_c} \]

where \( A_c \) is the cross-sectional area of the beam section. The minus sign denotes the state of compression. If external transverse loads are applied to the beam, causing a maximum moment (M) at midspan, the resulting stress is equal to
\[ f_t = -\frac{P}{A_c} - \frac{Mc}{I_g} \]

and

\[ f_b = -\frac{P}{A_c} + \frac{Mc}{I_g} \]

where \( f_t \) is the stress at the top fibers, \( f_b \) is the stress at the bottom fibers, \( c \) is the centroid of the cross-section, and \( I_g \) is the gross cross-sectional moment of inertia. The equation for stress at the bottom fibers shows that the presence of the compressive prestress force reduces the tensile flexural stress as intended in the design. The equation for stress at the top fibers shows that the prestress force induces compressive stresses in turn reducing the capacity for external loads to be applied. This situation is avoided by applying the prestress force eccentrically. For this case, the stresses become

\[ f_t = -\frac{P}{A_c} + \frac{Pe}{I_g} - \frac{Mc}{I_g} \]

and

\[ f_b = -\frac{P}{A_c} - \frac{Pe}{I_g} + \frac{Mc}{I_g} \]

where \( e \) is the eccentricity of the force from the centroid of the cross-section. In order to avoid high tensile stresses in the top fiber over the support, tendons are draped or harped over the length of the member (Figure 2.3). The harped profile is usually used in pretensioned beams and the draped profile in post-tensioned beams.

Figure 2.3: (a) Harped tendon. (b) Draped tendon. (Nawy 2003)
Pretensioning of the tendon occurs prior to the placement of concrete. Tendons are anchored at either end to large bulkheads. Then strands making up the tendon are stressed individually or simultaneously. The concrete is poured and once it reaches the required strength, the strands are cut at each end placing the member in a state of compression at the bottom fibers.

Post-tensioning occurs after concrete has hardened. Anchorages and ducts are cast into the concrete. There are various types of anchorages used in post-tensioning. Figure 2.4 shows a special anchorage device manufactured by DSI. Once the concrete attains the required strength, tendons are threaded through the ducts (Figure 2.5) and stressed with high-capacity hydraulic jacks (Figure 2.6). This creates the desired state of compression at the bottom fibers of the member.
Figure 2.5: Example of Post-Tensioning Ducts Inside of Bridge (FIGG)

Figure 2.6: DSI Post-Tensioning Jack (DSI)
2.2 Anchorage Zones

As discussed in the previous section, the tendons used in post-tensioned concrete must be anchored at both ends so as to create a compressive state in the concrete. Anchorage devices, which bear onto the concrete, generally over a small area, are used in this process. In order to prove effective, the tendons are usually jacked to very large stresses. This creates a large force over a small area or a highly compressive stress. To put this in perspective, it is not uncommon for an anchorage device with a cross-sectional area of a little more than one square foot to transfer a tendon force of approximately 892,000 pounds.

Since the area of the anchorage device is small in comparison to the area of concrete, a redistribution of stress occurs behind the anchorage device. According to Saint Venant’s Principle (Breen et al 1994), the compressive stresses spread to form uniform stress patterns at some distance into the concrete. The region within this distance where the redistribution occurs is identified as the anchorage zone.

The two areas of focus during design of the anchorage zone are the high compressive stresses ahead of the anchorage device and the large tensile stresses located at a distance behind the anchorage device as seen in Figure 2.7. Due to the difference in magnitude and location of these stresses, it has been proposed that the anchorage zone be divided into two regions: the local zone and the general zone, as shown in Figure 2.8.

The stress contours in Figures 2.7 and 2.8 are based on a single anchorage situation. However, most practical applications of post-tensioning involve the use of multiple tendons to stress the member. In 1990, Burdet investigated the stress contours of two anchorages with variable spacing. He determined that in cases where the anchorages were close (up to one plate size between the anchorages), the stress distribution was similar to that of a single anchorage. As the spacing between the plates increased (beyond one plate size), two clearly separate areas of stresses appeared and spalling stresses developed between the two anchorages. Figure 2.9 shows these results.
Figure 2.7: Principle Stress Contours in Anchorage Zone (Breen et al. 1994)

Figure 2.8: Local and General Anchorage Zone (Yazdani et al. 2002)
Figure 2.9: Stress Contours in the Anchorage Zone for Two Anchorages with Variable Spacing (Burdet 1990)
2.2 Local Zone

The local zone may be defined as the volume of concrete immediately ahead of and surrounding the anchorage device. It encompasses the region in which concrete compressive stresses exceed allowable values for unconfined concrete. The purpose of the local zone is to resist these high local compressive stresses and transfer them to the general zone. These compressive stresses begin at the interface between the anchorage device and the concrete of the local zone. Therefore, the complex geometry of the anchorage device strongly influences the behavior and geometry of the local zone.

In a study of the behavior and design of local anchorage zones, Roberts (1990) recommended a test procedure for the accepted use of special anchorage devices in post-tensioned concrete members. The test block should be rectangular. The dimensions perpendicular to the tendon in each direction should be the smaller of the edge distance or the minimum spacing as long as there is appropriate cover for all reinforcing steel. The length along the axis of the tendon should be two times the longer of the cross-sectional dimensions. The reinforcing should be as specified by the supplier for the particular device. The test block should be loaded cyclically, from 10% ultimate post-tensioning force to 80% ultimate post-tensioning force, for no less than 10 cycles or until crack widths stabilize. Or the test block could be loaded with 90% ultimate post-tensioning force and held constant for 48 hours, then loaded to failure, otherwise known as sustained loading. Crack widths, patterns, and progression should be recorded at initial loads of 40%, 60%, and 80% ultimate post-tensioning force. For cyclic loading, measurements should be recorded at every peak loading. For sustained loading, measurements should be recorded every 6 hours. During loading to failure, final measurements should be recorded at 90% ultimate post-tensioning force and the ultimate load should also be recorded. However, the test block does not need to be taken to failure if the system has achieved over 100% ultimate post-tensioning force.

The criterion for acceptance of the previously discussed test procedure was also recommended. The strength of the anchorage zone must equal or exceed 100% ultimate post-tensioning force. During the initial loading, at 40% ultimate post-tensioning force, there should be no cracks greater than 0.002 inches. After completion of the cyclic or
sustained loading, with 80% ultimate post-tensioning force still in place, there should be no cracks greater than 0.008 inches. At 90% ultimate post-tensioning force, there should be no cracks greater than 0.016 inches. The engineer may require stricter crack width limitations on projects in highly aggressive environments where additional protection against corrosion of reinforcing steel is necessary. A total of three tests, either cyclic or sustained, should be performed and meet the above specified criteria. If one of the three tests fails to meet the criteria, a series of two identical specimens may be tested with both meeting the above specified criteria. These test procedures were later adopted by AASHTO Standard Specifications for Highway Bridges (1998).

AASHTO Standard Specifications for Highway Bridges (Sixteenth Edition, 1996) imposes detailed definitions to determine local zone dimensions, which are more easily understood as depicted in Figure 2.10. AASHTO Section 9.21.7.1.3 specifies that the length of the local zone along the tendon axis shall be taken as the greater of:

1. The maximum width of the local zone;
2. The length of the anchorage device confining reinforcement; or
3. For anchorage devices with multiple-bearing surfaces, the distance from the loaded concrete surface to the bottom of each bearing surface plus the maximum dimension of that bearing surface.

In no case shall the length of the local zone be taken as greater than 1½ times the width of the local zone.

The first mode of failure in anchorage zones occurs in the local zone due to a lack of concrete compressive strength or confining reinforcement. Figure 2.11 shows the suppression of an anchor during jacking and Figure 2.12 shows the corresponding compressive failure. Lateral confinement can be added to the local zone in order to enhance the compressive strength of concrete, thereby reducing the likelihood of failure in the local zone. A combination of spirals and orthogonal ties are the most commonly provided lateral steel reinforcement. However if the surrounding concrete area is significantly larger than the anchorage device bearing area, confinement steel may not be needed provided that general zone requirements are satisfied.
Figure 2.10: AASHTO Local Zone Definitions (Breen et al 1994)
2.3 General Zone

The general zone is defined as the region of the structure ahead of and behind the anchorage device where the linear stress distribution or ordinary beam theory is disturbed by the introduction of the concentrated tendon force. It overlaps and encompasses the local zone. The purpose of the general zone is to transfer the flow of stresses and forces from the concentrated tendon force to the structure. Some examples of general zone dimensions are shown in Figure 2.13.
Figure 2.13: Examples of General Zone Dimensions (Wollmann 2000)
The second mode of anchorage zone failure occurs in the general zone due to tensile bursting and spalling stresses in the concrete. The bursting stresses act perpendicular to the tendon path and are caused by the lateral distribution of the tendon force from the anchorage device across the general zone. The spalling stresses act parallel to the surface of the anchorage device due to the incompatibility of displacements in the concrete. When these tensile stresses exceed the modulus of rupture of concrete, bursting or spalling cracks occur. Reinforcing steel must be placed in the general zone to carry the tensile forces propagated through the cracks.

Even though the state of stress within the general zone is complicated, fairly simple and conservative design approaches are adequate. AASHTO Section 9.21.3.1 (1996) allows the use of the following methods in the design of general zones:

1. Equilibrium based plasticity models (strut-and-tie models);
2. Elastic stress analysis (finite element analysis or equivalent); or
3. Approximate methods for determining the compression and tension forces, where applicable.

The strut-and-tie method is based on a plasticity approach that approximates the flow of forces in the anchorage zone using a truss structure following general equilibrium principles. The truss consists of straight compression struts and straight tension ties that are interconnected by nodes. The compressive forces are carried by the concrete compressive struts and the tensile forces are carried by reinforcement steel, either mild or prestressed. The total area of steel required in the general zone is determined by the yield strength of the reinforcement being used. Figure 2.14 shows a schematic of compression strut-and-tie force paths while Figure 2.15 shows typical strut-and-tie models for common situations in anchorage zones. Strut-and-tie models typically give conservative results (Nawy 2003).
Figure 2.14: Schematic of Compression Strut-and-Tie Force Paths (Nawy 2003)
Figure 2.15: Typical Strut-and-Tie Models for Anchorage Zones (Nawy 2003)
Elastic stress analysis involves computing the detailed state of stresses using the finite element method. As mentioned previously, the general zone is subjected to extensive tensile stresses normal to the tendon axis. A linear-elastic stress analysis can predict cracking locations and give a reasonably reliable approximate estimate of the flow of stresses after cracking (Nawy 2003). A nonlinear finite element analysis predicts the post-cracking response more accurately. However, the process is more time consuming and labor intensive. In general, most design engineers expect faster answers and possess the experience that is necessary to make assumptions that would validate a linear elastic analysis. From the analysis, the maximum moment \( M_{\text{max}} \) is calculated which helps determine the potential position of the horizontal bursting crack. The maximum moment is resisted by the tensile force \( T \) of the reinforcement and the compressive force \( C \) provided by the concrete. The horizontal shear force \( V \) at the crack split surface is resisted by aggregate interlock forces. So the tensile force is calculated by,

\[
T = \frac{M_{\text{max}}}{(h-x)}
\]

where \( h \) is the distance from the face of the concrete to the centroid of the compressive force, and \( x \) is the distance from the face of the concrete to the centroid of the tensile force. Once the tensile force is known, the total required area of the steel reinforcement is calculated by,

\[
A_t = \frac{T}{f_s}
\]

where \( f_s \) is the stress in the steel and should not exceed 20,000 psi for the purpose of crack width control.

Guyon’s Method provides simplified equations that can also be used to compute the tensile bursting force \( T_{\text{burst}} \) and its centroid distance \( d_{\text{burst}} \) from the anchorage. However, the following equations apply only to rectangular members having no discontinuities along the span length.

\[
T_{\text{burst}} = 0.25 \times \text{Sum}(P_{su}) \times (1 - (a/h))
\]

\[
d_{\text{burst}} = 0.5 \times (h - 2e)
\]

where \( \text{Sum}(P_{su}) \) is the sum of the total factored prestress loads for the stressing arrangement considered, \( a \) is the plate width of the anchorage device or single group of closely spaced devices in the direction considered, \( e \) is the eccentricity (always positive) of the anchorage device or group of closely spaced devices with respect to the centroid of
the cross-section, and \( h \) is the depth of the cross-section in the direction considered. ACI 318 requires that the design of confining reinforcement in the end anchorage block of post-tensioned members be based on the factored prestressing force \( P_{su} \), as follows) for both the general and local zone.

\[
P_{su} = 1.2 \times A_{ps} \times (0.8f_{pu})
\]

where 1.2 is the load factor applied, \( A_{ps} \) is the area of prestressing steel, and \( 0.8f_{pu} \) is the end anchorage stress level for low-relaxation strands.

In situations containing more complex anchorage zones, force path methods should be used for design. The strut-and-tie model has previously been discussed. Another method is Morsch’s Model, which is an early application of the strut-and-tie modeling concept (Wollmann, 2000). Morsch approximated the flow of compressive stresses using two straight compressive forces and equilibrium condition to solve the magnitude of the transverse tensile force. Another force path approach to general zone design is the deep beam analogy, which is based on the similarity of an anchorage zone to a deep beam (Wollmann, 2000). This analogy views tendon forces as reaction forces and the stresses at the end of the general anchorage zone as the applied load.

### 2.4 Fiber Reinforced Concrete

Fiber reinforced concrete (FRC) consists mainly of hydraulic cements, fine and coarse aggregates, and reinforcing fibers. The discontinuous discrete reinforcing fibers are made from different materials in varying shapes and sizes. Unreinforced concrete has low tensile strength and cracks easily under tensile stress and impact loads. To overcome this weakness, post-tensioning tendons and steel reinforcing bars are placed in the concrete. Fibers may be cast into and scattered throughout the concrete matrix. They bridge the cracks and provide improved serviceability of the structure. The amount of improvement depends on many factors, such as fiber type, modulus, aspect ratio, strength, surface bonding characteristics, content and orientation (Yazdani et al 2002).

Currently the most commonly used fiber types are steel, synthetic, glass, and natural fibers. The different types are used to improve performance in different situations. Steel fibers can increase load carrying capacity when cast in concrete by
transferring the load from the concrete to the fiber. This happens by shear deformation at
the fiber-matrix interface as a result of differing physical properties between the fibers
and the concrete (Beaudoin 1990). Steel fibers should be used in a supplementary role to
inhibit cracking, to improve resistance to impact or dynamic loading, or to resist material
disintegration (ACI 544.3R 1993). Synthetic fibers should be applied to non-structural
and non-primary load bearing applications (ACI 544.1R 1996). Glass fibers should be
used in non-structural architectural cladding (ACI 544.1R 1996) and are usually
combined with mortar instead of concrete. For the structural purposes of this research,
only steel fibers were used and studied.

2.5 Steel Fiber Reinforced Concrete (SFRC)

SFRC consists of a mixture of hydraulic cements with fine and coarse aggregates
and steel fibers. The steel fibers (Figure 2.16) used in SFRC are short, discrete lengths of
steel having an aspect ratio (ratio of length to diameter) from about 20 to100, with any of
several cross-sections, and that are sufficiently small enough to be randomly dispersed in
an unhardened concrete mixture using usual mixing procedures (ACI 544.1R 1996).
Figure 2.17 shows some of the various geometries of the steel fibers used in SFRC. The
influence that fibers have on the mechanical properties of concrete depends on the type of
the fiber, the aspect ratio of the fiber, the amount of fiber, the strength of the concrete-fiber
matrix, the size, shape and method of preparation of the specimen being analyzed,
and the size of the aggregate used in the concrete mix.
Figure 2.16: Various Types of Steel Fibers (Yazdani et al 2002)

Figure 2.17: Various Geometries of Steel Fibers (ACI 544.1R 1996)
The presence of fiber has a direct effect on the mechanical properties of concrete including compression, direct tension, shear, and flexural strength. The fiber shares induced stress with the concrete until the concrete cracks. Then, eventually the fiber carries all of the stress. There are many advantages that suggest the reduction or replacement of conventional reinforcing steel with steel fibers, such as the following:

- Enhanced flexural strength, shear strength, ductility and toughness.
- Impact and fracture resistance.
- Internal stresses are more evenly distributed throughout the structure because multi-directional reinforcement is provided.
- Crack widths are minimal, if cracks are found at all, because fibers bridge the cracks.
- Decreased chance of corrosion due to crack control and the fact that fibers do not provide a continuous path for corrosive currents to flow through.
- Savings in labor and time costs of a project because FRC placement is less demanding than conventional rebar placement.

In 2003 Haroon completed a study of the application of fiber reinforced concrete in the end zones of precast prestressed bridge girders. Tests were performed to determine the compressive strength, tensile strength, and flexural toughness characteristics of FRC using testing procedures as specified by ASTM. Three types of fibers were used: two steel fibers (XOREX and ZP305) and a synthetic fiber (Harbourite H-330). In order to determine the behavior of fiber at varying amounts, 0.5%, 0.75% and 1.0% volumes were selected based on manufacturer recommendations. Control specimens were also tested for comparison purposes.

Thirty cylinders were prepared for the compressive strength test and compared against control specimens. They were tested following ASTM C-39 (1996) specifications. Figure 2.18a shows a failed specimen after the compressive strength test. The ZP305 steel fibers provided an increased compressive strength in the order of 14% to 18% with 0.5% to 1.0% volume used. The XOREX steel fibers provided an increased compressive strength in the order of 3% to 15% with 0.5% to 1.0% volume used. The synthetic fibers produced a decrease in compressive strength, which could be due in part
Concrete failure due to direct tension occurs when micro cracks propagate into larger cracks eventually causing the structure to be in an unstable condition. It is anticipated that steel fibers can prevent the formation of micro cracks, thereby significantly increasing the tensile strength of concrete. Haroon prepared thirty cylinders for testing split tensile strength. They were tested following ASTM C-496 specifications (1996). Figure 2.18b shows a failed specimen after the split tensile strength test. The ZP305 steel fibers provided an increased tensile strength in the order of 5% to 37% with 0.5% to 1.0% volume used. The XOREX steel fibers provided an increased tensile strength in the order of 4% to 9% with 0.5% to 1.0% volume used. The synthetic fibers produced a decrease in tensile strength. Table 2.2 shows the results as they appear in the original study.

Table 2.1: Compressive Strength Results of Haroon Study (2003)

<table>
<thead>
<tr>
<th>Type of fiber</th>
<th>Fiber volume (%)</th>
<th>Compressive strength results</th>
<th>Difference with control (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Test value, psi (MPa)</td>
<td></td>
</tr>
<tr>
<td>Control specimen</td>
<td>0.00</td>
<td>6350 (43.75)</td>
<td>0</td>
</tr>
<tr>
<td>with no fiber</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>XOREX</td>
<td>0.50</td>
<td>6600 (45.47)</td>
<td>+3.94</td>
</tr>
<tr>
<td></td>
<td>0.75</td>
<td>6900 (47.54)</td>
<td>+8.66</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>7350 (50.64)</td>
<td>+15.75</td>
</tr>
<tr>
<td>ZP305</td>
<td>0.50</td>
<td>7300 (50.30)</td>
<td>+14.96</td>
</tr>
<tr>
<td></td>
<td>0.75</td>
<td>7400 (50.99)</td>
<td>+16.54</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>7510 (51.74)</td>
<td>+18.27</td>
</tr>
<tr>
<td>Harbourite H-330</td>
<td>0.50</td>
<td>5100 (35.14)</td>
<td>-19.69</td>
</tr>
<tr>
<td></td>
<td>0.75</td>
<td>3410 (23.5)</td>
<td>-46.3</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>3800 (26.18)</td>
<td>-40.16</td>
</tr>
</tbody>
</table>
Table 2.2: Tensile Strength Results from Haroon Study (2003)

<table>
<thead>
<tr>
<th>Type of fiber</th>
<th>Fiber volume (%)</th>
<th>Tensile strength results</th>
<th>Difference with control (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Test value, psi (MPa)</td>
<td></td>
</tr>
<tr>
<td>Control specimen</td>
<td>0.00</td>
<td>630 (4.34)</td>
<td>---</td>
</tr>
<tr>
<td>with no fiber</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>XOREX</td>
<td>0.50</td>
<td>660 (4.55)</td>
<td>+4.76</td>
</tr>
<tr>
<td></td>
<td>0.75</td>
<td>670 (4.63)</td>
<td>+6.68</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>690 (4.75)</td>
<td>+9.52</td>
</tr>
<tr>
<td>ZP305</td>
<td>0.50</td>
<td>660 (4.58)</td>
<td>+5.56</td>
</tr>
<tr>
<td></td>
<td>0.75</td>
<td>730 (5.03)</td>
<td>+15.87</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>870 (5.96)</td>
<td>+37.3</td>
</tr>
<tr>
<td>Harbourite H-330</td>
<td>0.50</td>
<td>580 (4.03)</td>
<td>-7.14</td>
</tr>
<tr>
<td></td>
<td>0.75</td>
<td>400 (2.76)</td>
<td>-36.5</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>530 (3.62)</td>
<td>-16.67</td>
</tr>
</tbody>
</table>

(a) Compressive Strength  (b) Split Tensile Strength

Figure 2.18: Failed Specimens (Haroon 2003)
Twenty beams were prepared to test flexural toughness using third-point loading. Results yielded that specimens containing fibers attained higher first crack strengths than specimens without fiber (Figure 2.19). It was also observed that the concrete beams without fiber failed suddenly when the first flexural crack formed. However, the concrete beams containing fiber did not (Figure 2.20).

Haroon also concluded that fresh fiber reinforced concrete is stiffer and less workable than non-fibrous concrete. However, he used the specifications provided in ASTM C-143 to obtain slump values for the fiber reinforced concrete and all slumps measured were found to be within the provision of ACI 544.3R. In order to improve the workability of the FRC, he recommended that it be vibrated for a longer amount of time than non-fibrous concrete. In addition, he noted that continuing the vibration of the concrete while trowelling the top surface helped to prevent fibers from protruding out of the finished surface.

![Figure 2.19: Influence of Fiber Type and Volume Fractions on First Crack Strengths (Haroon 2003)](image-url)
In addition, Haroon conducted research to determine the effect of using SFRC in the local zone of precast prestressed bridge girders. Local zone specimens were tested based on the AASHTO Special Anchorage Device Acceptance Test requirements, as previously discussed in this chapter. The test block studied was 12.5” by 12.5” in cross-section and 25” long, with a VSL EC5-7 anchorage and local zone reinforcement as specified by the manufacturer and seen below in Figure 2.21.

The spiral and skin reinforcement was varied as shown in Table 2.3. Each reinforcement type was tested with 1% XOREX and ZP305 steel fibers, then 0.75% ZP305 steel fibers. The H-330 polypropylene fiber was not used in this part of the testing due to the results obtained from the property testing. Haroon recommends the reduction of local zone reinforcement with the amount of steel fiber in Table 2.4.
Figure 2.21: Configuration of AASHTO Test Block (Haroon 2003)
Table 2.3: Spiral and Skin Reinforcement Combinations (Haroon 2003)

<table>
<thead>
<tr>
<th>ID</th>
<th>Spiral reinforcement</th>
<th>Skin reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement 1</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>Reinforcement 2</td>
<td>None</td>
<td>3</td>
</tr>
<tr>
<td>Reinforcement 3</td>
<td>None</td>
<td>5</td>
</tr>
<tr>
<td>Reinforcement 4</td>
<td>3-turn</td>
<td>None</td>
</tr>
<tr>
<td>Reinforcement 5</td>
<td>3-turn</td>
<td>3</td>
</tr>
<tr>
<td>Reinforcement 6</td>
<td>3-turn</td>
<td>5</td>
</tr>
<tr>
<td>Reinforcement 7</td>
<td>6-turn</td>
<td>None</td>
</tr>
<tr>
<td>Reinforcement 8</td>
<td>6-turn</td>
<td>3</td>
</tr>
<tr>
<td>Reinforcement 9</td>
<td>6-turn</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 2.4: Recommended SFRC Configurations (Haroon 2003)

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Reduction or elimination of secondary reinforcement (%)</th>
<th>Fiber configuration</th>
<th>Minimum compressive strength at post-tensioning, psi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Spiral</td>
<td>Skin</td>
<td>Type</td>
</tr>
<tr>
<td>1</td>
<td>100</td>
<td>100</td>
<td>ZP305</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ZP305</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>XOREX</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>100</td>
<td>ZP305</td>
</tr>
<tr>
<td>3</td>
<td>100</td>
<td>40</td>
<td>ZP305</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>0</td>
<td>ZP305</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>XOREX</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>40</td>
<td>ZP305</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>XOREX</td>
</tr>
</tbody>
</table>

Research has also been done to show the effect of steel fibers on shear resistance of concrete. There are many advantages to using fibers to replace vertical stirrups, such as (1) when fibers are distributed, they actually achieve a much closer spacing than what can be accomplished with the smallest of reinforcing bars; (2) decreased crack sizes; (3) the tensile strength of the concrete is increased; and (4) the fibers resist pullout and
bridge cracks thereby increasing shear-friction strength. It has been found that for 1% volume of fibers, an increase of up to 170% in ultimate shear strengths is possible (Narayanan and Darwish 1987).

Steel fibers have also been used to improve the flexural strength of concrete. This improvement increases with increased fiber content. However, due to the presence of coarse aggregate along with mixing and placing considerations, fiber volume in concrete is limited to 1.5% to 2.0%. At lower fiber contents, the increase in flexural strength is not as noticeable. In a study by Swamy and Al-Ta’an (1981), the following conclusions were drawn:

- Fibers are effective in resisting deformation at all stages of loading
- Fibers inhibit crack growth and crack widening at all stages of loading.
- The number of cracks at the working load stage is about half of those fully developed before failure.
- Increasing fiber content results in a consistent increase in ductility and energy-absorption capacity.
- The influence of fibers in reducing deformation and increasing flexural stiffness is evident even at the failure stage.

In 1987, Gopalaratnam developed a modified instrumented Charpy test that enabled dynamic flexural tests in brittle matrix composites at different loading rates. This was done to measure the affect that fiber has in impact and fracture resistance. Impact tests were conducted on four different mixes with five different impact velocities. Overall it was determined, that the incorporation of ductile fibers in a brittle matrix greatly enhances its energy absorption capacity, improves its ductility and resistance to crack growth, and marginally enhances its tensile strength.

Experience has proven that if SFRC has a 28-day compressive strength greater than or equal to 3000 psi, is well compacted, and complies with ACI 318 (2002) recommendations for water-cement ratio, then corrosion of fibers will be limited to the surface skin of the concrete (Yazdani et al 2002). When the surface fibers have corroded, it appears to be limited to that region due to the fact that the fibers are short and rarely touch. Therefore, there is not a continuous conductive path provided that would allow corrosive currents to pass between different areas of the concrete.
2.6 Application of Steel Fiber Reinforced Concrete

Currently, SFRC is being used in many structural applications, such as: building slabs, airport runways, bridge decks, pavements, roadway overlays, tunnel supports, industrial floors, piles, retaining walls, and hydraulic structures. It is also being utilized in various marine structures. The idea of using SFRC in more areas of engineering practice shows great potential. There are vast amounts of research being done to prove the structural integrity and effectiveness of SFRC. Before long the use of it will be common practice.
3.1 Introduction

This chapter describes the selection process for the theoretical test specimens. The details of the chosen specimen are presented here. Fiber types and amounts were selected based on the results from previous laboratory testing done by Haroon (2003) using fiber reinforced concrete in the local zone.

3.2 Specimen Dimensions

Several existing bridges were studied in order to determine a common bridge cross section for modeling. The main bridges considered include the Santa Rosa Bay Bridge and the Choctawhatchee Bay Bridge (Mid-Bay), designed by FIGG Engineering Group. These bridges were chosen for the study because both are concrete segmental box girder bridges located in the state of Florida and owned by the Florida Department of Transportation. Further comparison revealed that the cross sections were also fairly similar (Table 3.1). Due to geometry and modeling complexity, the Choctawhatchee Bay Bridge was chosen to be a representative model and a simplified drawing can be seen in Figure 3.1.

<table>
<thead>
<tr>
<th>Project</th>
<th>Width</th>
<th>Height</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Santa Rosa Bay Bridge</td>
<td>7’-10”</td>
<td>8’-0”</td>
<td>9’-5”</td>
</tr>
<tr>
<td>Choctawhatchee Bay Bridge</td>
<td>8’-1”</td>
<td>9’-0”</td>
<td>9’-5”</td>
</tr>
</tbody>
</table>
3.3 Anchorage Selection

The post-tensioning steel required for the bridge was provided in the contract plans of the Choctawhatchee Bay Bridge. The segment that was chosen detailed two 19-0.6” diameter longitudinal tendons (Figure 3.1) on each face. VSL type EC 5-19 anchorages were used in this bridge and modeled in this study (Figure 3.2). The ultimate post-tensioning force ($P_u$) for this anchorage is determined by the following equation:

$$P_u = A_s * n * f_{pu}$$

$$P_u = 0.153 * 19 * 270 = 785 \text{ kips}$$

where $A_s$ is the area of each strand, $n$ is the number of strands, and $f_{pu}$ is the ultimate strength of the tendon. An ultimate post-tensioning force of 785 kips was applied to the modeled specimen.

The duct that is provided with the VSL type EC 5-19 anchorage is 3.75” in diameter and was modeled in the specimen. The local zone reinforcement is specified by VSL Corporation to accompany the chosen anchorage and includes a #5 spiral around the anchorage. The spiral has an outside diameter of 15” and 8 turns with a 2.25” pitch.
3.4 Mild Steel Reinforcing Selection

The mild reinforcing steel required in the general zone for the chosen segment was provided in the contract plans of the Choctawhatchee Bay Bridge and is very complex. The detailed contract drawing can be seen in Figure 3.3.

3.5 Fiber Reinforcing Selection

Steel fiber reinforced concrete (SFRC) was discussed in Chapter 2. There are many different types of steel fibers on the market. There were three (3) types of fibers considered for this study. They are Dramix ZP305, Helix, and Novomesh 850. It was decided to use the Dramix ZP305 for the finite element model based upon consideration of the results from earlier tests conducted by Yazdani, et al (2002) that were discussed in detail in Chapter 2. Table 3.1 provides general information about the Dramix ZP305 fibers that were selected for use in this study. A photograph of this steel fiber can be seen in Figure 3.4.
Figure 3.3: Choctawhatchee Main Span Pier Segment Reinforcing (FIGG)
Figure 3.4: Dramix ZP305 Samples (Yazdani et al 2002)

Table 3.2: Information about the Dramix ZP305 Fiber

<table>
<thead>
<tr>
<th>Type</th>
<th>Manufacturing Company</th>
<th>Fiber Length, in.</th>
<th>Avg. Diameter, in.</th>
<th>Aspect Ratio</th>
<th>Appearance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>Bekaert Corporation</td>
<td>1.2</td>
<td>0.022</td>
<td>55</td>
<td>Cold drawn wire fiber, with hooked ends, and glued in bundles</td>
</tr>
</tbody>
</table>
4.1 Introduction to the Finite Element Method

The finite element method (FEM) is a powerful technique used to numerically solve engineering and mathematical physics problems. An example of some problems would be structural analysis, heat transfer, fluid flow, and mass transport. This chapter mainly discusses structural applications of the FEM with specific emphasis on concrete structures. A main span pier segment of the Choctawhatchee Bay Bridge will be used in this study to analyze the effect of post-tensioning stresses on fiber reinforced concrete at the anchorage zone of a segmental bridge girder as previously described in Chapter 3.

In order to apply the FEM, a structure has to have all geometries (keypoints, lines, areas, and volumes) input into the computer program. It is then divided into an equivalent system of smaller bodies or units (finite elements) that are interconnected at points common to two or more elements (nodes) (Logan 2000). These elements are assigned with properties that reflect the materials pertaining to the structure under investigation. Load can then be applied and the problem analyzed. The analysis returns theoretical results at each node based on the physical description of the actual situation, i.e. stress plots, deformation graphics, etc. To provide a visual concept of how structures are subdivided and results displayed, see Figure 4.1.

![Figure 4.1: Example of FEM](image-url)
The complexity of concrete structures is basically in modeling the reinforcement of the concrete and in modeling the process of cracking once the principle stresses in the structure reach the upper limit of the stress strain relationships of the material. Currently, most of the commercially available FEM programs are capable of handling static and dynamic analyses of concrete structures. For the purpose of this study, it was decided to use ANSYS 10.0 program to analyze the behavior of fiber reinforced concrete at the anchor zone. ANSYS was chosen because it provides a wide range of elements and constitutive models for different materials including concrete.

4.2 Development of the Finite Element Model

As previously mentioned, the first step in developing the 3-D finite element model was to input the geometry of the segment, including the post-tensioning duct and anchorage, into ANSYS. First, keypoints were defined. Second, lines were drawn between keypoints to form the boundaries of the segment and also to break the segment into pieces for meshing purposes. Then, areas were formed within the lines. Lastly, volumes were created based on the inputted areas. Several ways of breaking up the segment were explored and the optimal segment was taken as seen in Figure 4.2.

Figure 4.2: ANSYS Model of Volumes
The optimal designation has to do with the meshing capabilities of the program. A few rules to follow are given. The mesh should consist of quad (brick) elements; therefore all volumes must be either four or six sided. Due to the complex geometry of the segment, there were volumes that had to be five sided. In this situation two sides were chosen to act as one, which the program calls concatenation. This means that the mesh will flow from one side and disperse to the two concatenated sides. The mesh should also be fairly consistent. The density of the mesh should be similar throughout the segment to prevent clusters of nodes from forming in places with tight geometry. These clusters can cause stress concentrations in the analysis, raising questions on the validity of the results. The current model has the 463 keypoints, 1,155 lines, 914 areas, 230 volumes, and 6,082 solid and shell elements.

4.3 Elements & Material Properties

Once the geometry was input, the necessary properties of the segment had to be input in ANSYS. The necessary properties involved choosing the element that would be used to mesh the segment along with defining the material properties of the segment. The segment consists of concrete (with reinforcing steel), steel anchorages, and steel ducts. A complete list of the required material properties is provided in Table 4.1

The concrete portion of the segment was meshed using the SOLID65-3D reinforced concrete element from ANSYS. This element is used for the 3-D modeling of solids with or without reinforcing bars. It is capable of simulating tension cracks, compressive crushing, plastic deformation, and creep for the concrete. It also simulates tension and compression in the reinforcing (ANSYS 10.0, 2004). In concrete applications, for example, the solid capability of the element may be used to model the concrete while the rebar capability is available for modeling reinforcement behavior. Other cases for which the element is also applicable would be reinforced composites (such as fibers). The element is defined by eight nodes each having three degrees of freedom (Figure 4.3): translations in the nodal x, y, and z directions. Up to three different rebar specifications may be defined.
The constitutive model for concrete material is based on Willam and Warnke (1975) yield criterion. The criterion for failure of concrete due to a multiaxial stress state can be expressed in the form:

\[(F/f_c) - S \geq 0\]

where \(F\) is a function of the principal stress state, \(S\) is the failure surface (to be discussed) expressed in terms of principal stresses and five input parameters \(f_t, f_c, f_{cb}, f_1\) and \(f_2\), and \(f_c\) is the uniaxial crushing strength. If the above equation is satisfied, the material will crack or crush.

A total of five input strength parameters (each of which can be temperature dependent) are needed to define the failure surface as well as an ambient hydrostatic stress state. These are \(f_t\) (the ultimate uniaxial tensile strength), \(f_c\) (the ultimate uniaxial compressive strength), \(f_{cb}\) (the ultimate biaxial compressive strength), \(f_1\) (the ultimate compressive strength for a state of biaxial compression superimposed on hydrostatic stress state), and \(f_2\) (the ultimate compressive strength for a state of uniaxial compression superimposed on hydrostatic stress state). The first and the second parameters are input into the material properties for concrete. These values are usually obtained from laboratory testing or from the assumed design values for the structure. The stiffness parameter is also required in the material properties section.

Since the segment was modeled using nonlinear plastic behavior, numerous material properties had to be defined. The required concrete properties include density, modulus of elasticity, Poisson’s ratio, ultimate tensile strength, and ultimate compressive strength. The weight density was assumed as 150 pcf since this is the density of
reinforced/prestressed normal weight concrete. Poisson’s ratio is the ratio of the transverse to the longitudinal strains under axial stress within the elastic range and varies between 0.15 and 0.20 for both normal and lightweight concrete (Hassoun, 2002). The modulus of elasticity, \( E_c \), is calculated by the following equation:

\[
E_c = 33 \times w^{1.5} \times (f'_{c})^{1/2}
\]

where \( w \) is the density of the concrete and \( f'_{c} \) is the compressive strength of the concrete. The compressive strength of the concrete was chosen based on research done by Haroon (2002). As previously discussed in detail in Chapter 2, the compressive and tensile strengths increase with the addition of steel fiber. Therefore, that was taken into account in this study as reflected in Table 4.2. The tensile strength of concrete ranges from 7% to 11% of its compressive strength, with an average of 10% (Hassoun, 2002).

To model reinforcing in concrete, one of two methods is usually followed. In the first method the reinforcing is simulated as spar elements with geometric properties similar to the original reinforcing (Figure 4.4a). These elements can directly be generated from the nodes in the model. This method of discretization is useful in simple concrete models. The second idealization of steel reinforcing is the smeared concrete element method. In this case the concrete and the reinforcing is discretized into elements with the same geometrical boundaries and the effects of reinforcing are averaged within the pertaining element (Figure 4.4b). Cracks can also be idealized into either the discrete type or the smeared type.

Figure 4.4: Discrete vs. Smeared Element for Concrete Reinforcing
Since the SOLID65-3D concrete element simulates tension and compression in reinforcing bars, the volumetric ratio of reinforcing steel to concrete along with the direction of the steel had to be provided for each volume in order for the program to account for the reinforcing steel. The required steel properties include density, modulus of elasticity, and Poisson’s ratio and are shown in Table 4.1. There are many different grades of steel, but the modulus of elasticity is constant for all types (Hassoun, 2002). The weight density of steel and Poisson’s ratio were obtained from Gere (2001).

The VSL anchorages used in the segment were simplified for the finite element model due to their complex geometry. They were modeled as plates based on the fact that the curvature is minimal with respect to the size of the entire segment and therefore will not alter results. The plates provided a bearing surface by which to apply the post-tensioning force of the tendons. They were defined as one-inch thick steel plates because that is the thickness of the bearing plate assembly of the actual VSL anchorage. They were meshed using the SHELL63 element from ANSYS. This element has both bending and membrane capabilities and permits in-plane and normal load application (ANSYS 10.0, 2004). The steel plates were modeled assuming linear elastic behavior, which required density, modulus of elasticity, and Poisson’s ratio to be defined. These steel properties are the same as those listed for reinforcing steel and are shown in Table 4.1.

The ducts were also modeled using the SHELL63 element from ANSYS. Following manufacturer specifications, they are one-eighth of an inch thick, steel ducts. The material properties defined were density, modulus of elasticity, and Poisson’s ratio. These steel properties are the same as those listed for reinforcing steel and are shown in Table 4.1.
Table 4.1: Material Properties for ANSYS Finite Element Model

<table>
<thead>
<tr>
<th>Material Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Concrete Properties</strong></td>
<td></td>
</tr>
<tr>
<td>Density</td>
<td>150 pcf</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>4,792,817 psi</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.20</td>
</tr>
<tr>
<td><strong>Steel Properties</strong></td>
<td></td>
</tr>
<tr>
<td>Density</td>
<td>490 pcf</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>29,000,000 psi</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.30</td>
</tr>
</tbody>
</table>

4.4 Boundary Conditions & Loading

Once volumes were meshed with the corresponding element and material properties, the next step was defining boundary conditions and applying load. The boundary conditions describe how the segment is to be supported. At the bridge location, the segment is supported by elastomeric bearings on a concrete pier. Therefore, the boundary condition was defined by preventing the segment from vertical displacement across the bottom slab.

The other boundary condition that applies to this segment is symmetry. The segment is symmetric about the centerline in the vertical direction. The use of symmetry allows us to consider a reduced problem. Modeling half of the actual pier segment saves space in ANSYS, enabling greater mesh refinement and therefore more accurate results. At the plane of symmetry, the displacement in the direction perpendicular to the plane must be equal to zero (Logan 2000). This was achieved through placing rollers perpendicular to the centerline of the segment and preventing horizontal displacement.

Finally, the post-tensioning force was applied to the plates on the segment. The force applied was determined by calculating the ultimate post-tensioning force arrived at by multiplying the number of strands (19), the individual strand area (0.153 sq. inches) and the ultimate tensile strength of the strands (270,000 psi). This yields 785,000 pounds of force. Based on design specifications, the allowable stressing force is 80% of the
ultimate post-tensioning force, which is 628,000 pounds of force. Since there are two plates, a total of 1,256,000 pounds was introduced to the segment on each face. The final model used for analysis complete with boundary conditions is shown in Figure 4.5.

Figure 4.5: Final ANSYS Model

4.5 Segments Modeled for FEM Analysis

Nine segments were initially modeled for the FEM analysis. Table 4.2 details these segments. As previously mentioned, the compressive and tensile strengths increase with the addition of steel fiber. Therefore, that was taken into account in this study as reflected in Table 4.2. The segments had varying amounts of steel fibers and the design required mild steel reinforcement. The designation of 100% mild reinforcement is based on the design-required amount and indicates that 100% of what was required by design was modeled. The purposes of the differing amounts of fiber were to determine the optimal amount of fiber to add for increased strength. Additional segments were analyzed with the optimal amount of fiber and decreased mild reinforcement to prove that the addition of fiber allows a reduction in mild reinforcement. All results are discussed in Chapter 5.
Table 4.2 Segments Modeled Using ANSYS

<table>
<thead>
<tr>
<th>Finite Element Model No.</th>
<th>Fiber Volume (% concrete vol.)</th>
<th>Mild Reinforcement (% required)</th>
<th>Compressive Strength (psi)</th>
<th>Tensile Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>100</td>
<td>6,250</td>
<td>625</td>
</tr>
<tr>
<td>2</td>
<td>0.25</td>
<td>100</td>
<td>7,112</td>
<td>631</td>
</tr>
<tr>
<td>3</td>
<td>0.25</td>
<td>0</td>
<td>7,112</td>
<td>631</td>
</tr>
<tr>
<td>4</td>
<td>0.50</td>
<td>100</td>
<td>7,187</td>
<td>655</td>
</tr>
<tr>
<td>5</td>
<td>0.50</td>
<td>0</td>
<td>7,187</td>
<td>655</td>
</tr>
<tr>
<td>6</td>
<td>0.75</td>
<td>100</td>
<td>7,281</td>
<td>724</td>
</tr>
<tr>
<td>7</td>
<td>0.75</td>
<td>0</td>
<td>7,281</td>
<td>724</td>
</tr>
<tr>
<td>8</td>
<td>1.0</td>
<td>100</td>
<td>7,393</td>
<td>863</td>
</tr>
<tr>
<td>9</td>
<td>1.0</td>
<td>0</td>
<td>7,393</td>
<td>863</td>
</tr>
</tbody>
</table>
DISCUSSION OF RESULTS

5.1 FEM Stresses

An explanation of the stress results available in ANSYS 10.0 is provided in this section in order to provide a general understanding of the results. ANSYS Version 10.0 (2004) offers a list of different stresses for viewing results. The most common stresses are X, Y, and Z component stresses. They are simply the stresses caused by the load placed on the segment in the X, Y, or Z direction. The X-direction, in the case of this study, is in the transverse direction of the segment. The X-component stresses represent the tensile bursting stresses found in the general zone. The Y-direction, in the case of this study, is in the longitudinal direction of the segment. The Y-component stresses are the stresses parallel to the ducts in the segment. The Z-direction, in the case of this study, is in the vertical direction of the segment. The Z-component stresses run vertically through the segment. The program also offers planar shear stress solutions. The choices are the XY, YZ, and XZ planes.

ANSYS Version 10.0 (2004) offers principal stress results. Principle stresses are the maximum and minimum normal stresses in a plane (Logan 2002). Since there are three planes: XY, YZ, and XZ, the program offers three principle stress solutions: 1st, 2nd, and 3rd, respectively. The maximum and minimum principle stress in each plane is calculated with the following equations:

\[
S_{1\text{max}} = \frac{(S_x + S_y)}{2} + \frac{(S_x - S_y)}{2} + T_{xy}^2 \frac{1}{2} + \frac{T_{xy}}{2}^2
\]

\[
S_{2\text{max}} = \frac{(S_y + S_z)}{2} + \frac{(S_y - S_z)}{2} + T_{yz}^2 \frac{1}{2} + \frac{T_{yz}}{2}^2
\]

\[
S_{3\text{max}} = \frac{(S_x + S_z)}{2} + \frac{(S_x - S_z)}{2} + T_{xz}^2 \frac{1}{2} + \frac{T_{xz}}{2}^2
\]

\[
S_{1\text{min}} = \frac{(S_x + S_y)}{2} - \frac{(S_x - S_y)}{2} + T_{xy}^2 \frac{1}{2} - \frac{T_{xy}}{2}^2
\]

\[
S_{2\text{min}} = \frac{(S_y + S_z)}{2} - \frac{(S_y - S_z)}{2} + T_{yz}^2 \frac{1}{2} - \frac{T_{yz}}{2}^2
\]

\[
S_{3\text{min}} = \frac{(S_x + S_z)}{2} - \frac{(S_x - S_z)}{2} + T_{xz}^2 \frac{1}{2} - \frac{T_{xz}}{2}^2
\]

where \(S_x, S_y, \) and \(S_z\) are the \(x, y,\) and \(z\)-component stresses (psi), respectively, and \(T_{xy}, T_{yz},\) and \(T_{xz}\) are the torsional stresses (psi) in the xy, yz, and xz planes, respectively.
ANSYS Version 10.0 (2004) also offers Von Mises (or equivalent/effective) stress results that are often used as failure criterion in design (Logan 2002). Von Mises stress can be calculated in relation to principal stress by the following equation:

\[ S_{VM} = \left( \frac{1}{2^{1/2}} \right) \times \left( (S_1 - S_2)^2 + (S_2 - S_3)^2 + (S_3 - S_1)^2 \right)^{1/2} \]

where \( S_1, S_2, \) and \( S_3 \) are the principle stresses (psi). There are other stress results available in ANSYS 10.0 (2004), however the stresses previously discussed are better suited for this study and will be used to analyze the segments modeled.

5.2 FEM Stress Results & Discussion

Stress results obtained from the FEM are presented and discussed in this section. A discussion of each analysis along with the reasoning behind the particular analysis follows. Individual analysis results are presented separately. Finally, a comparison of the analyses is presented.

Segment 1 (Table 4.2) was the control analysis that contained the design required amount of mild steel reinforcement (Figure 3.3) with no steel fiber reinforcement. This analysis was performed to provide predicted results to a solution in order to prove the validity of the model. It also gave a basis for which to compare results of segments with fiber. This segment was loaded with 1,256 kips of post-tensioning force on each face and the resulting stress contours at the general zone can be seen in Figures 5.1 through 5.7. Figures 5.1 through 5.3 show the X-, Y-, and Z-component stresses, respectively. Figures 5.4 through 5.6 show the 1\textsuperscript{st}, 2\textsuperscript{nd}, and 3\textsuperscript{rd} principal stresses, respectively and Figure 5.7 shows the Von Mises stress contour. These are shown in order to emphasize that all stresses were studied to provide confidence in the finite element model and results. Out of all of the stresses considered in this study, the X-component stress is the most important since it reflects the tensile bursting force in the general anchorage zone. Figure 5.8 shows a plot of the X-component stresses versus distance across the ducts in the general zone. Table 5.1 details the maximum X-component stress results in the general zone for segment 1.
Figure 5.1: X-Component Stress Contour in Segment 1

Figure 5.2: Y-Component Stress Contour in Segment 1

54
Figure 5.3: Z- Component Stress Contour in Segment 1

Figure 5.4: 1st Principal Stress Contour in Segment 1
Figure 5.5: 2\textsuperscript{nd} Principal Stress Contour in Segment 1

Figure 5.6: 3\textsuperscript{rd} Principal Stress Contour in Segment 1
Figure 5.7: Von Mises Stress Contour in Segment 1

Figure 5.8: X-Component Stress vs. Distance Across Ducts in Segment 1
Segments 2, 4, 6, and 8 contained 100% of the design required amount of mild steel reinforcement and 0.25%, 0.50%, 0.75%, and 1.0% steel fiber reinforcement, respectively. A post-tensioning force of 1,256 kips was applied to these models on each face. These analyses were performed in order to determine the effect of the corresponding percentage of steel fibers on the modeled segments. Figures 5.9 through 5.11 show the X-, Y-, and Z-component stress contours, respectively and Figure 5.12 shows the Von Mises stress contour for Segment 2. Figure 5.13 shows a plot of the X-component stresses versus the distance across the ducts in Segment 2. Figures 5.14 through 5.16 show the X-, Y-, and Z-component stress contours, respectively and Figure 5.17 shows the Von Mises stress contour for Segment 4. Figure 5.18 shows a plot of the X-component stresses versus the distance across the ducts in Segment 4. Figures 5.19 through 5.21 show the X-, Y-, and Z-component stress contours, respectively and Figure 5.22 shows the Von Mises stress contour for Segment 6. Figure 5.23 shows a plot of the X-component stresses versus the distance across the ducts in Segment 6. Figures 5.24 through 5.26 show the X-, Y-, and Z-component stress contours, respectively and Figure 5.27 shows the Von Mises stress contour for Segment 8. Figure 5.28 shows a plot of the X-component stresses versus the distance across the ducts in Segment 8. Table 5.1 also details the maximum X-component stresses in the general zones of segments 2, 4, 6, and 8.
Figure 5.9: X-Component Stress Contour in Segment 2

Figure 5.10: Y-Component Stress Contour in Segment 2
Figure 5.11: Z-Component Stress Contour in Segment 2

Figure 5.12: Von Mises Stress Contour in Segment 2
Figure 5.13: X-Component Stress vs. Distance Across Ducts in Segment 2

Figure 5.14: X-Component Stress Contour in Segment 4
Figure 5.15: Y-Component Stress Contour in Segment 4

Figure 5.16: Z-Component Stress Contour in Segment 4
Figure 5.17: Von Mises Stress Contour in Segment 4

Figure 5.18: X-Component Stress vs. Distance Across Ducts in Segment 4
Figure 5.19: X-Component Stress Contour in Segment 6

Figure 5.20: Y-Component Stress Contour in Segment 6
Figure 5.21: Z-Component Stress Contour in Segment 6

Figure 5.22: Von Mises Stress Contour in Segment 6
Figure 5.23: X-Component Stress vs. Distance Across Ducts in Segment 6

Figure 5.24: X-Component Stress Contour in Segment 8
Figure 5.25: Y-Component Stress Contour in Segment 8

Figure 5.26: Z-Component Stress Contour in Segment 8
Figure 5.27: Von Mises Stress Contour in Segment 8

Figure 5.28: X-Component Stress vs. Distance Across Ducts in Segment 8
Table 5.1: Comparison of Maximum X-Component Stresses

<table>
<thead>
<tr>
<th>Finite Element Model No.</th>
<th>Fiber Volume (% concrete volume)</th>
<th>Mild Reinforcement (% required)</th>
<th>Max. X-Component Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>100</td>
<td>175.1</td>
</tr>
<tr>
<td>2</td>
<td>0.25</td>
<td>100</td>
<td>105.0</td>
</tr>
<tr>
<td>4</td>
<td>0.50</td>
<td>100</td>
<td>97.0</td>
</tr>
<tr>
<td>6</td>
<td>0.75</td>
<td>100</td>
<td>95.7</td>
</tr>
<tr>
<td>8</td>
<td>1.0</td>
<td>100</td>
<td>99.2</td>
</tr>
</tbody>
</table>

Segments 3, 5, 7, and 9 contained no mild steel reinforcement and 0.25%, 0.50%, 0.75%, and 1.0% steel fiber reinforcement, respectively. These analyses were performed in order to study the behavior of segments that contain only steel fiber reinforcement in the corresponding amounts. Segments 3 and 5 failed at approximately 40% of the load application in the analyses. This was determined because the finite element model would not produce a converged solution past the 40% load step. Failure cracking occurred around the ducts and can be seen in Figures 5.29 and 5.30. Segments 7 and 9 sustained the entire load. However, severe cracks formed around the ducts similar to those in segments 3 and 5, and can be seen in Figures 5.31 and 5.32. It may be inferred that mild steel in the general zone cannot be completely replaced by steel fiber, at least in the amounts studied. Therefore, these cases were not further investigated herein. See Table 5.2 for failure load results of segments 3, 5, 7, & 9.

Table 5.2: Failure Load Results

<table>
<thead>
<tr>
<th>Finite Element Model No.</th>
<th>Fiber Volume (% concrete volume)</th>
<th>Mild Reinforcement (% required)</th>
<th>Failure Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.25</td>
<td>0</td>
<td>502.4</td>
</tr>
<tr>
<td>5</td>
<td>0.50</td>
<td>0</td>
<td>502.4</td>
</tr>
<tr>
<td>7</td>
<td>0.75</td>
<td>0</td>
<td>1256</td>
</tr>
<tr>
<td>9</td>
<td>1.00</td>
<td>0</td>
<td>1256</td>
</tr>
</tbody>
</table>
Figure 5.29: Cracks in Segment 3 at Failure

Figure 5.30: Cracks in Segment 5 at Failure
Figure 5.31: Cracks in Segment 7

Figure 5.32: Cracks in Segment 9
Maximum stress versus amount of steel fiber results were plotted for segments 1, 2, 4, 6, and 8 and are shown in Figure 5.33. As expected, the curve shows that stresses decreased with the addition of steel fiber. From the curve, it can be seen that the greatest stress reduction occurred when 0.75% steel fibers were added to the concrete. It can also be seen that the addition of steel fiber in 1.0% amounts actually causes an increase in stresses in the general zone as compared to the 0.50% and 0.75%. This is likely due to the addition of steel fiber. Placement of steel fiber adds dead load to the segment. This increase in dead load causes an increase in stresses in all directions, since the steel fibers are placed in all directions. Table 5.3 shows the amount of load that was added to the general zone with each amount of steel studied. These loads are relatively small in comparison to the post-tensioning load and the increase in stress is relatively small in comparison to the lower fiber percentages.

Figure 5.33: Maximum X-Component Stress vs. % Fiber
Table 5.3: Load Addition due to Steel Fiber

<table>
<thead>
<tr>
<th>Segment</th>
<th>% Steel Fiber</th>
<th>Load Addition (lbs.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.25</td>
<td>68</td>
</tr>
<tr>
<td>4</td>
<td>0.50</td>
<td>135</td>
</tr>
<tr>
<td>6</td>
<td>0.75</td>
<td>203</td>
</tr>
<tr>
<td>8</td>
<td>1.0</td>
<td>270</td>
</tr>
</tbody>
</table>

Based on the comparison in Figure 5.33, it was reasoned that by adding 0.50% steel fiber the mild steel reinforcement could be decreased. The selection of 0.50% was made due to the fact that steel fiber in this amount will be more workable in construction situations than higher amounts, as previously discussed in Chapter 2. So, the next task involved finding the maximum amount by which the mild steel reinforcement could be reduced with 0.50% fiber application. Several FEM analyses were performed in order to complete this task. Three additional segments were loaded with 1256 kips and studied. Table 5.4 shows results for steel amounts and X-component stress values in comparison to the control analysis.

Table 5.4: Comparison of Additional Segments

<table>
<thead>
<tr>
<th>Segment</th>
<th>Fiber Volume (% concrete volume)</th>
<th>Mild Reinforcement (% required)</th>
<th>Max. X-Component Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>100</td>
<td>175.1</td>
</tr>
<tr>
<td>10</td>
<td>0.50</td>
<td>50</td>
<td>129.3</td>
</tr>
<tr>
<td>11</td>
<td>0.50</td>
<td>45</td>
<td>143.1</td>
</tr>
<tr>
<td>12</td>
<td>0.50</td>
<td>44</td>
<td>Failure</td>
</tr>
</tbody>
</table>

Segment 10 contained 0.50% steel fiber and 50% design required mild steel reinforcement. Figure 5.34 shows the X-component stress contour of segment 10 at the general zone. Figure 5.35 shows a plot of X-component stresses versus distance across the ducts. The resulting maximum X-component stress was lower than the control analysis; so further reduction of the mild steel reinforcement was possible. Segment 11 contained 0.50% steel fiber and 45% design required mild steel. Figure 5.36 shows the
X-component stress contour of segment 11 at the general zone. Figure 5.37 shows a plot of X-component stresses versus distance across the ducts. The maximum stress was still lower than the control analysis. In a final attempt to determine the maximum reduction, segment 12 was analyzed with 0.50% steel fiber and 44% mild steel reinforcement. The segment failed at 70% load application, or at approximately 879 kips. The cracking around the ducts was similar to segments 7 and 9 and is presented in Figure 5.38.

Figure 5.34: X-Component Stress Contour in Segment 10
Figure 5.35: X-Component Stress vs. Distance Across Ducts in Segment 10

Figure 5.36: X-Component Stress Contour in Segment 11
Figure 5.37: X-Component Stresses vs. Distance Across Ducts in Segment 11

Figure 5.38: Cracks in Segment 12 at Failure
The maximum permissible reduction of mild steel reinforcement with the addition of 0.50% steel fiber was 65% in the general zone. However, the segment failed with a 66% reduction. Therefore, reducing the mild steel reinforcement in the general zone by 65% is not recommended. The optimal reduction would occur at 50% in order to be conservative and safe.

As previously mentioned, the addition of steel fiber adds load to the segment. The effect of this was discussed solely for the general zone of the segment. However, it is reasoned that the increased load would also impact other areas of the segment where fiber was added. Figure 5.39 shows the overall stress contour of segment 8, which contained 1.0% steel fiber and 100% mild steel reinforcement. Stress concentrations can be seen in the top slab of the segment near the web-flange connection and in the unsupported portion of the top slab of the segment near the diaphragm (where the plates are located). To a large extent this is due to self-weight of the segment in the corresponding areas, which also includes the weight of the steel fiber. The impact of this weight was not covered in this study.

Figure 5.39: Overall Stress Contour for Segment 8
5.2 Strut-and-Tie Model Results and Discussion

In order to compare FEM stress results and justify the use of X-component stresses, the strut-and-tie model (STM) method was employed. The STM that was drawn to represent Segment 1 in this study is shown in Figure 5.40. The lines defined by the letter C are compression struts (C1 through C5) and the line defined by the letter T is the tension tie (T1). The tendon force for each anchorage was placed to total 1256 kips (628 kips per anchorage). Figure 5.41 shows the resulting forces. Hand calculations arriving at these results can be seen in Appendix A.
The force in the tension tie in the STM was calculated to be 137 kips when loaded with 1256 kips on one side of the segment. In the study, 1256 kips was placed on both sides of the segment at anchorage locations. This situation would cause the force in the tension tie to double, which would be 274 kips. In order to compare this force to the FEM, the X-component stress in Segment 1 was used to calculate the force with the following equation:

\[ F = S_x \cdot A_{GZ} \]

\[ F = 175.1 \cdot 1728 \]

where \( S_x \) is the X-component stress in Segment 1 (psi) and \( A_{GZ} \) is the area of the general zone at the section across the ducts (in\(^2\)). When solved, the above equation totals approximately 302 kips, which is comparable to 274 kips (only about 10% difference). This justifies the use of X-component stresses from the FEM.
CONCLUSIONS & RECOMMENDATIONS

6.1 Conclusions

Based on the previously discussed results of the finite element models, conclusions can be made about the use of FRC in the general zone and are as follows:

1. The maximum reduction of mild steel reinforcement, with the addition of 0.50% steel fiber, is 65% in the general zone.
2. The stresses in the general zone of the segments with reduced mild steel reinforcement and 0.50% steel fiber are conservatively comparable to those in the control segment.
3. In segments that contain no mild steel reinforcement in the general zone, extensive cracking and, in some cases, failure occurs around the ducts in the area of the general zone.
4. The addition of steel fiber to the general zone in amounts above 0.50% can cause an increase in stresses.

6.2 Recommendations

Based on the previously discussed results and conclusions, recommendations may be made, including:

1. Steel fiber reinforced concrete is recommended for use in the general anchorage zone of post-tensioned bridge girders.
2. Mild steel reinforcement in the general zone can be effectively reduced by 50% when 0.50% steel fibers are added to the concrete volume.
3. Dramix ZP305 fibers were used in this study. However, other steel fibers available on the market may also be used for this purpose provided that they have similar strengthening properties.
4. Further investigation of the use of steel fibers in the general zone of post-tensioned bridge girders in the testing laboratory is recommended.
5. Further investigation of the impact of load applied with the addition of fibers to areas outside of the general zone is recommended.

6. It is also recommended that the potential use of steel fibers in other structural elements, including but not limited to, the web-flange connection in box girders, bridge piers, and prestressed piles be studied.
A.1 STM Calculations

Unsolved STM
\begin{align*}
\overline{C_1} &= 1' - 8'' = 1.67' \\
\overline{C_2} &= 4' - 25\frac{3}{8}'' = 4.23' \\
\overline{C_3} &= 4' - 25\frac{7}{8}'' = 4.22' \\
T_1 &= 3' - 5\frac{1}{2}'' = 3.416'
\end{align*}

\begin{align*}
2F_x &= 6038 - \frac{413}{4.22} C_2 = 0 \\
&\text{due to symmetry: } C_3 = 641.7 K \\
2F_y &= C_1 - \frac{5.90}{4.22} C_2 = 0 \\
&\text{due to symmetry: } C_1 = 136.9 K
\end{align*}

\begin{align*}
2F_x &= \frac{413}{4.22} C_2 - C_4 = 0 \\
&\text{due to symmetry: } C_5 = 628.0 K \\
2F_y &= \frac{5.90}{4.22} C_2 - T_1 = 0 \\
&\text{due to symmetry: } T_1 = 136.9 K
\end{align*}
Solved STM
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