Reusability and Impact Damage Repair of Twenty-Year-Old AASHTO Type III Girders
REUSABILITY AND IMPACT DAMAGE REPAIR OF TWENTY-YEAR-OLD AASHTO TYPE III GIRDERS

FINAL REPORT

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Abstract

Prestressed concrete has been used as a bridge construction method in the United States since 1949. Presently, there are thousands of pretensioned prestressed concrete bridges in service in North America. Each year, approximately 200 girders are damaged as a result of impact damage (primarily overheight vehicles striking a bridge from below). This thesis describes the results of a four girder test series which evaluated impact damage and repairs. The girders used for the study were fabricated in 1967 and placed in service. They were removed from service in 1984 as a result of a road realignment project. The objectives of the research project were: 1) to determine the effective prestress in the strands after 20 years, 2) to determine the influence of impact damage on girder performance, 3) to evaluate the performance of two impact damage repair schemes under static, fatigue, and ultimate loadings, and 4) to develop a model to estimate the strand stress ranges in damaged girders.
Acknowledgments

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Chapter 1 - Introduction

1.1 Bridge Construction

Many bridges in the United States are approaching the end of their design life. With fiscal constraints on federal, state, and local transportation agencies, it is important to determine the present structural condition of these bridges in order to make decisions on replacement, rehabilitation, and maintenance. Knowledge of the approximate life expectancy of a particular bridge, the potential reusability of its elements, and repair technologies available, allow transportation agencies to use their resources wisely.

This thesis is based on the results of a research project funded by the Minnesota Department of Transportation and National Science Foundation Grant BCS-8451536. The research was carried out in the Structural Engineering Laboratory in the Department of Civil and Mineral Engineering at the University of Minnesota. The primary aim of the research was to experimentally determine the structural condition of four prestressed bridge girders removed from an existing bridge after twenty years of service. In addition, the influence of impact damage on the static and fatigue load response of a girder was also studied. Finally, two impact damage repair methods were examined under static and fatigue loadings.

1.2 Prestressed Concrete Construction

Prestressed concrete construction utilizes two structural materials: high strength steel strands and high strength concrete. It capitalizes on the favorable properties of each material. Concrete is an excellent material in compression but has poor tensile properties, whereas slender steel elements have excellent tensile properties but exhibit buckling problems in compression.

Prestressed concrete, described in the most fundamental manner, utilizes "prestressed" steel within a concrete element to resist internal tensile stresses occurring within a concrete cross section.
The steel is tensioned to induce compressive internal stresses in the concrete. The compressive stresses caused by tensioning the steel or "prestressing" the concrete are usually larger than the tensile stresses caused from service loadings. Hence, full prestressing usually prevents the section from cracking under service loads. Figure 1.1, adapted from Collins and Mitchell[1] summarizes the differences between prestressed concrete and nonprestressed concrete beams. With no external loads present, a prestressed beam cambers upward. This is due to the eccentricity of the prestressing strands with respect to the centroid of the beam. Before cracking can occur in the prestressed cross section, the external loads must overcome the internal moment from the prestressing and the tensile capacity of the concrete section, whereas in the nonprestressed beam, the external loads only need to overcome the tensile capacity of the concrete section to produce cracks. The initial stresses and strains shown in Figure 1.1 do not include those due to the dead weight of the beam.

Prestressing allows concrete to be used in long construction spans unattainable with conventional reinforced concrete. Prestressed concrete elements exhibit smaller deflections and fewer corrosion problems than similar reinforced concrete elements.

While prestressed concrete has many advantages, there are both economic and design constraints. The most significant drawback is associated with time-dependent effects. The time-dependent material properties of prestressing steel (relaxation), and concrete (creep and shrinkage), reduce the effective prestress with time. An underestimation of the prestress losses may result in cracking of the element, while an overestimation may cause excessive camber. Because prestressed concrete is a relatively new bridge construction practice, there is little data available on long term prestress losses.

In the 1920's, Eugene Freyssinet, a French engineer, was the first to successfully demonstrate the concept of prestressing elements[2]. His work succeeded because he recognized the need to account for the time-dependent properties of concrete. Freyssinet utilized what is now known as post-tensioning, while pretensioning was
successfully applied in practice by the German engineer, E. Hoyer, in 1938[3].

The use of prestressed concrete as a construction method for bridges has been used in the United States since 1949 with the construction of the Walnut Lane Bridge in Philadelphia, Pennsylvania. Since that time, use has increased so that as of 1980, Shanafelt and Horn[4] reported over 23,000 prestressed concrete bridges in service in the United States and Canada. Shanafelt and Horn also reported, that of the prestressed bridges in service, 54% were constructed with I-sections, 26% with box sections, 12% bulb or multiple tees, and the remaining 8% were constructed with slab sections. This breakdown is shown in Figure 1.2.

Today most of the bridges in the 60 to 120 foot span range are designed with pretensioned prestressed concrete elements. A growing percentage of the bridges in the 150 to 650 foot span range are constructed with continuous or cantilevered prestressed concrete[3].

1.3 Prestressed Concrete Bridges in Minnesota

The State of Minnesota began using prestressed concrete for bridge construction in 1954[5]. At the end of 1987, over 1,000 prestressed concrete bridges had been constructed by the Minnesota Department of Transportation (MnDOT). Figure 1.3 shows the number of prestressed bridges constructed each year from 1954 through 1989. As of 1989, there were 13,082 bridges over 20 ft. in length maintained by public agencies in Minnesota. Over 7,000 of these were constructed since 1954. Figure 1.4 shows how prestressed concrete has become a significant portion of the new bridges constructed.

The National Bridge Inventory reported that fifty percent of the 5000 new bridge projects each year are constructed with prestressed concrete[6]. In addition to a large percentage of the new construction being prestressed, the performance of prestressed concrete is excellent. Only five percent of the prestressed concrete bridges in service (constructed between 1950 and 1988) are classified as structurally deficient. This compares to approximately eight percent for reinforced concrete, over twenty percent for structural
steel, and nearly fifty percent of the timber bridges constructed during the same time period[7]. With a large number of prestressed bridges currently being constructed, the information regarding the reusability and repair of prestressed concrete bridge girders will be of importance in the future.

1.4 Damage Statistics of Prestressed Concrete Bridges

Occasionally prestressed concrete bridge elements are damaged. This damage may be a result of fabrication, fire, impact damage, or other causes. Results of a survey conducted by Shanafelt and Horn as part of NCHRP Project 226 indicated, that on the average there are 201 damages per year. Figure 1.5 shows a breakdown of the causes of damage. Over 80% are a result of overheight vehicles striking a bridge from below, 2.5% are fire related, and the remaining 16+% are from a variety of other sources.

1.5 Problem Statement

The main objectives of this research project were:

(1) to determine the viability of reusing prestressed girders which had been removed from bridges for reasons other than structural integrity.

(2) to assess the amount of damage that prestressed concrete girders can sustain before major repairs are required.

(3) to study two repair techniques for restoring prestress to girders which had severed strands due to simulated impact damage.

To determine the feasibility of reusing or repairing the girders, a number of questions needed to be answered quantitatively or qualitatively:

(1) What are the actual prestress losses of these girders?

(2) What are the material properties and the load behavior characteristics of the girders?

(3) What is the behavior of an impact-damaged girder?

(4) How do girder repairs perform under static and fatigue loadings?
The first of the four girders donated for this project was designated as the control specimen. In girder one (G1), prestress losses were measured, material properties determined, and fatigue and ultimate loadings performed. The behavior of an impact damaged specimen was studied with girder two (G2). Girders three and four (G3 and G4) were used to study the behavior of damaged and repaired specimens.

1.6 Methods Used to Resolve the Problem Statement

To achieve the goals of this research project, an extensive experimental program was conducted. Response of the girders to various static and fatigue loadings were measured during full-scale tests. The girders were loaded with hydraulic actuators to induce various levels of stress. The local and global response of each specimen was monitored using strain gages, crack detection gages, load cells, and linear voltage differential transformers (LVDTs). In addition, the placement of strain gages on the reinforcing bars of the composite deck cast on the fourth girder enabled measurement of composite deck strains.

In conjunction with the experimental program, numerical modeling of behavior was conducted utilizing the program PBEAM[7]. PBEAM, a discrete element program, enabled estimation of the instantaneous and time-dependent response of the girders. For this project, PBEAM was used to estimate long-term prestress losses, cracking load, the stress ranges in strands, the influence of adding a new composite deck, and finally, the ultimate load response of the girders.

A short computational method was developed to allow an engineer to estimate strand stress ranges in impact-damaged prestressed concrete elements. With the stress ranges, an estimate of the remaining service life can be made.

1.7 Implications of Research

The results from this research project will enable design engineers to make informed decisions with regard to prestressed girders. The short computational method for determining stress ranges
in strands, allows engineers to estimate fatigue life. The method is applicable for both damaged and undamaged elements.

Should repair become necessary for a damaged girder, the performance of two repair schemes is presented. The repairs were evaluated under static, fatigue, and ultimate loadings to the test girders.

1.8 Organization of Thesis

Chapter 2 contains a review of literature relevant to the testing of prestressed concrete beams. First the static, fatigue, and time-dependent properties of the component materials are discussed; second, past beam tests relevant to this research are examined; and lastly, impact damage studies are referenced.

Chapter 3 gives a description of the girders tested, discusses the addition of the new composite deck, and describes the material properties of the specimens and repairs.

Chapter 4 provides a description of the test setup, the rationale for the setup, the loading system, instrumentation, and the loading sequences.

Chapter 5 provides a discussion on the background and design of the two repair schemes implemented. Internal strand splices were used to repair girder 3, while an external post-tensioning scheme was used with girder 4. The performance of each repair is included in the chapter describing the tests conducted on the appropriate girder.

Chapters 6-9 summarize the tests performed on each of the four girders. Each girder was subjected to an initial cracking test, a severe fatigue loading program, and finally loaded to failure. Girder 1 served as the "control" specimen; girders 2-4 were subjected to a damage or damage/repair procedure. The damage in girders 2-4 was intended to simulate impact damage caused by an overheight vehicle striking the side of the girders. The damage procedure consisted of removing a portion of concrete from the bottom flange of the girders and severing at least two prestressing strands. The repair schemes implemented on girders 3 and 4 are described in Chapter 5.
Chapter 10 gives the modeling results obtained with the program PBEAM. Included are estimates of prestress losses, cracking load, effect of a new composite deck, strand stress ranges, and ultimate load response.

Chapter 11 describes the short numerical method used to estimate strand stress ranges in a damaged girder. A number of parameters, (strain discontinuity, material parameters, effective prestress, etc.) were examined to determine their influence on stress ranges.

Chapter 12 summarizes the prestress loss estimates for the girders. Both indirect and direct experimental results are presented. The experimental values are compared with numerical results obtained with the program PBEAM and with AASHTO and PCI procedures.

Chapter 13 provides a summary of the project, including major conclusions and recommendations for further research.
Chapter 2 - Background

2.1 Introduction

The prestressed girders examined in this research project were composed of three materials: concrete, mild steel, and high-strength steel. Concrete, the largest component of the girders is a composite material. It is a combination of coarse aggregate, fine aggregate, water, and hydraulic cement. Mild steel was used as reinforcement in both the composite deck and the girder. High-strength seven-wire steel strand was used to prestress the girder.

The behavior of prestressed concrete beams subjected to fatigue loadings have been studied from two perspectives. From one vantage, the fatigue behavior of concrete, mild steel, and prestressing strand have been studied as individual components. And from the second, the fatigue behavior of complete prestressed elements has been investigated. This chapter begins with a description of individual component materials. It then continues by summarizing past prestressed beam and girder tests, and concludes with a summary of prestressed concrete girder damage studies.

2.2 Component Behavior

2.2.1 Behavior of Concrete

Concrete is composed of local coarse and fine aggregates, portland cement, water, and possibly admixtures. The possible combinations of components is endless.

The response of concrete has usually been correlated to its compressive strength. Provided a reasonable mix of coarse and fine aggregates are used, the primary strength design parameter is the water to cement ratio. Decreasing the water to cement ratio increases the compressive strength of the material.

Concrete varies with time. As time progresses the material gains strength, shrinks, and creeps (if under sustained load). Each of these effects are examined more closely in the following sections.
2.2.1.1 Static Behavior

The static behavior of concrete has usually been correlated to its compressive strength obtained from uniaxial cylinder tests. It has been shown that the strength is dependent on the initial water-to-cement ratio, the amount of entrained air, the type of cement used, and the temperature and humidity at which it is cured. In addition, the testing procedure, the type of capping, the type of testing machine used, and the strain rate of the test have a large impact on results.

Many researchers have used the equation developed by Hognestad[8] to analytically model the compressive stress-strain behavior of concrete. The equation fits a parabola to the loading portion of the curve and a straight line to the unloading portion. Figure 2.1 shows Hognestad stress-strain curves obtained for compressive strengths of three, four, and five thousand psi concrete. The peak loads shown in Figure 2.1 do not reach the nominal values of three, four, or five thousand psi. The model assumes that within a structural element, the concrete can only attain 85 percent of its nominal value. While concrete is able to carry some tension, this contribution is generally ignored in design. The tensile capacity is approximately 1/10 of its compressive capacity.

2.2.1.2 Fatigue Behavior

ACI Committee 215 (Fatigue of Concrete) issued a report which detailed "Considerations for Design of Concrete Structures Subjected to Fatigue Loading"[9]. The recommended maximum stress range for concrete varies with the minimum stress. The larger the minimum stress the smaller the allowable stress range. Figure 2.2, adapted from the report, is based on a design fatigue loading of one million cycles. To use Figure 2.2, one enters the graph from the left with the minimum stress, moves to the right until one intersects the lower line, moves vertically until one encounters the upper line, and from there, one goes to the right border and reads off the allowable upper stress.
2.2.1.3 Time-Dependent Effects of Concrete

2.2.1.3.1 Strength Aging of Concrete

Concrete is usually designed with a twenty-eight day compressive strength. Due to continued hydration, the strength continues to increase with time. Figure 2.3 shows a plot of the relationships suggested by ACI Committee 209[10]. The lines labelled MC-I and MC-III describe the strength aging of moist-cured concretes composed of Type I and Type III cements, respectively. Similarly, SC-I and SC-III represent the strength-time relationship of steam-cured concretes. The type of cement used and the type of early curing used affect how much strength the element will eventually attain. For the same mix design, moist-cured (MC) concrete gains more strength than steam-cured (SC), and concrete made with Type I cement gains more strength than that made with Type III cement. After twenty years, the compressive strength of concrete increases between two and eighteen percent above the twenty-eight day strength.

2.2.1.3.2 Shrinkage

Concrete is affected by the amount of moisture (or lack of) in its environment. If it is in a dry environment, it will lose moisture and hence volume. Figure 2.4 shows a plot of the Prestressed Concrete Institute suggested shrinkage strains for a concrete element with a volume to surface ratio of two and subjected to an average ambient relative humidity of seventy percent. The majority of the shrinkage strain takes place during the first six months of curing. Strains near 450-500 microstrain develop after three years of curing. The larger the volume-to-surface ratio (V/S), the smaller the amount of ultimate shrinkage. At final times, the correction factor for a V/S of 1 is 1.05 while the correction for a V/S of 6 is 0.54. Basic shrinkage strains also require correction for relative humidities. Forty percent relative humidity has a correction of 1.43, seventy percent has no correction, and ninety percent relative humidity has a correction of 0.43. A relative humidity of 100 percent produces zero shrinkage, as one would expect. An element with a high volume-to-surface ratio in a humid environment will experience minimal shrinkage.
strains, while on the other hand, a concrete element with a large water-to-cement ratio, a small volume-to-surface ratio, and located in a dry environment will experience large shrinkage strains.

2.2.1.3.3 Creep

Under constant stress, the strain in concrete increases with time. This increase in strain is known as creep. It can be two to three times as large as the instantaneous strain caused by the applied stress. The results of creep studies are quite scattered; estimates within plus or minus thirty percent are considered good[11]. ACI Committee 209 has suggested a basic creep equation shown plotted in Figure 2.5. This curve is based on a concrete with four inch or less slump, 40 percent ambient relative humidity, and loads applied at seven days for moist-cured elements and one to three days for steam-cured concrete elements. A number of correction factors are applied to this curve. Time of loading other than the above specified times, relative humidity different from forty percent, minimum thickness of the member, different slumps, percent fine aggregate and air content of the concrete are all associated with independent creep correction factors. The final creep coefficient is the multiplier by which the elastic strain is multiplied to determine the total long-term strain of the element.

2.2.2 Behavior of Mild Steel

Mild steel is usually utilized as deformed reinforcing bars in grades of steel with yield strengths of 40, 50, 60, or 75 ksi. In addition to deformed reinforcing bars, deformed bar mats, zinc-coated bars, epoxy-coated bars, plain and deformed wires, and welded-wire fabric are used as mild reinforcing steel. The mild steel within the test girders was uncoated deformed bars. Time-dependent effects such as creep or relaxation are usually of such small magnitude in mild steel that they are ignored in design.
2.2.2.1 Static Behavior

The static behavior of mild reinforcing steel in either tension or compression is usually idealized as an elastic perfectly plastic material. Typical idealized curves for Grade 40, 60, and 75 steel are shown in Figure 2.6. The modulus of elasticity for steel is usually assumed to be 29 or 30 million psi. Mild steels are ductile, with minimum elongations of seven to twelve percent required by the American Society of Testing and Materials (ASTM).

2.2.2.2 Fatigue Behavior

Researchers are divided as to whether or not the minimum state of stress affects the fatigue life of mild reinforcement[12]. The smallest stress range known to have failed an embedded bar is 17.5 ksi, and this was after 1.25 million cycles above a minimum stress of 21 ksi. In the data reported by ACI Committee 215, there appears to be an endurance limit near one million cycles. Figure 2.7, adapted from ACI 215, show the results of fatigue tests performed on mild steel.

2.2.3 Behavior of High-Strength Strand

Seven-wire prestressing strand is one of the most popular prestressing steel products. It is composed of seven high-strength steel wires; six wires wrapped around a center wire of slightly larger diameter. The pitch of the wrapped wires is between twelve and sixteen strand diameters. Three different types of seven-wire strand are produced: stress-relieved, low-relaxation, and compacted.

The original seven-wire strand used in prestressing applications was stress-relieved and had tensile strengths of 250 or 270 ksi. At 1 percent elongation, stress-relieved strand must reach a stress level equal to at least 85 percent of its tensile strength.

Low-relaxation strand is also produced in 250 and 270 ksi strengths and is identified by two performance criteria. First, the relaxation losses after 1000 hours should not be more than 2.5 percent at an initial loading of 70 percent of the tensile strength or 3.5 percent at a loading of 80 percent. Second, the stress, measured at 1
percent elongation must not be less than 90 percent of the tensile strength.

Compacted strand is a recent development. Compacted strand is produced by drawing strand through a die and then stress-relieving it. It is produced in three nominal diameters, 0.5 inch, 0.6 inch, and 0.7 inch. Each of the three compacted strand diameters has a different tensile strength.

All three products, standard stress-relieved strand, low-relaxation strand, and compacted strand, are currently available from suppliers[13]. The test girders were prestressed with 250 ksi stress-relieved strand.

2.2.3.1 Static Behavior

The Prestressed Concrete Institute (PCI) Design Handbook recommends two stress-strain equations for seven-wire strand. One for strand with an ultimate stress of 250 ksi and another for 270 ksi strand. Figure 2.8 shows a plot of both curves. ASTM requires a minimum elongation of 3.5 percent for seven-wire strand products. The modulus of elasticity assumed in the PCI equations is 28 million psi.

2.2.3.2 Fatigue Behavior

In 1957, Ekberg, Walther, and Slutter[14] from Lehigh proposed a modified Goodman diagram for prestressing strand. It is shown in Figure 2.9. This design figure is based on a fatigue loading of one million cycles. Similar to the Goodman diagram for concrete, the allowable stress range is a function of minimum stress. The larger the minimum stress, the smaller the allowable stress range.

In 1983, Paulson, Frank, and Breen[15] at the University of Texas, Austin, fatigue tested strand from six different manufacturers. From the reported test data in the literature and the 67 tests they performed, Paulson, et al. suggested a design equation based on a statistical analysis of the test data. The equation:

\[ \log N = 11.0 - 3.5 * \sigma \]
is plotted in Figure 2.10. \( S_r \) is the stress range in ksi and \( N \) is the number of cycles allowed. Paulson, et al. acknowledged that minimum stress was a significant variable in the stress life of strand, but to simplify the design process, chose to develop an equation without a minimum stress variable.

2.2.3.3 Relaxation

Relaxation is a time-dependent phenomenon of prestressing steel. High-strength steel stressed above fifty percent of its ultimate strength will undergo a significant loss of stress if placed within a constant strain field. Figure 2.11, developed from the relaxation equation of Magura, Sozen, and Siess[16] shows how the stress drops with time for three different initial stress levels. Shown are curves for initial stresses of 60, 70, and 80 percent of yield. The equation was based on work with stress-relieved strand. In most stress-relieved strand applications, the strand is initially tensioned to 70 percent of its yield stress. From the curve, after about thirty years, the strand has lost 7 percent of its initial stress. Similar equations have been developed for low-relaxation strand, these are also shown in Figure 2.11. The low-relaxation (lo-lax) curves were based on Ontario Highway Bridge Code[17] equations. This figure shows that the relaxation of stress-relieved strand is two to three times larger than that of lo-lax strand.

2.3 Tests of Beams and Girders

The research involved in testing prestressed concrete beams and girders has matured with the industry. Most of the early tests were performed to examine the behavior of a single parameter (e.g. 7/16 inch strands, lightweight concrete, draping). The most extensive experimental test series was performed at the University of Texas. Eleven composite girders were tested, examining a variety of parameters (overloads, cross sections, passive reinforcement). A short summary of the significant tests follows.

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Tests by Abeles[18] in 1954 sought to determine the static and fatigue behavior of partially prestressed concrete elements. The elements he tested were prestressed with 0.2 inch diameter wires.

Later, in 1956 Ozell and Ardaman[19] confirmed that the use of 7/16 inch diameter seven wire strand was an appropriate prestressing steel for fatigue applications.


Ozell and Diniz[22] in 1958 studied composite pretensioned prestressed concrete beams under fatigue loading. One of their conclusions was startling. "The coefficient of differential shrinkage plays an extremely important part in composite-beam stresses. The magnitude of this coefficient must be properly chosen for an accurate analysis or design. For beams used in these tests the difference in the live-load safety factor varied almost 100 percent when based on two differential shrinkage coefficients of 0.0002 and 0.0004."

Later in 1958, Ozell and Diniz[23] reported on the feasibility of using 1/2 inch diameter seven wire strand. They concluded that for their study, the 1/2 inch strand performed well.

In 1962, Warner and Hulsbos[24] sought to determine the fatigue life of prestressed concrete beams. Their research compared an analytical model to experimental results. They concluded that "The variability in predicting beam fatigue life is likely to be much greater than the variability indicated by the strand fatigue data."

Also in 1962, Ozell[25] examined the effect of strand hold downs on the performance of prestressed beams. Specifically he sought to determine if the hold-downs reduced the fatigue life of the element. His research found all the strand or wire failures to be located at crack locations rather than at hold-down points.

In 1965, Hanson and Hulsbos[26] sought to determine if fatigue failure of shear reinforcement was a limiting design constraint. They found that with 1/2 of the web reinforcement
required by AASHO, they did not see a fatigue failure under design loads.

Kaar and Magura[27] conducted a study to explore the effect of blanketing strands. Other than an increased development length the blanketed strand specimens performed well.

In 1970, Hanson, Hulsbos, and Van Horn[28] studied the fatigue life of prestressed beams which had been overloaded to cause flexural and inclined cracking prior to repeated loading. Their main conclusion was "The flexural fatigue life of the test beams was less than expected from the available information on the fatigue strength of strand. Additional experimental information is needed on the effect that conditions at cracks have on the fatigue strength of strand in a beam. Until this information becomes available, these tests indicate that the nominal tensile stress in the bottom fibers of prestressed concrete beams subjected to a large number of cycles of repeated loading should be limited to $6\sqrt{f'c}$.

In 1984, Overman, Breen, and Frank[29] reported the results of an extensive study at the University of Texas. The objective of the study was to summarize the flexural fatigue behavior of eleven full-scale precast pretensioned girders with unshored composite slabs. The main experimental variables included: (1) maximum load level as indicated by the nominal concrete tensile stresses based on uncracked gross section calculations, (2) girder strand stress ranges, (3) cross sections (AASHTO-PCI Type II and Texas Type C girders), (4) strand patterns including both draped and straight strands, (5) provision of passive reinforcing steel in the precompressed tension zone, (6) distribution and confinement of passive reinforcement, (7) degree of precracking of sections, and (8) the presence or absence of occasional modest overloads during static tests.

Overman, Breen, and Frank made the following conclusions:

(1) Pretensioned concrete bridge girders without well-distributed, confining passive reinforcement which are actually subjected to loads producing $6\sqrt{f'c}$ can fail as a result of fatigue of prestressing strands at less than three million cycles.
(2) The fatigue life of pretensioned concrete girders can be predicted using the following modification of the strand fatigue model developed by Paulson. \[ \log N = 11.0 - 3.5 \log \text{Sr} \]

where:  
- \( N \) is the fatigue life in numbers of cycles
- \( \text{Sr} \) is the strand stress range; maximum stress - minimum stress (ksi)

No endurance limit can be set, but it appears that stress ranges below 5 ksi would be insignificant. The probable stress range must be determined accurately or conservatively considering the effects of prestress losses, of section cracking, and the probability of overloads.

(3) In order to relate the design of pretensioned concrete girders to the general AASHTO fatigue provisions, it appears adequate to design these girders for stress ranges similar to AASHTO structural steel Category B values for redundant load path structures.

(4) A small number (less than 10) of occasional modest overload cycles (20 percent above the normal applied load level) can produce extremely detrimental effects in the form of increased strand stress ranges and sharply reduced fatigue lives.

(5) Prestress losses directly influence strand stress ranges and hence fatigue life. Realistic, conservative estimates of minimum prestress forces must be used in assessing girder fatigue life.

(6) Confined, well-distributed passive reinforcing steel can substantially increase fatigue life by reducing prestress losses, controlling crack propagation, and thus limiting strand stress ranges.

(7) The use of the current AASHTO Specification allowable concrete tensile stress of \( 6\sqrt{f'c} \) without specific inclusion of well-distributed, confined supplementary reinforcement will not ensure adequate fatigue life.
2.4 Tests of Specimens Which Had Been in Service

Relatively little information has been published on the performance of prestressed girders which have been in service for an extended period of time. Information on two tests is presented in the following sections.

2.4.1 Belgium Test[30]

Two thirty-year-old prestressed concrete beams were tested at the "Laboratorium Magnel voor Gewapend Beton" Rijksuniversiteit Gent. The girders had a span of 94.5 feet and a varying I-shaped cross section. The minimum depth of the section was 38.8 inches and the maximum depth was 44.1 inches. The width of the cross section was 20.3 inches.

No fatigue loadings were applied to the specimens. Each was loaded to failure individually. The first beam failed at a load of 61.2 kips and a deflection of 20.7 inches. The second beam failed at a load of 67.8 kips and a deflection of 12.6 inches.

2.4.2 PCA Test - 1984

Three twenty-five-year-old girders were removed from a bridge with the deck section in place and were tested[31] at the Construction Technology Laboratories in Skokie, Illinois. The bridge was removed due to road realignment work. Each girder was forty-two feet long and weighed forty tons. Ultimate failure occurred at 285 kips and a deflection of 14.7 inches. Initial prestress was specified at 175 ksi. The effective prestress was calculated and based on the observed decompression moment.

The tests indicated[32] that the total prestressing loss was 19.1 ksi. Using the 1954 U.S. Department of Commerce specifications to compute losses, the expected total prestressing loss was 35.4 ksi. The total prestressing loss calculated from the present AASHTO and ACI codes was 42.1 and 40.6 ksi, respectively.

2.5 Damage Studies of Prestress Concrete Beams

Shanafelt and Horn, consulting engineers from Olympia, Washington, published two National Cooperative Highway Research
Program (NCHRP) Reports regarding damage to prestressed concrete bridges. The reports are described in the following sections.

2.5.1 NCHRP Report 226

NCHRP Report 226[4] was titled "Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members." The objectives of the study were: (1) to provide guidance for the inspection and assessment of accidental damage to prestressed concrete bridge members and (2) to identify, develop, and evaluate the effectiveness of repair and replacement techniques.

Shanafelt and Horn surveyed the transportation agencies of the fifty states, the provinces of Canada, and several railroad companies. The survey was intended to determine a number of damage related statistics. The frequency of damage, how significant the damage was, who was responsible for inspecting and deciding whether repair or replacement was warranted. In addition, repair schemes used by the various agencies were evaluated.

The major cause of accidental damage to prestressed members in the United States is due to impact from overheight vehicles. On the average, there are 162 impact damages per year, 5 fire damages per year, and 34 "other" damages per year.

Eleven different repair methods (including strand splices, metal sleeves, and external post-tensioning) were evaluated as to their ability to restore the damaged element to its original condition. The repairs were evaluated for cost, esthetics, and speed of repair among other things. In addition, four different schemes for applying preload to the damaged element during the "patching" phase of repair were discussed in detail.

2.5.2 NCHRP Report 280

This report was titled, "Guidelines for Evaluation and Repair of Prestressed Concrete Bridge Members"[33].

Five tasks were to be completed in this report:
(1) Evaluate available information on selected repair methods.
(2) Develop concepts for improved repair methods and perform component testing as needed.

(3) Perform full-scale tests to demonstrate the effectiveness of the repair methods developed in Task 2.

(4) Prepare manual of recommended practice.
   - Inspection of damage
   - Guidelines for assessment of damage
   - Selection of repair method
   - Guidelines for repair of damage

(5) Preparation of final report.

Ten different static full-scale load tests were performed in addition to three component tests. (One-strand internal splice, two-strand internal splice, and external post-tensioning corbels). All of the load tests were performed on a single girder which was repaired and retested with different repair schemes.

2.6 Current Research

This chapter has presented background information in three different areas of prestressed concrete construction. The areas are: component material properties, tests of beams and girders, and damage studies.

The research program described in the following chapters presents information related to static and fatigue tests on girders. The girders were tested in undamaged, damaged, and repaired states. New areas studied are; girders subjected to nonsymmetrical damage and the fatigue performance of two repair schemes.

With nearly fifty percent of the new bridges constructed being fabricated with prestressed concrete, the information from this research will be of use to future designers, bridge raters, and researchers.
Chapter 3 - Girders and Materials

3.1 Girder History - Bridge Data

The girders were removed from Minnesota Bridge Number 27915, a four span bridge carrying Boone Avenue over Interstate 694. The bridge was located in Brooklyn Park, a northern, first tier suburb of Minneapolis. The thirty-five foot wide overpass was designed using the 1961 AASHO code and constructed during the summer of 1967. The two end spans were 45 feet long and supported by three girders, while the two interior spans were 64 feet 8 inches long and supported by five girders. The test specimens were removed from the interior spans, where the girders were placed on seven foot centers. The overpass was not orthogonal to the interstate, but skewed twenty-eight degrees. For the interior spans, diaphragms were placed at the third points.

3.2 Removal of Girders from Original Bridge

The overpass was removed as a result of road realignment. The contractor, C.S. McCrossan of Osseo, Minnesota, removed the old bridge by cutting the slab between girders, breaking the diaphragms, and transporting the girders to a storage yard. The girders were removed from the bridge in 1984. They remained in the storage yard until January of 1987. Before transport to the University, the remaining slab and diaphragms were removed from the test specimens. This minimized the amount of weight the cranes would carry while lowering the girders into the underground Structural Engineering Laboratory.

3.3 Records from MnDOT

The Minnesota Department of Transportation records of bridge number 27915 were sent to the University as background information for the project. Among the records were inspection reports of the bridge, the material reports of the fabricator, the design calculations, and a copy of the original plans.

The design calculations were dated July 30th and 31st of 1962. The impact factor applied to the live load was 26.4 percent. The

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applied live load moment per girder was computed as 750 kip-feet, and the total dead load moment per girder was 804 kip-feet. Mu, the factored moment was computed from the following equation:

\[ Mu = 1.5 \times \text{Dead Load} + 2.5 \times \text{Live Load} \]

\[ = 3081 \text{ kip-feet} \]

The girders had a nominal capacity of 3955 kip-feet, well above the 3081 kip-feet required.

The original plans show that the first drawings for the bridge were filed in July 1963. Three intermediate revisions were performed before the final revision of November 1964.

Fabrication of the girders took place during the last half of July 1967. The fabrication records of Prestressed Concrete Incorporated include data on four of the ten interior girders. Of the four reported, three were interior girders and one was a facia girder. The 1/2 inch diameter seven-wire strand was supplied by Roebling with a specified elongation constant of \(0.763 \times 10^{-10} -25.2 \text{ kips} \). The load required to initially prestress each strand to 0.70 fu (175 ksi) was 25.2 kips. Under this load, a ten foot length of strand would elongate 0.763 inch. The length of strand pulled was 184 feet and each strand was tensioned separately. A two kip preload was put on all of the strands resulting in elongations of 1-5/32 inch. At transfer of prestress, the camber in the girders varied between 3/4 and 7/8 inch.

Information concerning the girder concrete was included with the records. The girders were poured in twenty-five to thirty minutes. The cement brand was Universal Atlas Type III. Daratard M.C. admixture was added to the girder mix at a rate of nine ounces per yard. Slumps measured between 2 and 2.5 inches. At transfer of prestress, the minimum girder concrete compressive strength was specified as 4500 psi. Average cylinder strength at transfer (one day) was 4905 psi. The average 28 day cylinder strength was 6773 psi (5000 psi was assumed for design).

The inspection reports show that extensive inspections were performed on an annual basis beginning in 1971 and continuing through 1984. As of 1971, the estimated remaining life was 50 years with a
normal load rating of an HS-20. Expansion joint details appeared as the first inspection problem. Leaking joints and restrained joints appeared in 1971 and continued through 1984. In 1978 records show that strand at the girder ends were starting to rust. Interestingly, in 1982 it was reported that facia beams had been hit over both roadways with minor damage. From the 1984 structure inventory sheet, the capacity ratings were: operating HS 47.0 and inventory HS 27.0. The sufficiency rating was 82.4.

3.4 Transport of Girders to the University

The girders were loaded at the storage yard with two truck cranes supplied by Truck Crane Service. Figure 3.1 shows one of the girders on a truck. Two girders were transported per day. Because of the location of the lab within the Civil and Mineral Engineering Building, and because of constraints imposed by the overhead crane (the weight of the girders was approximately forty thousand pounds each and the overhead crane in the Structural Engineering Laboratory had a capacity of thirty thousand pounds), an additional truck crane was required to lower the girders into the lab. Figure 3.2 shows a girder being lowered into the laboratory. The additional crane was also supplied by Truck Crane Service.

3.5 Removal of Remaining Composite Concrete

The girders were transported to the lab with the stool of the bridge deck still attached to the top flange. The stool covered the vertical No. 4 hairpins used to develop composite action between the girder and deck. The girders were to be tested individually with a new composite deck. For this reason, the stool was removed. A ninety pound jackhammer broke the cold joint between the top flange and the stool, and the remaining fragments within the hairpins were removed with a twelve pound sledge hammer.
3.6 Girders

3.6.1 Dimensions

The cross section of the girders was a standard AASHTO-PCI Type III. A cross section of the test girders is shown in Figure 3.3. The fabrication records reported the lengths of the girders to be within 3/8 inch of the design length of 64 feet 8 inches after transfer of the prestress.

3.6.2 Strand Pattern

Thirty 1/2 inch diameter seven-wire strands were used in the construction of each girder. The strands were placed in two groups. Twenty-two strands had a straight profile in the bottom flange of the section, while the remaining eight were draped with two hold-down points. The hold-down points were located five feet either side of the centerline of the girder. As shown in Figure 3.4, all strands in each group were placed on two inch centers both horizontally and vertically. The center of gravity of the twenty-two strand group was 3.82 inches above the bottom of the girder. The draped strand group was located 37 inches above the girder bottom at the ends; while at the hold down points, the distance was 5 inches. The minimum specified concrete cover for the strands was 2 inches. The strands in one of the girders (girder 3) were found to be located 1 inch lower than specified. This observation was noted while exposing girder 3 strands during the damaging procedure in the laboratory.

3.6.3 Section Properties

The section properties of the girders are listed in Table 3.1. The area and inertia were computed assuming a gross concrete cross section (the strand and mild steel were neglected).

As mentioned previously, a new composite slab was cast on each girder in the laboratory. The deck had a thickness of six inches, like the deck in bridge 27915, but instead of an 84 inch width, a narrower (64 inch wide deck) was chosen to accommodate lab constraints. The deck was cast in unshored plywood forms. Reinforcement similar to that of the original deck was used. This
consisted of two layers of reinforcement. The top layer had a clear cover of 1-1/2 inch, while the bottom had a clear cover of 1 inch. All deck reinforcement was Grade 60. See Figure 3.5 for a cross section of the deck. The longitudinal bars in the top layer were No.4’s while the longitudinal bars in the bottom layer were No.5’s. All transverse bars were No. 5’s placed on five inch centers.

No. 4 hairpins were used to transfer the horizontal shear between the deck and the girder. There were approximately 70 No. 4 hairpins located in each girder. The No. 4 bars were placed on 6 inch centers near the supports and on approximately 18 inch centers near midspan.

3.6.4 Girder Material Properties
3.6.4.1 Concrete
Coggins[33] tested the girders for concrete uniformity and compressive strength using four nondestructive test methods: rebound hammer, Windsor probe, pulse velocity, and break-off tests. The results of the nondestructive methods were correlated with those of two and four inch diameter cores. Coggins work is summarized in Table 3.2. The average compressive strength of the girders was estimated to be 8400 psi.

3.6.4.2 Mild Steel
To determine the stress-strain behavior of the mild steel used in the girders, five small sections of No.3 rebar were tested in a 30 kip Ametek material testing machine at the conclusion of the structural tests performed on girder 1. The No.3’s were used for confining steel in the bottom flange of the girder. A load cell internal to the testing machine was used to measure the applied tensile load while strain was measured with a 2 inch extensometer attached to the test bars. The bars exhibited a yield stress of 57 ksi and an ultimate stress of 93 ksi.

3.6.4.3 Prestressing Strand
Two seven foot sections of strand were removed from the end of the first girder at the conclusion of the structural tests. The
strands were anchored with standard 1/2 inch strand chucks which in turn were held by small frames which mated with the testing machine. A comparison of the stress-strain curves obtained from the test strand with that recommended by the PCI Handbook[34] for 250 ksi stress-relieved strand is shown in Figure 3.6. Loads were measured with the load cell incorporated in the testing machine. Strains were measured three different ways: (1) Two EA-060120-LE strain gages were applied to each strand. The strains were measured with a Vishay Strain Indicator Box reading the gages which were wired to a switch and balance box. (2) An LVDT with a 0.1 inch full-scale range was mounted on the strand and used as a two inch extensometer. The output of the LVDT was monitored with a multimeter. (3) The distance between the chuck grips was also measured. The displacement of the test machine actuator divided by the length of strand between the chucks gave an average strain measurement. All of the measured strains were approximately the same.

3.7 Deck Concrete Material Properties

As of 1987, the Minnesota Department of Transportation used a 3X46 concrete mix for the construction of new bridge decks. With the premise that the girders potentially could be reused, the new deck was constructed with the 3X46 mix. The 3X46 mix components are listed in Table 3.3. The concrete was delivered from Cemstone Concrete and placed with the use of a two yard concrete bucket. The concrete was then vibrated, screeed, floated, and troweled.

Cylinders were cast with the composite deck concrete of girders 1, 3, and 4. Four-inch diameter cores were taken from the composite deck on girder 2. The average compressive strengths of decks 1, 2, and 4 concrete was 5547 psi, 5064 psi, and 4053 psi, respectively. The data from deck 3 was lost. Split cylinder tests conducted on the concrete of deck 4 indicated a failure tensile stress of 452 psi.
3.8 Composite Section Properties

Section properties of the composite specimen, given in Table 3.4, were computed assuming nominal design compressive strengths. The nominal 3000 psi concrete deck was converted into an equivalent width of 5000 psi concrete for the computations of area and inertia.

3.9 Repair Materials
3.9.1 Internal Strand Splice Steel

The material used to fabricate the strand splice for girder 3 was 1045 steel. The 1045 steel has a nominal yield strength of 59 ksi and a nominal ultimate strength of 98 ksi. It was chosen because it had sufficient strength and still could be machined readily. The components of the splice fabricated from 1045 steel were the strand chuck mating pieces, the two threaded rods and the center turnbuckle. A schematic of the splice and a description of the individual components is provided in Chapter 5.

3.9.2 High-Strength Rods

The high-strength rods used to repair girder 4 were donated by Dywidag. The No.5 deformed prestressing bars had an area of 0.28 square inches, a specified ultimate stress of 157 ksi and an ultimate load capacity of 43.5 kips.

3.9.3 Hairpin Epoxy

The corbels which were attached to the bottom flange of girder 4, were designed with the shear friction method. The clamping force across the interface was delivered with No. 4 hairpin bars. Holes were drilled into the bottom flange of the girder. The holes were cleaned with compressed air and partially filled the two-part epoxy, HIT C-100. The hairpins were then inserted into the holes. Using a standard embedment length of 4.25 inches, a No. 4 rebar was specified to provide a working load of 2335 pounds in 2300 psi concrete. In order to estimate the ultimate capacity of one of the hairpins, a load test was conducted. A hairpin was inserted into a concrete block with a compressive strength of approximately 4 ksi. The pullout value
obtained from this concrete was a lower bound of the response of hairpins attached to the 8.4 ksi girder concrete. The hairpin pulled out of the block at a load of 13 kips. This value was used as the clamping force of a single hairpin in the design of the corbels.

3.9.4 Patch Concrete

The concrete mix used to patch the damaged regions of girders 3 and 4, was suggested by Elk River Concrete. Table 3.5 shows the breakdown of the components on a weight basis. Elk River Concrete reported a one day compressive strength of 4580 psi, a seven day strength of 7040 psi and a 28 day compressive strength of 8500 psi. The patch cylinders for specimen 3 had a strength of approximately 6500 psi. The patch concrete of specimen 4 had a strength of 6630 psi. The patch concrete mix was also used to construct the corbels cast on specimen 4 as part of the post-tensioned rod repair scheme.
4.1 Loading System

4.1.1 Support Blocks

Two pairs of support blocks were constructed. One was used to store the girders prior to testing and the second set served as end supports for the girders as they were tested. The blocks were reinforced with shrinkage and temperature steel, and blockouts were made to accommodate large end rotations of the girders during the ultimate load tests. The east support block resisted vertical forces by bearing on the strong floor, and resisted horizontal forces by bearing against the strong wall of the lab.

The west block carried its reactions to the strong floor in a different manner. Part of the west block was located over a testing machine cutout in the strong floor. Therefore, vertical reactions of the west block were carried by a combination of bearing on the strong floor and of bearing on a W12x120 steel shape which spanned the cutout. Horizontal forces were carried to the strong floor by one inch diameter rods which were tied to the top of the support block. The test girders contained steel bearing plates which were cast integrally with the section. Neoprene pads were placed between the support blocks and the test specimens. During the initial cracking static tests and the fatigue loading programs, a one inch thick pad was used. For the ultimate strength tests, a five inch thick pad reinforced with steel plates was used to accommodate the larger end rotations.

4.1.2 Loading Tubes

Loads were applied to the top of each of the test girder decks at two locations. A 60 inch long, 10 inch wide, 1 inch thick neoprene pad was placed on the deck at each location. A 60 inch long 10x4x1/2 inch steel loading tube with a one inch cover plate was placed on top of the pad. The actuator heads were bolted to the tube with 3/4 inch diameter bolts. The tubes were fabricated to allow attachment of either one or two actuators. At each load point, one actuator was
used for fatigue and cracking tests while two actuators were used to provide the necessary capacity during the ultimate strength tests. At approximately 500,000 cycle intervals during fatigue loading 1/4 inch plywood shims were placed under the tube and above the pad to change the mean position of the actuator. This was done to prevent creation of wear bands on the actuators. Figure 4.1 shows a schematic of the load frame cross section with one actuator, and a photo of the load frame setup with two actuators.

4.1.3 Actuator Tubes

The one or two actuators at each load point framed into a 16x20x1/2 inch steel actuator tube which spanned across the test specimen. The actuators were attached to the tubes with 3/4 inch diameter threaded rods. One inch thick cover plates were fillet welded to the end of the tubes to frame into the columns. To reduce the stress range on the end plate welds during fatigue loading, triangular stiffeners were welded between the end plates and the top and bottom faces of the tubes.

4.1.4 Columns

Four W12x120 columns transferred the applied actuator loads to the strong floor of the lab. The columns, twenty-five feet tall with 1-1/2 inch thick base plates, were fabricated with a 15/16 inch diameter hole pattern on four inch centers along the height of the column. The hole pattern was drilled on both flanges as well as the web. To orient the strong axis of the columns in the same plane as the actuator tubes rotation plates were used between the base plate of the column and the hole pattern of the strong floor. Three plates were used at each column. Two 1-3/4 inch nut plates sandwiched a 1-1/2 inch shear plate. Four 1-1/2 inch diameter threaded rods tied the columns to the floor. Two were used to tie the shear plate to the floor and two were used to tie the column to the shear plate. The 1-1/2 inch diameter rods were tightened with a hammer wrench and a twelve pound sledge hammer.
4.1.5 Bracing

Three types of bracing were used in the load frame. First, the actuator bodies were tied to the columns by W18x71 steel shapes (Actuator Restraining Beam) which framed into the columns by the means of large end plates. The flange of the W18x71 framed into an "Actuator Bracket" (u-shaped in horizontal plane) which in turn, tied into the actuator hole pattern with six 1-inch diameter bolts. Restraint was required because the actuators had spherical joints at both ends. Second, the columns at the two different load points were tied together by means of 6x6x1/2 inch tubes (Bracing Tubes) which framed into the column webs. Third, one inch diameter rods were used as cross-bracing to prevent twisting of the load frame. The "Cross-Bracing Rods" are identified in the photo of Figure 4.1.

4.1.6 Ultimate Test Stubs and Tie Rods

Because the specimens were capable of large deflections ( > 20 inches) during the ultimate load tests, and the maximum stroke of the actuators was limited to 12 inches, a method was devised to reposition the hydraulic actuators during testing. One foot tall column sections with end plates were used to accomplish this. The end plates were drilled to match the hole pattern of the loading frame columns.

After the actuators were extended their full stroke during a test, the test stubs (shown on the near frame in photo of Figure 4.1) were bolted to the column. Steel and wood cribbing was then driven between the loading tubes and test stubs. The actuators were slowly retracted to transfer the load to the test stubs. The actuators were then freed from the loading tubes and the whole actuator assembly (actuator tube, actuator restraining beam, and actuators) was lowered eight inches to a new set of holes. Two 7/8 inch diameter rods ("tie rods" in Fig. 4.1) prevented the columns from separating while the assembly was being lowered.
4.2 Test Equipment

4.2.1 Hydraulics

The hydraulic power for the actuators was supplied by a twenty-one gallon per minute MTS pump. The pump operated at a pressure of 3000 psi and utilized a water heat exchanger to keep the hydraulic fluid cool. The fluid was driven to an MTS accumulator through 1-1/2 inch hydraulic lines. From the accumulator, the fluid traveled to the hydraulic manifold. The manifold served as a switching station. Each of the actuators was connected to the manifold and its hydraulic power could be turned on or off there.

4.2.2 Computer

The computer system controlling the data acquisition and actuators was manufactured by Digital Equipment Corporation (DEC). Specifically, a MicroVax-II, with the VMS operating system was used. The test programs were written in Pascal and linked to libraries of MTS Toolkits. The MTS libraries contained procedures to control the various MTS hardware devices in the test processor and data acquisition system.

4.2.3 Data Acquisition Equipment

The MTS 468 test processor contained a number of devices. Four microsegment generator boards allowed command signals to be sent to the hydraulic actuators to prescribe a load or displacement path. A test processor clock was located on a separate board within the 468. Also located within the 468 test processor, were A/D boards (analog signal to digital signal converter boards) which collected instrument data. There were two different data acquisition devices: 32 channels of instrumentation were optionally conditioned by an MTS 450 conditioner, and 96 channels collected by an MTS 462 multiplexer. The first eight channels of the MTS 450 were dedicated to the load cells and LVDTs located within the actuators. The sample rate of this device was 1000 samples/sec. The MTS 462 multiplexer was set up to acquire data from strain gages configured as quarter Wheatstone bridge, constant current circuits. Data could be acquired at
approximately 10 samples/second with this device. Instruments located on the girder were connected to the data acquisition devices via quick-connect boxes. Instruments in the lab were wired with 9-pin connectors which plugged into the quick-connect boxes.

4.2.4 Software

Programs were written for pretest, in-test, and post-test processing of instrument data. Pretest programs verified that instrument signals were being received properly by the test processor. This was accomplished with printouts of the mean voltage, maximum voltage, minimum voltage and standard deviation for a device, for a specific number of samples at a prescribed sampling rate. Another program sampled data from the various devices and stored the mean voltages in a text file. The text files served two purposes. First, the initial voltage readings could be read into test programs as initial offsets. Subtracting the initial offsets from the instrument outputs, allowed net deflections to be computed and displayed readily. Second, the instrument data text files included a time stamp, and thus the change in an instrument versus time could be determined.

The main test program was flexible. The two main functions of the program were: (1) to collect the data from the various instruments, and (2) to control the behavior of the actuators via the use of the microsegment generators. In addition the program also performed the following functions:

1) data collected immediately sent to a binary file
2) wrote reduced data to a text file
3) on-line plotting of test parameters
4) hard copy printout of reduced data
5) automated data collection during a prescribed load or displacement increment
6) screen display of critical test parameters
7) ability to change data collection resolution while testing

The post-test programs also performed a variety of functions. Reduced data from the test program was reformatted to accommodate file retrieval by spreadsheet programs. Another program read data from a
specific file, channel, and index, and displayed the values on a screen. Data from specific test files could be reprinted. Another data reduction program allowed multiple files to be plotted to show deviation in specimen behavior.

4.2.5 Hydraulic Actuators/Controllers

The hydraulic actuators which applied the loads to the girders were manufactured by MTS. Each of the four actuators had a load capacity of 77 kips tension or compression and a total stroke range of 12 inches. The actuators were controlled by servo valves which could be controlled by either a load feedback signal or a displacement feedback signal. Each actuator had an MTS 406 controller. The controller contained conditioners for both the integral load cell and LVDT for each actuator. Interlocks contained within each controller, served as a safety feature, to shut off hydraulic power if an instrument voltage level exceeded a specified limit.

4.3 Instrumentation

4.3.1 Gross Specimen Behavior Instrumentation

In addition to the instruments within the actuators, a number of other instruments were utilized during testing. Two 1-inch LVDTs were placed vertically at each end of the girder to measure the displacement between the bottom of the girder and the top of the support block. One LVDT was placed on each side of the girder. This setup served to measure the average deflection into the bearing pad as well as any minor torsional rotation. Displacements were measured between the strong floor of the lab and the center of the bottom flange of the specimen at locations 140 inches and 260 inches away from centerline. Two LVDTs were utilized at centerline. One was placed on each side of the girder to measure deflection between the girder and the strong floor. In addition to measuring the centerline deflection, torsional rotation at centerline could also be monitored. The layout of LVDTs for the specimens is shown in Figure 4.2. In all, displacements were measured at 12 locations, and loads at 2 locations.
4.3.2 Prestress Loss Instrumentation

Prior to the initial cracking tests conducted on girders 1 and 4, Micromerasurement crack detection gages were placed on the bottom flange of each girder across the center ten feet of the span. The gages were wired in series and connected to a multimeter to monitor their cumulative resistance during the initial test. The presence of cracking was determined when the circuit broke. During the initial tests of girder 1, 0.1-inch full-scale LVDTs were placed across cracks marked during the first two cracking tests. The small LVDTs were used to measure average strains over a 3 inch gage length.

During the construction of the composite deck cast on girder 4, 75 steel strain gages were applied to the longitudinal reinforcement at the west quarter point. In addition to the steel gages, 8 concrete gages were positioned in the deck concrete near the steel gages. These strain gages were used to monitor shrinkage strains in the new composite deck. The layout of the strain gages installed in deck 4 is shown in Figure 4.3.

4.3.2 Strand Strain Gages

Four strands exposed during the damaging procedures performed on girders 2-4, were instrumented with strain gages. Micro Measurements EA-060-120-LE gages were installed on individual wires of strands. Six gages were installed in girder 2, twenty gages in girder 3, and eight gages were used in girder 4.

4.3.3 Turnbuckle Load Cells

Micro Measurements EA-060-120-LE gages were also used to create turnbuckle load cells used to repair girder 3 (Section 5.2). Four gages were installed on the 1045 rod stock to create full bridge circuits. The gages were oriented to cancel bending effects.

4.3.4 Post-tensioning Rod Gages

Two Micro Measurements EA-060-120-LE gages were installed on each of the two post-tensioning rods used to repair girder 4. The gages
were installed midway between the corbels and on opposite sides of each bar.
Chapter 5 - Background of Repairs

5.1 Introduction

Two damage repair schemes were carried out for this project. An internal strand splice method was used to repair girder 3, and an external post-tensioning method was utilized on girder 4. The repair schemes were among the most promising presented in NCHRP 226 (1980), and later tested (1985), on a symmetrically damaged girder by Shanafelt and Horn for NCHRP 280. Shanafelt and Horn conducted 10 static load tests. However, because all the tests were conducted on a single girder, they were unable to load each repair scheme to failure. For each of the tests they loaded the girder to 90 percent of the calculated capacity and then unloaded.

The research conducted at the University of Minnesota extended the work of Shanafelt and Horn in two ways. First, the damage introduced and subsequently repaired in the specimen was non-symmetric. This being a more realistic representation of actual damage occurring in prestressed concrete bridges from overheight vehicles. Second, each repair method was monitored through a complete life series of tests (installation, static, fatigue, and ultimate). This allowed weaknesses at any of the different phases to be investigated.

Neither repair technique was ideal. Both introduced discontinuities into the cross section of the girder. The turnbuckle splice had a much larger stiffness (axial and flexural) compared to the strand. The external post-tensioning method while not introducing as large an axial stiffness difference with the post-tensioning rods, introduced significant changes in girder cross section at corbel locations.

The following sections describe the details of the two repair schemes implemented for this study. For each repair, the internal strand splice and the external post-tensioning, the background and implementation as provided by the NCHRP studies are presented. The performance of the internal splice is described in Chapter 8 with the tests performed on girder 3. The performance of the external post-
tensioning repair is described in Chapter 9 with the response of girder 4.

5.2 Turnbuckle Splice

Sections 5.2.1-5.2.3 summarize the previous research on the turnbuckle details as provided in NCHRP Reports 226 and 228.

5.2.1 Background

NCHRP 226 provided a design splice to repair a 0.5 inch diameter 270 ksi seven-wire strand. The design required two strand grips (chucks) to be incorporated into a turnbuckle. The strand grips allow the turnbuckle to reintroduce force into the severed strand. Inherent to the repair scheme are two advantages. First, is the ability to restore prestress locally and internally. Second, a variable number of strands can be repaired with the splices. The repair uses 1 inch diameter ASTM A-722 Grade 150 rods as part of the turnbuckle. A schematic of the detail presented in NCHRP 226 is shown in Figure 5.1.

5.2.2 Implementation

NCHRP 280 presented results of a monotonically loaded girder which had four 0.5 inch diameter strands repaired with the internal strand splice technique. The specimen used for the tests was an AASHTO Type III girder on a 60 foot span. The girder was pretensioned with 16 0.5 inch diameter 270 ksi strands. A 90 inch wide, 6.5 inch thick composite deck was cast on the girder. Figure 5.2 shows the detail which was implemented.

Shanafelt and Horn reported the following construction procedure:

1. Determine preload requirements. If stresses permit, it is preferable to apply preload after stressing the splices.
2. Assemble splice, locating splice sleeves and strand grips to allow seating of the strand grips and sufficient thread length in the splice sleeves.
3. Torque lubricated splice sleeve to approximately 22,000 lb (the working strength of one strand). The strand grips must be prevented from rotating during torquing.
4. Repeat steps 2 and 3 for other severed strands.
5. Apply preload.
7. After concrete repair has gained required strength, remove preload.

5.2.3 NCHRP 280 - Results

Shanafelt and Horn give little information as to how the internal strand splice itself performed. They provide a plot indicating that the load versus deflection response returned to coincide with that of an undamaged, cracked girder. They mention strand stress levels (two instrumented strands) exceeded yield before the full 90 percent loading was applied to the girder.

The repair was inexpensive, and quick and easy to install. They reported 2.5 hours to install the repair. An indirect method was used to measure the tension in the turnbuckle. Prior to installation, the splice was tensioned in a test machine to calibrate the number of foot-lbs torque equal to various levels of axial load. Hence, load was never measured directly within the repair, nor monitored after the repair was in place.

5.2.4 Minnesota Repair

5.2.4.1 Description

The repair described in NCHRP reports 226 and 280 was changed slightly before implementation in girder 3. Whereas Shanafelt and Horn’s repair indirectly measured initial load, a load cell was incorporated into the turnbuckles used in girder 3. Figure 5.3 shows the layout of a single repaired strand. The pieces used in the repair were: two strand chucks, two chuck couplers, one load cell rod, one right hand/left hand threaded rod, and one turnbuckle coupler.

Other changes include: using modified standard anchor chucks as opposed to Supreme Splice Chucks, and using 1045 steel instead of ASTM
A-722 or A-322 steel for the turnbuckle. Shanafelt and Horn's minimum splice length was approximately 2.5 feet. The incorporation of a load cell into the turnbuckle increased the length of the repair to over four feet. Instead of using coarse threads and a 4.3 foot torque wrench, fine threads (18 threads per inch) were used throughout the turnbuckle assembly. The smaller pitch to the fine threads decreased the torque required to tension the splice. Six standard chucks were donated by a local precast fabricator. In addition to the chucks, six feet of 1.25 inch diameter, and six feet of 2.0 inch diameter 1045 stock were used to fabricate three splices. Details of the splice are included in Appendix A.5. Two of the three splices fabricated were intended to be used to repair strands. The last splice severed as a backup, and if needed, component testing could be performed.

5.2.4.2 Design of Repair

Each splice was designed to repair a 0.5 inch diameter 250 ksi strand. The design procedure given in the NCHRP reports was followed. In addition to changing the detail to fine threads, the tensioning rods were mated to pieces which went around the outside diameter of the strand chucks. This change was made to confine the chuck barrel. Instead of a hexagonal splice fitting in the turnbuckle, flats were machined into the round stock to allow placement of pipe or crescent wrenches. One of the threaded rod pieces was instrumented to produce an axial load cell. The load cells were calibrated by testing them inside an MTS testing machine with an NBS traceable load cell. The output from the load cells produced data with linear regression, \( R^2 \) squared coefficients greater than 0.9998.

5.2.4.3 Repair Procedure

After the splices had been fabricated, and the tests conducted at the various stages of damage (no concrete, 1 strand cut, 2 strands cut), the repairs were installed. The severed strands were trimmed to accommodate the splice. After the opening in the strand had been cut to the proper length, the chucks were installed on both strand ends. The turnbuckle was shortened to its minimum length and then threaded
onto one of the strand chucks. It was important during the installation to avoid sliding the chuck further onto the strand, and opening up too long a repair space. The retaining spring within the chucks prevented release of the wedges unless the chucks were completely disassembled. With the turnbuckle fully installed on the left chuck, the turnbuckle was extended by opening the right hand threads. At the same time, the turnbuckle was threaded onto the right chuck. Once the threads had started on the right chuck, the pieces were manipulated until each had sufficient threads engaged, the system was symmetric, and there was plenty of stroke to tension the turnbuckle.

After the pieces were set in the proper position, the load cell was wired to the data acquisition system. A short program was written to monitor the load in the turnbuckles as they were tightened and to collect information from the strain gages installed on the exposed strands. Two pipe wrenches and a large crescent wrench were used to tension the splice. The load in the repair was monitored during the tensioning procedure. Each splice was loaded to approximately 18 kips.

5.2.4.4 Installation Figure

Figure 5.4 shows the influence of strand severing and strand repair on the adjacent instrumented strands 3 and 4. The left vertical axis shows the change in strand stress while the right vertical axis shows the change in splice load. Readings were taken at one second intervals. During the first 200 readings strand 1 is being severed. The severing of strand 1 produced approximately a 0.5 ksi increase in stress. The next 100 readings show the effect of severing strand 2. This increased the stress in strands 3 and 4 an additional 1 ksi. The repair sequence begins near reading 350. As one would expect, as the turnbuckles are installed and tensioned, the stress in the undamaged strands returns to their level before strand severing.

The retensioning of splice one is shown beginning near reading 500. Retensioning was required due to losses which occurred in the splice after static testing. During the static test, additional load
was induced in the repair. The additional load increased the seating loss of the strand chucks. As a result, after the static tests, the load in the turnbuckle under dead load had dropped from over 18 kips to 13 kips.

Initial installation of splice 2 began near reading 540. Before the proper load could be attained, the threads of the splice locked up because the threads were cleaned but not lubricated prior to installation. The splice was removed by torching and replaced by the remaining splice.

5.3 Post-tensioned Repair

As for the turnbuckle splice technique, previous research on the post-tensioned repair described in NCHRP studies 226 and 280 will be presented (Sections 5.3.1-5.3.3).

5.3.1 Background

The post-tensioned repair described in NCHRP 226 consisted of two components. The first consisted of corbels attached to the bottom flange of the damaged girder. The corbels were cast beyond the damaged region a sufficient length to allow the severed strands to redevelop their effective stress into the undamaged regions of the girder. The second component consisted of high strength rods tensioned between the corbels. The rods restored the lost prestress to the cross section. Because the rods restored prestress at a point different from where the strands were severed, an indirect repair resulted. The prestress in the cross section at the location of the corbels was larger than before damage, while the prestress at the damaged region was similar to the level before damage.

Figure 5.5, shows the details Shanafelt and Horn provided in NCHRP 226. The rough computations are given in Figure 5.6. Calculations are given for prestress loss due to damage, prestress gain from the repair, ultimate strength, and corbel reinforcing.
5.3.2 Implementation

In the implementation of the NCHRP 226 post-tensioned repair recommendations (NCHRP 280), Shanafelt and Horn used three different corbel designs for attachment to the girder. The NCHRP 280 tests were conducted using a single monotonically loaded girder for each test.

The first two designs involved the use of 0.5 inch anchor bolts with embedments of 1.5 inches for corbel 1 and 2.5 inches for corbel 2. They report that the use of anchor bolts with a short embedment length was not practical. The corbels used for the load tests on the girder were attached with No. 4 hairpins epoxied in holes with an embedment length of 6 inches. Four severed strands, two on each side of the girder (symmetric damage), were repaired with post-tensioning. The corbels transferred the load from a 1 inch diameter, Grade 150 threaded bar. Each rod repaired two strands and was tensioned to 84 kips. The details of the repair used for the test are given in Figure 5.7.

5.3.3 Results - NCHRP 280

Shanafelt and Horn reported little on the performance of the post-tensioned repair. They mention that the repair increased the stiffness of the test girder during the static load test. They also mentioned that two of their corbel designs were satisfactory and that overall the repair scheme works.

5.3.4 Minnesota Repair
5.3.4.1 Description

With experience from the internal strand splice, minor modifications were made to the post-tensioned repair described in NCHRP reports 226 and 280. Knowing the influence of axial stiffness on strand stresses in girder 3, the post-tensioning steel was chosen with the smallest practical area. This resulted in two 0.625 inch diameter, 157 ksi rods used to repair two 0.5 inch diameter 250 ksi strands. In addition to having the minimum axial stiffness, the repair with two rods provided redundancy. The repair would not completely fail if one rod fractured. Lastly, because corbels were
cast on only one side, no through-web reinforcement was used. The lack of this reinforcement led to the debonding of the corbel (loss of the repair) during the ultimate strength test. The layout of the repair is shown in Figure 5.8.

5.3.4.2 Design of Repair

Two 5/8 inch diameter 157 ksi high-strength rods were used to repair the girder. The design calculations are included in Appendix A.5. The corbels were designed to be attached to the girder with the shear friction concept. The clamping steel for the shear friction consisted of No. 4, grade 60 hairpins. The hairpins were epoxied into 4.25 inch deep holes in the girder. A two part epoxy was used to anchor the hairpins.

To estimate the pullout capacity of the hairpins, a pullout test was performed. Sample holes were drilled into an unused concrete block of 4 ksi concrete. A hairpin was epoxied into 4.25 inch deep 0.625 inch diameter holes. The next day the hairpin was loaded to failure with a crane. This allowed the hairpin to be pulled out of a zone where the concrete was in tension. The failure mode of the hairpin was a tension cone around one hairpin leg. The load at failure was 13 kips. The girder concrete strength was greater than 8 ksi, hence the corbel design was based on a hairpin clamping capacity of 13 kips. In addition to the hairpins, No. 3 reinforcing bars were used as confining steel and also as longitudinal reinforcement. The complete list of details, used for the corbels is provided in Appendix A.5.

An Enerpac center-pull jack was used to tension the two post-tensioning rods through ducts in the corbels. The load was measured with both the pressure gage on the pump, and from the output of two strain gages installed on each rod. The output from the two gages on each of the rods was averaged to give the net axial load. Because the strain gages were installed on opposing sides of the rod, bending effects were removed from the axial load measurement.
5.3.4.3 Repair Procedure
The ducts were grouted after the rods were tensioned. The remainder of the girder 4 repair proceeded similarly to that of girder 3. After the grout had cured, static tests and stress ranges were collected. A preload was then applied to decompress the bottom flange, and the patch concrete placed. The patch was allowed to cure and then the preload removed. After which, static tests and stress ranges were collected.

5.3.4.4 Installation Figure
Figure 5.9 shows a plot of the tensioning of the rods. One of the hydraulic fittings attached to the jack had a leak. The jack was unable to develop a force larger than 5 kips. Once the fitting was replaced, the tensioning procedure went smoothly. The bottom rod was tensioned first. The top rod was tensioned and then the bottom rod retensioned. The final load in both rods was 25 kips.

The performance of the repairs after installation is described later. The turnbuckle splice is described in Chapter 8 with girder 3 and the external post-tensioned splice is described with girder 4 in Chapter 9.
Chapter 6 - Testing of Girder 1

6.1 Introduction

Girder 1 served as the overall control specimen. It was used to determine the response of an undamaged girder to both static and fatigue loadings. No impact damage or repair methods were investigated. Initial static tests were performed to determine the effective prestress in the strands. Initially, the girder was statically loaded to produce cracks and later reloaded to reopen the cracks. Following the initial tests, a fatigue loading program was imposed, after which, the girder was loaded to failure.

Before, during, and after the fatigue loading program, static tests were performed to monitor degradation in stiffness caused by the fatigue program. Table 6.1 lists the seventeen static tests which were performed on girder 1. For each test, a label, the cumulative number of fatigue cycles, and comments are listed. All tests were performed on the first girder, hence the label prefix G1. The label suffixes C1-C4, F1-F12, and U1 correspond to the three different phases of load testing: cracking, fatigue, and ultimate.

6.2 Initial Static Tests - Girder 1

Static tests G1C1 and G1C2 are shown in Figure 6.1. The figure shows that the girder began to soften with approximately 130 kips of live load during G1C1. Subsequently, during G1C2, more cracking was introduced into the girder as the live load was increased above 140 kips. The prestress loss measurements made during the four initial static tests indicated the effective stress in the strands to be approximately 130 ksi. The 130 ksi value was obtained by back figuring from crack reopening loads. Subsequent data obtained from strand instrumentation on girders 2 through 4 indicated the effective prestress to be 110 ksi. The prestress loss measurements and calculations are described in Chapter 12.
6.3 Fatigue Loading Program - Girder 1

The AASHTO Code allowable tensile stress for prestressed concrete has changed with time. Prestressed concrete was initially included in the AASHTO design code in 1962, with a maximum tensile stress of decompression (0\(\sqrt{f'_c}\)) under service loads. Later in 1965, the allowable stress was increased to a nominal value of 3\(\sqrt{f'_c}\). In 1971, the allowable stress was increased to 6\(\sqrt{f'_c}\), the current allowable. Figure 6.2 shows the variation of allowable bottom fiber tensile stress with date of the AASHTO code. The allowable stress is based on an uncracked cross section.

After an estimate of strand effective stress had been obtained, computations were made to determine nominal stresses. Nominal stresses for the test girders were calculated for various levels of applied load. The girders were designed with a nominal concrete compressive strength of 5000 psi. Concrete cores taken from the girder indicated an in situ compressive strength of 8400 psi. For the fatigue program, loadings were determined from the nominal concrete strength, 5000 psi, and conducted at four different levels. The effect of using the nominal concrete strength to determine applied load levels reduced the actual nominal stresses by \(\sqrt{5000}/\sqrt{8400}\), or 0.77. The different fatigue loading levels are described in the following chart.

<table>
<thead>
<tr>
<th>bottom fiber tens. stress</th>
<th>bottom fiber tens. stress</th>
<th>range</th>
<th>minimum moment (kip-in.)</th>
<th>maximum moment (kip-in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(* (\sqrt{f'_c}) <em>nom</em>)</td>
<td>(* (\sqrt{f'_c}) <em>exp</em>)</td>
<td>0</td>
<td>7,430</td>
<td>20,500</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>1</td>
<td>7,430</td>
<td>20,500</td>
</tr>
<tr>
<td>3</td>
<td>2.35</td>
<td>2</td>
<td>7,430</td>
<td>22,500</td>
</tr>
<tr>
<td>6</td>
<td>4.7</td>
<td>3</td>
<td>7,430</td>
<td>24,500</td>
</tr>
<tr>
<td>12</td>
<td>9.3</td>
<td>4</td>
<td>7,430</td>
<td>28,500</td>
</tr>
</tbody>
</table>

The first two columns in the table represent the maximum bottom fiber tensile stresses imposed on the girders during the different levels of fatigue loading. The first column is based on the nominal concrete compressive strength of 5000 psi and the second column is based on the experimentally determined concrete strength of 8400 psi. For both
cases, the effective strand stress was assumed to be 130 ksi under dead load. The four rows in the table correspond with the four different "ranges" of fatigue loading used in the tests. The actual minimum and maximum moments imposed on the girders for each fatigue range are listed. The moments were computed at centerline and included dead load bending moments (6160 kip-inches).

Fatigue loading for girder 1, began by applying range 1 loading. The girders were designed with an allowable service stress of decompression. During later portions of the fatigue program the load was increased to produce range 2, range 3, and range 4 load levels.

Figure 6.3 shows the live load peak versus the cumulative cycles for the loading program. Live load peak being the sum of the loads applied by both actuators. The initial phase of fatigue loading consisted of 400,000 cycles of range 1 loading. The assumed effective prestress (130 ksi) and cross section of the composite girder resulted in a live load peak of 89 kips to decompress the girder at centerline.

Throughout the fatigue loading a small seat load of 4 kips was used on each actuator. This seat load was used for each of the loading phases for all four girders. Following the range 1 loading, 1,000,000 cycles of range 2, and 1,300,000 cycles of range 3 loading were applied. The total peak live load for the range 2 and range 3 loadings was 101 and 114 kips, respectively. Later, approximately 60,000 cycles were applied at range 4. The range 4 loading was performed to determine the effect of overloads on the response of the test girder. The peak load for the range 4 load level was 139 kips.

Figure 6.4 shows the actuator deflection span versus cumulative cycles for the fatigue loading of girder 1. The actuator span plot is not as smooth as the loading plot. One reason for the plot not being as smooth is because the tests were conducted under load control. Similar to the intermediate static tests (Section 6.4), the actuator span versus cycles plot demonstrates girder softening. Girder softening is demonstrated by the actuator span increasing under constant applied load.

Figure 6.4 shows that the actuator span was stable for decompression loading. Only small amounts of actuator span increase
were noted during the range 2 and range 3 loading levels. This was attributed to cracks propagating upward in the girder under fatigue loading until they reached a stable position. However, at the range 4 loading level, the actuator span did not stabilize, rather the actuator span increased with additional cycling.

6.4 Intermediate Static Tests – Girder 1

To determine the effects of the fatigue program on girder 1, intermediate static tests were performed. Most of the intermediate static tests were conducted by applying a total live load of 128 kips. This value was chosen because it was a modest amount above the 114 kips required to produce range 3 fatigue loading. However, before and after the range 4 fatigue loadings, static tests were performed to 144 kips (slightly greater than the range 4 peak load of 139 kips). Five intermediate tests are plotted in Figure 6.5. G1C3, G1F1, G1F10, G1F11, and G1F12 are shown. The entire fatigue program through the range 3 load level (2.7 million cycles), caused less than a 0.1 inch or L/7760 increase in deflection over that measured at 128 kips live load. While after 60,000 cycles at range 4, the deflection increased by approximately 0.14 inches or L/5543 over that measured at 144 kips live load.

Overall, the girder performed well during the fatigue loading program. Theoretical stress ranges were obtained with 2D_SECT, a plane section program. Experimental material properties were used in the analysis.

The following chart shows the computed stress ranges and estimated fatigue lives for the girder. Shown are data for both 130 and 110 ksi effective prestress, for each of the four loading levels. The stress ranges are in ksi and the fatigue lives are in millions of cycles. The stress ranges were obtained with the program 2D_SECT which is described in Chapter 11. The fatigue lives were estimated with Paulson's equation[15].
Effective stress = 130 ksi

<table>
<thead>
<tr>
<th>loading level</th>
<th>stress range (ksi)</th>
<th>fatigue life (* 1 million)</th>
</tr>
</thead>
<tbody>
<tr>
<td>range 1</td>
<td>8</td>
<td>69.5</td>
</tr>
<tr>
<td>range 2</td>
<td>9.7</td>
<td>35.2</td>
</tr>
<tr>
<td>range 3</td>
<td>14.4</td>
<td>8.8</td>
</tr>
<tr>
<td>range 4</td>
<td>31.4</td>
<td>0.57</td>
</tr>
</tbody>
</table>

Effective stress = 110 ksi

<table>
<thead>
<tr>
<th>loading level</th>
<th>stress range (ksi)</th>
<th>fatigue life (* 1 million)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10.7</td>
<td>25.2</td>
</tr>
<tr>
<td></td>
<td>17.9</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>27.6</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>47.8</td>
<td>0.13</td>
</tr>
</tbody>
</table>

The fatigue program was terminated at the range 4 level after 60,000 cycles. During this loading, the mean position of the actuators did not stabilize. Increasing set was observed during the loading. The loading was intended to determine the response of the girder to a modest number of overloads.

6.5 Ultimate Strength Test - Girder 1

Satisfied with the fatigue behavior of girder 1, the ultimate strength test (G1U1) was performed. Three changes in the test setup were made prior to G1U1. First, five-inch neoprene pads were placed under the girder to accommodate large end rotations. Second, four hydraulic actuators were placed within the loading frame to achieve the expected loads. And third, the LVDTs were recalibrated to measure large deflections. The test was videotaped and cracks were marked at approximately one inch deflection increments.

The load versus deflection plot of G1U1 is shown in Figure 6.6. The initial and intermediate static load tests were plotted with applied live load versus net centerline deflection. Net centerline deflection was computed by subtracting the average displacement of the girder into the neoprene pads from the displacement measured with the LVDTs located at centerline. However, for the test to failure the load versus deflection plot displays total live load versus average actuator deflection. A comparison of the net centerline deflection and the average jack deflection is shown in Figure 6.7. While the jacks or actuators were located five feet away from centerline, they must also compress the neoprene pads as they apply load to the girder. The net result is that the deflections determined from each method
varied by a small amount, (less than 0.2 inch for the first 8 inches).

Because the actuator stroke was limited to 12 inches, girder 1 had to be tied off twice while the actuator frames were lowered to develop enough deflection to reach failure. The two points at which the actuators were moved are evident in Figure 6.6. The first is at a deflection of 9.5 inches and second at 18 inches. The load dropped at each location. Lowering of the actuator frames required approximately four hours of labor. The first move was conducted during the afternoon of the first day of the test. Loading began again that evening. The second move took place the morning of the second day. As a result, a displacement of 18 inches was left on the girder overnight. Overnight the live load on the girder reduced from 288 to 262 kips. Testing resumed the afternoon of the second day.

The maximum applied live load was 293 kips and the deflection imposed on girder 1 before failure was 21.5 inches. The test ended when the composite deck failed in compression. The deck concrete crushed, the longitudinal deck reinforcement buckled upward, and the compression block moved down into the top flange of the girder. The top flange was unable to resist the compressive force and also failed in compression. After the test, it was observed that six inches (longitudinally) of the composite deck had crushed and a two foot diameter hole was left in the middle of the girder web.

The total live load of 293 kips produced a moment at centerline of 53,110 kip-inches assuming a simply supported girder. However, because the girder was supported on five inch neoprene pads, rather than frictionless bearings, the girder was partially restrained horizontally. This reduced the net moment at centerline while the neutral axis of the girder was above the elevation of the pads. As the deflections imposed on the girder became larger, the horizontal reactions became larger, but the lever arm became smaller as well. The magnitude of the horizontal reactions was not determined.

The flexural capacity of the section was computed manually three different ways. First, the method described in AASHTO Section 9.17[35] was followed. This resulted in a flexural capacity of 47,100 kip-inches. Second, the ACI[36]/PCI[37] method was
used. A capacity of 47,140 kip-inches was determined with the ACI procedure. Lastly, simply assuming all the strands yielded (250 ksi), resulted in a flexural capacity of 48,800 kip-inches. The calculations for each of the methods are given in Appendix A.6.

6.5.1 Ultimate Test Crack Patterns - Girder 1

Cracks were marked as the girder was loaded to actuator deflections of 2, 2.75, 3.2, 3.6, 4.1, 4.6, 5.3, 5.9, 6.7, 7.7, 9.1, 12, 14, 16, 18, and 21 inches. The cracks which had developed at three of the deflection stages are shown in Figure 6.8. The top crack pattern was obtained at a deflection of 5.9 inches. The total live load on the girder at 5.9 inches was 238 kips (this live load was split approximately equally between the two load points). The middle crack pattern shows cracks present at a deflection of 12 inches and a live load of 269 kips. The bottom crack pattern shows the cracks at 21 inches of actuator deflection and a peak load of 293 kips.

The crack patterns demonstrate that over half of the girder was significantly cracked before failure. Cracking propagated high into the web at relatively low deflection levels. Lastly, as one would expect, the top of the inclined cracks in the shear spans pointed in the direction of increasing bending moment.
Chapter 7 - Testing of Girder 2

7.1 Introduction - Girder 2

Girder 2 was used to investigate an incrementally damaged specimen. After a new composite deck was cast, the girder was statically loaded to produce cracks. The cracks were marked and a predamage fatigue program conducted. The fatigue loading was performed to ensure that the initial behavior of girder 2 was similar to that of girder 1. Subsequently, a damaging scheme was implemented to incrementally sever strands. The strands severed represented those which were most susceptible to overheight vehicle impact damage. Exposed strands were instrumented with strain gages during the damaging procedure. A constant load (range 3 - see Section 6.3), fatigue program was applied through the various damage stages. In all, 3.2 million fatigue load cycles were applied. Similar to the tests conducted on girder 1, static tests were performed periodically to monitor response. The response of the damaged girder was found to be unstable with four strands cut. The girder was then loaded to failure. Strain gage output was recorded during static tests, fatigue loading, and strand severing.

7.2 Initial Static Tests - Girder 2

During the initial static test, girder 2 was loaded to 128 kips. This was 16 kips less than the initial loading of girder 1. The peak load level was chosen to be consistent with the intermediate static tests performed on girder 1 which were conducted with a peak load of 128 kips. As a result, the initial cracking of girder 2 was less than that of girder 1. The results of the initial static test are shown in Figure 7.1. The overall load-deflection behavior of the element was used as an indicator of prestress losses. No special crack instrumentation was used to measure the effective prestress. The initial static test of girder 2 was essentially the same as girder 1 up to 128 kips. A comparison of the response of the initial static tests for both girders is shown in Figure 7.2. Because the initial
response was similar, the prestress losses were assumed to be of the same magnitude.

7.3 Damage Inducing Procedure - Girder 2

The strategy used during the damaging procedure was to incrementally sever pairs of strands. Figure 7.3 shows the strand pattern of the girder near centerline, also shown are three stages of damage. In the first damage stage, four strands were exposed by removing the concrete encasing them. During the second stage, two of the four strands were severed, and during the third stage, the remaining exposed strands were severed. The top of Figure 7.3 shows the labeling of the exposed strands. They are identified for later data presentation.

With the concrete removed, two strain gages were installed on each of the four strands. The fatigue loading program consisted of 500,000 cycles of range 3 loading. After 500,000 cycles, a static test was performed. More damage was introduced into the section, another static test performed, and fatigue loading resumed. This process was continued until the response of the girder became unstable. Unstable was defined as increasing permanent deformation in the girder, or fatigue fractures of prestressing steel. The entire test sequence is shown in Table 7.1.

7.4 Fatigue Loading Program - Girder 2

Fatigue loading for girder 2 was performed at two different maximum load levels. Initially, the undamaged girder was subjected to 500,000 cycles of range 2 loading. Range 2 had a minimum live load of 8 kips and a maximum of 101 kips. Under this loading, the bending moment at centerline (including dead load) varied between 7,430 and 22,500 kip-inches. Range 2 loading was followed with 1,500,000 cycles of range 3 loading. Range 3 had a live load varying between 8 and 114 kips and a moment field varying between 7,430 and 24,500 kip-inches. Girder 2 withstood the predamage fatigue program without visible distress. Convinced that the girder was performing similarly to
girder 1, the damaging procedure began. All fatigue loadings on the damaged girder were conducted at range 3.

In all, over three million fatigue cycles were applied, 0.5 million at range 2, and 2.7 million at range 3. Actuator load and deflection spans were checked daily. The total live load peak versus cumulative cycles plot is shown in Figure 7.4. Similar to girder 1, the fatigue loading was conducted under load control. As a result, the loading plot shown in Figure 7.4 is stable.

Figure 7.5 shows a plot of actuator deflection span versus cumulative cycles. Similar to girder 1, the deflection plot is not as stable as the loading plot. Before damage, the actuator span is slowly increasing as the fatigue program continues. The increase is slightly more than what was demonstrated by girder 1. This was attributed to conducting the initial static tests for girder 2 at a lower load. During the initial static tests less damage was introduced into the element. Hence, during the fatigue program more damage was required to attain cracking similar to that of girder 1.

The fatigue loadings between damage levels were the following: The girder was cycled 500,000 cycles with the concrete removed before strands were severed. Fatigue loading resumed after strands 1 and 2 were severed. Approximately 300,000 cycles into the fatigue loading with strands 1 and 2 severed, strand 3 broke. After strand 3 broke, static tests were performed, and fatigue loading resumed. After an additional 200,000 cycles, loading was stopped, and static tests performed. Strand 4 was then severed, a static test performed and fatigue loading restarted.

As one would expect, as more damage was introduced, the softer the girder became. The softening is evident by the jumps in the actuator deflection span at each damage level. Only 17,600 cycles were applied to the girder after the fourth strand was severed. During this time the actuator deflection span increased 18 percent. Following this large softening, the fatigue loading was terminated.
7.5 Intermediate Static Tests - Girder 2

Similar to girder 1, static load tests were conducted throughout the fatigue loading program. Each test was conducted with a peak load of 128 kips. The 128 kip peak load level was retained to be consistent with the tests performed on girder 1. For the initial tests, the net centerline deflection was less than 1 inch. As the fatigue program progressed and as more damage was introduced into the girder, the girder softened. After the fourth strand was severed and the fatigue program finished, the peak deflection during a static test was 1.7 inches.

Figure 7.6 shows a plot of the predamage static tests. The plot shows tests beginning with the initial static tests and ending just before the damaging procedure. The figure clearly demonstrates that the girder significantly softened without loss of section or strands.

The effect of damage and a fatigue program on the damaged girder are shown in Figure 7.7. This figure shows that, as one would expect, loss of section and loss of strands softened the element. Figure 7.7 shows the softening as a result of fatigue loading. Shown in the figure are static tests before damage, with concrete removed, 2 strands cut, 3rd strand broke, 4th strand cut and after cycling had finished at the 4th strand cut damage level.

A comparison plot between the response of girder 1 and girder 2 is shown in Figure 7.8. In this figure, the response of girder 1 at the end of its range 3 fatigue loading is compared with the response of girder 2 before the damaging sequence. Girder 1 had been subjected to 2.7 million fatigue cycles and girder 2 million when these tests were conducted. This figure shows that the net deflections differed by less than 0.1 inch.

7.6 Strain Gages - Girder 2

Two Micro Measurements EA-060120-LE strain gages were installed on each of the four exposed strands. Each strain gage was installed on an individual wire of the strand. The strain gages were attached to a switch and balance box which in turn was connected to a Vishay Strain Indicator Box.
The left side of Figure 7.9 shows two strain gages partially installed. The right side shows two gages fully installed and protected.

7.6.1 Static Test Data - Girder 2

The response of the strain gages during the static test following their installation is shown in Figure 7.10. As expected, the bottom layer of strands picked up stress at a faster rate than the second layer. This was due to the bottom layer of strands having a larger eccentricity from the neutral axis. Because the strands had been tested earlier and found to closely follow the PCI handbook equation for 250 ksi strands, stresses were obtained by multiplying the strains by 28,000 ksi (stresses were in the elastic range < 225 ksi).

7.6.2 Stress Ranges - Girder 2

Strain ranges were collected from the strain gages installed on the exposed strands. An XY recorder was connected to the strain indicator. The output of the strain gages was then plotted during range 3 fatigue loading. From the peaks on the xy recorder plot, the strain range was determined. Again, stress ranges were obtained by multiplying the strain range by 28,000 ksi.

Figure 7.11 shows a plot of the stress ranges obtained from the exposed strands. Initially, all four strands exhibited a stress range near 25 ksi. Using the Paulson strand fatigue model, this stress range had an estimated fatigue life of 1,280,000 cycles. After the third strand had broken, the stress range in strand 4 had increased to over 40 ksi. The Paulson equation, given in Section 2.2.3.2 [15], predicted a fatigue life of 247,100 cycles for a stress range of 40 ksi. With 3 strands severed (10%), the expected fatigue life of the girder had decreased by over 80 percent. A modest amount of damage resulted in a drastic reduction in predicted fatigue life. After an additional 200,000 cycles of loading, strand 4 was severed in keeping with the planned test program. Because the strand was severed before reaching the predicted fatigue life, we can only say the Paulson equation was not grossly unconservative.

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Two items are interacting to produce the dramatic reduction in fatigue life. First, is the nature of the fatigue life curve. The curve is logarithmic. This results in approximately a 90 percent reduction in fatigue life for a doubling in the stress range. Second, the range 3 loading was significantly above the decompression load of the girders. Because the loading was above decompression, strand stress ranges were sensitive to the amount of prestress on the cross section. A small reduction in prestress force, resulted in a relatively large change in stress range (from 25 to 40 ksi). The interaction of the two items resulted in the large reduction in estimated fatigue life (from 1.2 million to 250,000 cycles predicted by the Paulson equation) for a slightly damaged girder.

7.6.3 Strand Severing Data - Girder 2

In the damaging procedure, the exposed strands were severed with a torch. The output of the strain gages was measured with a strain indicator. The strain gages indicated an effective stress of 114 ksi lost in the strands during severing. Table 7.2 shows the changes in strain and corresponding changes in stress measured from the gages on the severed strands. The data from strand 1 was suspect. When the concrete was removed in the damaged region, the hammer drill had damaged the strand. No data was available for strand 3 which broke during fatigue loading.

7.7 Ultimate Strength Test - Girder 2

Upon completion of the fatigue program, girder 2 was loaded statically to failure. The total live load versus average jack deflection plot is shown in Figure 7.12. Because of the reduction in strand area in the damaging procedure, a smaller ultimate load was obtained. In addition to the strands which were intentionally severed, "popping" sounds were heard during the final two intermediate static tests and during the initial stages of the ultimate test. The sounds were attributed to additional strand wires failing internally in the concrete. The large drops in load at the 10 and 18 inch deflection levels were due to repositioning of the load frame.
The ultimate load obtained was 209 kips and the girder partially failed at a deflection of 25 inches and completely failed at 26 inches. At the first major drop in load near failure, the composite deck concrete failed. There was still enough compressive capacity in the longitudinal deck bars and the top flange of the girder to sustain load. More deflection was imposed, the deck bars then buckled, and the girder top flange failed. Back figuring from 209 kips peak load, the estimated remaining number of effective strands was 24.

Figure 7.13 shows a plot of the ultimate test response of both girder 1 and girder 2. Both exhibited compressive failures and deflections of over 20 inches before failure. For girder 2, near centerline, the cracking had propagated through the girder and into the composite deck before failure.

The deflection at failure of girder 2 was larger than girder 1. Because strands were severed and had broken, less prestress was present on the cross section. In order to fail the deck of girder 2, the neutral axis had to be driven higher into the cross section. This was accomplished by imposing more deflection into the girder. Once the depth of the compressive block had been reduced sufficiently, the girder failed.
Chapter 8 - Testing of Girder 3

8.1 Introduction

Girder 3 was used to evaluate a damaged and repaired girder. After initial cracking, static tests and a predamage fatigue program, damage was introduced into the specimen. Two strands were severed during the damaging procedure. Both severed strands were individually repaired with internal strand splices which are described in Chapter 5. After both strands had been spliced, a preload was applied to the girder and the missing concrete replaced. The preload, applied with the actuators, decompressed the bottom of the girder. After 3 days, the preload was removed, static tests were conducted, and the fatigue program resumed. Unlike girder 2, which was loaded at a constant load level, girder 3, once repaired, was fatigue loaded at increasing load levels. The fatigue loading continued until a partial failure occurred. The failure occurred in one of the repaired strands just outside of the splice. With one splice remaining fully effective, fatigue loading was stopped and the ultimate load test performed.

8.2 Initial Static Tests - Girder 3

The initial cracking test of girder 3 is shown in Figure 8.1. The peak load applied was 128 kips. As a result, similar to girder 2, less initial cracking was introduced compared with girder 1. The initial static test conducted on girder 3 showed it to possess approximately the same load-deflection behavior as girders 1 and 2 when they were loaded to 128 kips. For this reason, the prestress losses for girder 3 were assumed to be the same as the previous girders. The initial loading to 128 kips produced a deflection of 0.9 inch. No special instrumentation was installed to monitor crack opening during static tests. A comparison of the initial tests for the first three girders is shown in Figure 8.2.

8.3 Damage/Repair Procedure - Girder 3

Fifteen steps were performed during the damage/repair procedure. The list is summarized in Table 8.1. At the end of the predamage
fatigue program, a static load test was performed to determine the response of the girder prior to damage. Following the test, strands 1 through 4 were exposed for approximately six feet at centerline. During concrete removal, the strand pattern was found to be placed one inch lower than specified on the plans. Strain gages were installed on each of the exposed strands. Subsequently, a static test was performed to measure girder response with the concrete removed. Strand stress ranges were obtained as the girder was slowly fatigue loaded.

With data collected for this state of damage, additional damage was introduced by severing strand 2. Subsequently, a static test was performed and strain ranges collected. Strand 1 was then severed and tests were performed to measure the response of the girder at this damage level. The strands are labeled in Figure 7.3.

A similar method was used to measure behavior at various levels of repair. The first repair splice was installed on strand 1. It was tensioned to 18.5 kips. The 18.5 kip load was determined by multiplying the strand area, 0.144 inches$^2$, by a stress of 130 ksi. A static test was conducted and strain ranges collected. However, additional seating loss occurred as a result of the static test. The load in strand 1 dropped from 18.5 kips to 13.2 kips. Prior to the installation of the second splice, the load in the first splice was restored to 18.5 kips. During installation of the second turnbuckle splice, the load in splice 1 dropped from 18.05 to 17.8 kips.

The second turnbuckle repaired strand 2. It was initially tensioned to 18.0 kips. After installation, a static test was performed and strain ranges collected. The load dropped to 13.8 kips in the second splice after the static test. No additional force was reintroduced into the splice. Thus, the response of both a retensioned and a singly tensioned splice were examined with girder 3.

Because the girder was fabricated with a larger strand eccentricity, all strands placed one inch lower than specified in design drawings, no confining steel was placed around the repair (lack of space). In addition, the patch concrete cover, in the vicinity of
the turnbuckle splices, was negligible. Figure 8.3 shows both
turnbuckles installed. The load cells were located in the left half
of the turnbuckles. One can see there was minimal concrete cover
below the bottom splice.

A preload of 89 kips was applied to the girder. Under preload,
the turnbuckles had loads of 20.2 and 17.8 kips in splices 1 and 2,
respectively. Formwork was placed around the damaged region and the
missing concrete replaced. After three days the preload was removed,
a static test performed and strain ranges obtained. The turnbuckle
loads after the preload had been removed were 10.0 kips in splice 1
and 7.1 kips in splice 2. The damaged region after the repair was
completed as shown in Figure 8.4. The patch concrete was of a
slightly darker color than the original girder concrete. The wires
protruding from the patch were attached to strand strain gages and the
turnbuckle load cells.

8.4 Fatigue Loading Program – Girder 3

Similar to girders 1 and 2, fatigue loading for girder 3 was
carried out at discrete levels. Range 1 corresponded to centerline
moments varying between 7,430 and 20,500 kip-inches. Range 2
corresponded to moments varying between 7,430 and 22,500 kip-inches.
Lastly, range 3 produced centerline moments varying between 7,430 and
24,500 kip-inches.

Figure 8.5 shows the complete peak load versus cumulative cycles
plot for girder 3. Prior to the damaging procedure, 500,000 cycles of
range 3 loading were applied to ensure that the girder was similar to
the two previous girders. Subsequently, girder 3 was damaged and
repaired. The repaired girder was initially subjected to 500,000
cycles of range 1 loading. This was followed with 500,000 cycles of
range 2 loading. After 280,000 cycles of range 3 loading, one of the
repaired strands failed. The fatigue program was then terminated in
order that one complete strand splice repair could be tested during
the ultimate load test.

The actuator deflection span versus cumulative cycles plot is
shown in Figure 8.6. Other than a small jump in span during the
predamage fatigue loading, the girder performed well through the range 2 level. The actuator deflection span increased significantly during the range 3 fatigue loading. The actuator deflection span before damage, at the range 3 load level was 0.9 inch. The actuator span at the same loading level in the repaired girder was over 1.2 inches.

A number of items could be considered as possible reasons for the increase in actuator span. During the damaging sequence, static tests were conducted under load control to 128 kips live load. This was done independent of the deflection. Because strands were severed, more deflection was required to develop the internal moment to resist the external load. As more deflection was imposed in the girder, more cracking was introduced. The increased cracking softened the specimen. This softening continued during the fatigue program.

8.5 Intermediate Static Tests - Girder 3

The complete series of static tests is shown in Figure 8.7. During the initial cracking test, the midspan deflection was 0.9 inch. At the end of the fatigue program, the deflection had increased to 1.5 inches.

Similar to the previous girders, static tests were performed on girder 3 at key points of the fatigue loading program. The predamage static tests are shown in Figure 8.8. The predamage fatigue program produced a larger loss of stiffness in girder 3 than in girder 2. The increase in static test deflection from just over 1.0 inch to an 1.25 inches corresponds to the jump in the actuator span versus cumulative cycle plot of Figure 8.6. The cause of this softening is unclear.

A comparison of the load-deflection response of girder 1 after range 3 loading and girder 3 prior to the damage procedure is shown in Figure 8.9. The peak deflections at 128 kips differ by less than 0.1 inch. Slightly more hysteresis is demonstrated by girder 3. This may be attributed to a less severe initial static test series and a much shorter fatigue program.

The static tests which were performed during the different phases of the damaging procedure are shown in Figure 8.10. There were
approximately equal amounts of softening of the girder as a result of concrete removal, strand 1 being cut, and strand 2 being cut. Each increased the midspan deflection of the girder approximately 0.1 inch. Before damage, the peak deflection was 1.2 inches. After two strands were cut, the peak deflection for a 128 kip static test was 1.5 inches.

Figure 8.11 shows the response of girder 3 at various stages of repair. The repair of each severed strand and the placement of patch concrete stiffened the girder slightly. Before repair, the peak deflection was 1.5 inches. After the patch concrete was placed and the repair completed, the girder had a peak deflection of 1.4 inches. The response of the girder after repair was quite similar to the response of the damaged girder. This was attributed to cracks which formed or increased during the damaging sequence. The new cracks were not repaired, therefore, the stiffness of the girder was not improved as a result of the splices being put in place.

Static tests were conducted during the fatigue program of the repaired girder. A plot of these tests is shown in Figure 8.12. This figure shows a slight softening as a result of the fatigue program. After the patch was placed, the peak deflection was 1.4 inches. After the first repaired strand failed, a 128 kip static test resulted in a peak deflection of 1.5 inches.

8.6 Strain Gages - Girder 3

Twenty strain gages were installed on the exposed strands of girder 3. The layout of the gages is shown in Figure 8.13. Gages 1 through 16 were installed before strands 1 and 2 were severed. Gages 17 through 20 were installed after the strands were severed. In all, six gages were applied to strand 1, six to strand 2, three to strands 3 and 4. Lastly, two gages were applied to strand 5 which was partially exposed.

8.6.1 Static Test Data - Girder 3

One indication of the effectiveness of the internal strand splice was the response of the unsevered exposed strands (3 and 4). The
static test response of strands 3 and 4 through various levels of damage and repair is shown in Figures 8.14 and 8.15. The data plotted in these figures is the average of the strain gages on each strand. The trends in both figures are as one would expect. As more damage is introduced into the section, stresses increase at a faster rate, and as the repair procedure progressed the stress levels decreased. The response of the strands after the patch concrete was placed is shown as a dotted line. This was done to emphasize that the stress changes measured by the strain gages were not indicative of the stress levels at crack locations.

Unfortunately, the gages were not exclusively located at locations were the girder had been cracked. Because the girder was cracked during initial static tests, the cross section was weakened at discrete locations where cracks formed. Cracks would most likely form in the patch concrete at the same locations. If the strain gages had been placed on the strands at girder crack locations, strains close to the peak could have been measured. However, the gages were installed without consideration of girder cracks. Instead, the gages were installed on individual wires trying to facilitate a good installation. The patch line in Figure 8.15, indicates that the gages may have been located close to cracks for strand 4. This is demonstrated by the sharp increase in strand stress above decompression.

8.6.2 Stress Ranges - Girder 3

Software was developed for girder 3 to sample strain gage data during fatigue loading. Approximately 450 samples were taken over a 50 second time period while the girder was loaded at a frequency of 0.1 hertz. The maximum voltage, minimum voltage, voltage range, and converted strain range were stored in text files. Stress ranges were obtained by multiplying the strain range by Young's modulus (28,000 ksi). The average stress ranges for unsevered strands 3 and 4 are shown in Figure 8.16. Before the patch concrete was placed in the damaged region, the response of the strain gages was uniform. The output from the individual gages is shown in Figure 8.17. Gages 9
through 11 were installed on strand 3, and gages 12 through 14 on strand 4. All of the stress ranges were collected under range 3 fatigue loading.

The internal strand splice repair was a local repair, and for this reason, the stress ranges in the adjacent unsevered strands 3 and 4, returned almost to their original behavior before any strands were severed. If repair 2 had been retensioned to account for the seating loss, the stress range would have returned to the level before strands were severed.

The precut stress range of approximately 22 ksi corresponded to a Paulson fatigue life of 2,000,000 cycles. The 26 ksi stress range with one strand cut corresponded to a fatigue life of 1,120,000 cycles. The two strand cut damage level produced a strand stress range of approximately 30 ksi, which corresponded to a fatigue life of 676,000 cycles. Losing two of the 30 strands resulted in a fatigue life reduction of two thirds.

8.6.3 Strand Severing Data - Girder 3

The output of strand strain gages measured while severing strands 1 and 2 is shown in Figure 8.18. The figure shows the response of each gage on each strand. As expected, strands 3 and 4 picked up stress as strands 1 and 2 were cut. The increase was approximately 700 psi for each severed strand. The response of the gages on strands 1 and 2 was erratic. This was attributed to the data acquisition hardware and to the procedure used to sever the strands. The steps in the figure correspond with individual wires of the strands being cut.

The data acquisition system periodically collected a bad sample. A bad sample was determined by seeing the channel voltage go out of range for a single sample and then return to the approximate value prior to the bad sample. Because the bad samples were not filtered out during the data collection procedure, spikes appeared in the raw data plots.

In an effort to minimize the shock waves which occurred as the strands were cut, an untensioned strand was clamped on both sides of the cutting region to transfer load and reduce the shock on the strain
gages. However, the length required to "engage" the clamps transversely was longer than the shortening of the strand. In short, the benefit of clamping the strand was minimal. The strand severing data showed a lot of variability. The effective stress in strands 1 and 2 as demonstrated by strain gage output was estimated to be 111 ksi. This was obtained by averaging the top two gages for strand 1 and the top three gages for strand 2. The other gages may have been installed on wires which were damaged during the concrete removal phase of the damaging procedure. Girder 3 strands were damaged during concrete removal because they were located one inch lower than specified on the plans. (Strands in girders 1, 2 and 4 were located according to the specifications.)

8.7 Ultimate Strength Test - Girder 3

The response of girder 3 to the ultimate load test is shown in Figure 8.19. The test was performed under deflection control. Unlike the previous two girders which failed in compression, girder 3 exhibited a tension failure.

Girder 3 performed well up to an applied load of 160 kips. At this point, the patch concrete popped away from the girder. As a result, part of the repaired strands failed. While being popped away from the girder, the turnbuckles were still anchored in the patch concrete. At a load of 210 kips (midspan deflection of 10 in.), the repaired strands were visibly severed from one end of each of the turnbuckles. The dangling turnbuckles were removed from the girder with a torch. The complete failure was attributed to the flexural rigidity of the turnbuckle being much greater than the strand.

After the turnbuckle splices had failed, the damaged flange began to peel away from girder similar to behavior observed for girder 2. The peeling cracks on the west end of the girder are shown in Figure 8.20.

After a peak load of 239 kips was applied to the girder, audible popping or reports were heard as the test progressed. The reports were assumed to be individual wires of the strands fracturing at crack locations. As the tension steel failed, the moment resisted by the
girder dropped. This continued until a number of wires fractured together. The girder was unable to carry its own weight and fell onto cribbing. Summing up, the girder carried a peak load of 239 kips at a deflection of 7 inches. Just prior to failure, at approximately 21 inches of deflection the girder was carrying a load of approximately 100 kips.

The damaged region after the ultimate load test is shown in Figure 8.21. Shown on the floor of the lab is the rubble of the patch concrete and the turnbuckles. Also shown are the large flexural cracks which were produced during the test to failure.

The ultimate load test of girder 3 is compared to girder 1 in Figure 8.22. As can be seen from the figure, up to approximately 240 kips live load, the response of the two girders is similar. Thereafter, girder 1 continued to pick up load, and girder 3 lost load carrying capacity with increased deflection.

A possible reason for the tension failure was increased strand damage during the fatigue program. The stress ranges may have been magnified on the undamaged side of the girder. The turnbuckle splices may have worked as anchors on the damaged side of the girder. A second possible reason was the concrete patch itself. Because the prestress losses were underestimated, the preload placed on the girder was too large. Therefore, the cracks were locked slightly open on the repaired side. This also would have forced the strands on the undamaged side to move through a larger stress range under fatigue loading.

8.8 Performance of the Internal Strand Splice - Girder 3

After each splice was put into place, a static test was conducted. The load in each splice dropped significantly after the test. This reduction was attributed to additional anchorage loss in the chucks. The strand 1 splice was retensioned prior to the installation of the splice for strand 2. The splice for strand 2 was not retensioned following the static test performed after its installation.
8.8.1 Magnification of Repaired Strand Response - Girder 3

The turnbuckles had a much larger axial stiffness than the prestressing strands. As a result, the turnbuckles magnified the stress ranges in the repaired strands. Figure 8.23 shows a simple model to describe the behavior. The model assumes that the number of strands repaired is small. Because the repair is small, the deformations the strands move through to resist the applied loads is constant. The average stiffness of the repaired strand is larger than the undamaged strand, therefore, more force is required to move it through the same displacement. The amplification for the example shown in Figure 8.23 is 21/9.

Figure 8.24 shows plots of magnification factors. The figure shows the interaction for repairs with varying stiffnesses and splice lengths of varying percentages of the damaged region. The figure shows plots for repairs with stiffnesses 5, 7, and 9 times greater than that of the strand. If 60 percent of the repaired strand is turnbuckle, the stress ranges are approximately double those in the undamaged strands.

The model is valid until the concrete is replaced in the damaged region. The strain distribution becomes complex after the patch is in place. The strand at the strand/chuck interface is subjected to large stresses. As fatigue loading progresses, one would assume that the bond between the patch concrete and the strands near the chucks would deteriorate and the strand yield. It is anticipated that the repair would perform better under fatigue loadings if the patch concrete was not allowed to bond to the splice or repaired strands. This would allow a larger region of the strand to deform and accommodate the axial stiffness discontinuity.

8.8.2 Static Test Results - Girder 3

The performance of the strand splice during static tests is shown in Figures 8.25 and 8.26. Figure 8.25 shows the initial static tests which were conducted with one turnbuckle splice in place. The load cell cable was disconnected from the conditioner during the first static load cycle. As a result, the figure begins with the splice
having a load of 14 kips rather than the initial 18.5 kips. During
the additional static tests performed at this damage level, more
seating losses occurred. At the end of the tests, the effective load
in the turnbuckle was just over 13 kips.

Figure 8.26 shows the response of both turnbuckles after both
strands were spliced. Splice 1 had been retensioned and splice 2
exhibited a response similar to splice 1 before retensioning. During
the static test to a peak load of 128 kips, the force in each splice
increased approximately 9 kips.

The response of strands during a static test is shown in Figure
8.27. The response of both an undamaged and a repaired strand are
shown. As expected, the stress change in the repaired strand is
approximately twice that of the undamaged strand. The undamaged
strand exhibited a stress change of 33 ksi, while the repaired strand
exhibited a stress change of 67 ksi.

8.8.3 Stress Ranges - Girder 3

A comparison of stress ranges for strands 1 and 4 under range 3
loading is shown in Figure 8.28. Strand 1 was the second strand to be
cut and the first to be repaired. Strand 4 was not cut in the damage
simulation procedure. Figure 8.28 shows the stress ranges to be
similar in strands 1 and 4 for the cases of zero and one strand
severed. Strand 4 shows an increase in stress range for increasing
damage, and a reduction in stress range for increasing levels of
repair. Strand 1 shows a similar trend during the damaging phase.
However, the stress ranges in repaired strand 1 are much higher than
those of strand 4. As mentioned previously, the stress ranges of the
repaired strands were magnified by the axial stiffness of the
turnbuckle.

Figure 8.29 shows force range information for both load cells
under range 3 fatigue loading. The force range in splice 1 reduced as
a result of splice 2 being put in place. However, after the patch
concrete was placed, the force ranges increased dramatically. Both
increased to nearly twice that of the prepatch situation. The
increase in load in the load cell is attributed to the turnbuckle.
being bonded to the patch concrete. Instead of only the strand introducing load into the turnbuckle, additional load is introduced via bond and bearing with the patch concrete.

A failure occurred in repaired strand 1 after 280,000 cycles of range 3 loading. A crack was evident in the patch concrete, the load in turnbuckle 1 was reduced from the previous day, and the strain gages were not responsive on the east end of strand 1.

8.8.4 Ultimate Test Performance - Girder 3

The output of the turnbuckle load cells for the ultimate test of girder 3 is shown in Figure 8.30. The effective load in both load cells had dropped significantly as a result of the fatigue program. Turnbuckle 1 started the ultimate load test with an effective load of 6 kips. Turnbuckle 2 began the test with a load of 8 kips. During the ultimate load test, both load cells picked up load slowly until approximately 90 kips of live load. After this point, both load cells picked up load at an increasing rate until a live load of 150 kips was reached. Thereafter, turnbuckle 1 maintained a load of 20 kips until the live load on the girder reached 210 kips. The load in turnbuckle 2 dropped from 26 kips to 23 kips at 150 kips of live load. The turnbuckle 2 load increased to 27 kips when the girder was under a live load of 210 kips. No valid turnbuckle information was obtained after the 210 kip load level. The turnbuckles fell away from the girder. In the process, the load cell wiring was severed.

8.8.5 Potential Problem Areas - Girder 3

There are potential problem areas associated with the turnbuckle splice repair. The loss of effective stress due to additional anchorage seating loss is one area where attention should be given. The second area is the magnitude of the axial stiffness of the repair relative to the strand. A third problem exhibited by girder 3 was associated with the lack of room for confining steel. During the large displacements imposed in testing to failure, the turnbuckle was not forced to follow the curvature of the girder as it deflected.
Rather, the turnbuckle remained flexurally rigid and forced spalling of the patch concrete.

For the installation of the repair in the field, two items should be considered. First, blanketing the entire turnbuckle assembly should be considered. A bond breaker, such as PVC tubing, would eliminate the patch concrete from introducing additional load into the splice. Second, the exposed strands which are repaired should also be blanketed. This would allow the deformations which occur within the strand to be distributed over a significant length and not forced to occur at the strand/chuck interface.

After the turnbuckle splice has been detailed for sufficient static strength, the question arises as to what load level the repaired strand should be retensioned. If it is not required that the repair restore prestress to the cross section for corrosion protection or to reduce stress ranges, it could simply be put in place and hand tensioned. In this situation the splice would only be effective for extreme overload situations. However, if the repair is needed to restore prestress to the cross section it is important to know the appropriate level of retensioning.

Further research needs to address the issue of minimum stress on the fatigue life of prestressing strand. This is a design position where the minimum stress is at the discretion of the designer. In low cycle fatigue applications, the strand could be retensioned to the undamaged strand force. While in high cycle fatigue applications, the fatigue life should be estimated with the model presented in Section 8.8.1. The model would provide an estimated stress range in the repaired strand. The stress range coupled with Paulson’s strand fatigue equation[15] would result in an estimated repair life. If the fatigue life is too small, the length of undamaged strand could be increased, the repair designed with a smaller axial stiffness, or the repair not fully retensioned.
Chapter 9 - Testing of Girder 4

9.1 Introduction

Girder 4 was utilized as a damaged and repaired specimen. Two strands were severed during the damaging procedure. Two 5/8 inch diameter high-strength rods were used to repair the section. The rods were attached to the girder with corbels cast against the bottom flange. A predamage fatigue program was applied to the girder, similar to that of girders 2 and 3. Unique to girder 4 was the installation of strain gages on the longitudinal deck reinforcement. These strain gages were used to estimate composite deck strains during curing.

9.2 Initial Static Tests - Girder 4

In the initial static tests, girder 4 was loaded to 144 kips similar to girder 1. This load exceeded the 128 kip initial static load imposed on the other girders in the damaged series (girders 2 and 3). Similar to girder 1, crack detection gages were installed on the bottom flange of girder 4 to determine the load at which cracking first occurred. However, due to improper gluing, the gages did not function properly. The gages detected cracking at the same time audible cracking was heard. Immediately after the cracking test, a subsequent load test was performed with a peak load of 128 kips. Figure 9.1 shows a plot of the initial static tests.

A comparison of the initial cracking tests for all four girders is shown in Figure 9.2. Girder 4 demonstrated stiffer response than the previous girders. The cause for the additional stiffness is unclear. One possible reason may be less deck shrinkage affecting the girder 4 tests. The age of the concrete deck at the time of the girder 4 test was slightly over one month; whereas the other girders were tested with concrete decks approximately 4 months of age.

9.3 Damage/Repair Procedure - Girder 4

Twelve steps were performed during the damage/repair procedure. A complete list is given in Table 9.1. The damaging procedure was
identical to that of girder 3. The repair procedure differed slightly.

At the end of the predamage fatigue program, a static load test was performed to determine the response of the girder prior to damage. Following the test, strands 1 through 4 were exposed for approximately four feet at centerline and strain gages were installed.

A static test was performed to measure girder response with the concrete removed. In addition, strand stress ranges were collected as the girder was fatigue loaded. Strand 2 was then severed. Again, at this damage level, stress ranges were collected, and a static test performed. The procedure was repeated after strand 1 was severed.

Response was similarly measured during the repair procedure. The high-strength rods were tensioned to 25 kips each. A static test was then performed and stress ranges collected. The girder was then preloaded to decompress the bottom of the girder. Formwork was placed around the damaged region and the missing concrete replaced. The patch concrete was allowed to cure for three days. After which the preload was removed and the repair considered complete. A static test was then performed and stress ranges collected to measure the initial response of the repaired girder.

9.4 Fatigue Loading Program - Girder 4

Similar to the previous girders, fatigue loading was conducted at discrete levels. Four different levels or ranges were used. Range 1 produced a moment field varying between 7,430 and 20,500 kip-inches and range 2 correlated to a moment field between 7,430 and 22,500. Range 3 varied between 7,430 and 24,500, and lastly, range 4 varied between 7,430 and 28,500 kip-inches. Each of the moment fields was determined at centerline and included dead loads. The basis for the fatigue loading levels is described in Section 6.3.

A short predamage fatigue program was applied to girder 4. This consisted of 36,000 cycles of range 1, 252,000 cycles of range 2, and 94,000 cycles of range 3 loading. After repair, an increasing load fatigue program was applied. For each of ranges 1-3, 500,000 cycles of load were applied. Subsequently 14,000 cycles of range 4 loading
were applied. Figure 9.3 shows the plot of peak total live load versus cumulative cycles. In all, over 2 million cycles were applied to girder 4. Figure 9.4 shows the corresponding plot for actuator span versus cumulative cycles. The girder exhibited stable response at each of the load levels. The repaired girder showed little distress as a result of the fatigue loadings. The only potential problem was noticed in the corbels near the bearing plate ends. During range 3 fatigue loading, a small amount of movement could be seen at the interface of the corbel and girder flange. This movement extended approximately six to eight inches until a flexural crack intersected the corbel. The interface movement was "drained" out through the crack. From the flexural crack towards centerline no interface movement was detected.

9.5 Intermediate Static Tests - Girder 4

Similar to the previous girders, static load tests were conducted at key points during the fatigue loading program. Figure 9.5 shows a plot of all the intermediate static tests which were performed on girder 4. A subset of results shown in Figure 9.5 is given in Figure 9.6 which shows the intermediate static tests which were performed after initial cracking and before the damaging sequence began. As can be seen from the latter figure, the predamage fatigue program increased the static load test deflection 0.1 inch. The predamage fatigue program could be expected to produce less damage than girders 2 and 3 for two reasons. First, the number of cycles at the range 2 and range 3 loading levels were less than the previous girders. And probably more importantly, the initial static tests which were performed on the girder induced more damage than what was initially introduced into girders 2 and 3. In other words, the cracks had propagated further before the fatigue program commenced.

Figure 9.7 shows a comparison between the predamage static test of girder 4 with the static test conducted on girder 1 at the end of its range 3 fatigue loading. Girders 2 and 3 showed predamage response within 0.1 inch of girder 1. As can be seen from the figure, girder 4 deflected 0.2 inch less than girder 1. A possible
explanation for part of the increased stiffness of girder 4 was the
timetable under which the tests were conducted. Girders 1, 2, and 3
were tested after the composite deck had aged approximately six
months. The predamage test of girder 4 was performed only three
months after the deck had been cast. Being tested in a shorter time
frame would allow less deck shrinkage to take place. Less deck
shrinkage results in more compression in the bottom of the girder.

The static tests which were performed during the various stages
of damaging the specimen are shown in Figure 9.8. As in previous
tests, the removal of concrete section and the severing of strands
soften the response of the girder. As a complement to Figure 9.8 is
Figure 9.9 which shows the subsequent static tests which were
performed during repair stages. Similar to girder 3, the repair
slightly stiffened the response during static tests.

Lastly, static tests were conducted at key points during the
fatigue loading program. The tests are shown in Figure 9.10. Like
girder 3, there was a small amount of deterioration as a result of the
fatigue program.

9.6 Strain Gage Data - Girder 4

Strain gages were installed on both exposed strands at centerline
and in the composite deck at the west quarter point. The deck gages
were used to determine the magnitude of the strains in the composite
deck during curing. The strand gages were used to measure strand
behavior under static and fatigue loadings as well as during strand
severing and repair.

9.6.1 Shrinkage Gage Data - Girder 4

Seventy-five gages were installed on deck reinforcement at the
quarter point. The quarter point was chosen because effects from both
supports and the loading system were minimized at this location. In
addition to the gages installed on the reinforcement, eight concrete
strain gages were embedded in the deck. The output of the strain
gages was monitored during the first eighty days of composite deck
curing. Overall, the reinforcing gages showed an average strain of
approximately 250 microstrain during this time period. This shrinkage strain must be resisted by a redistribution of internal stresses. Using an average shrinkage strain of 250 microstrain in the deck, and using normal plane section analysis, the net effect would be to change the bottom fiber stress at centerline by 0.200 ksi. Using superposition, this represents a decrease in theoretical crack reopening load of 11.5 kips. The output from a longitudinal bar strain gage is shown in Figure 9.11. Figure 9.12 shows the corresponding plot for a concrete gage placed in the deck.

9.6.2 Strand Gage Data - Girder 4
Once the concrete was removed as part of the damage simulation procedure, eight strain gages were applied to the four exposed strands, two on each strand. These gages were monitored during static and fatigue loadings as well as strand severing and repairs.

9.6.2.1 Static Test Data - Girder 4
Figures 9.13 and 9.14 show the change in output of the strain gages on unsevered strands 3 and 4, respectively, during static tests. Similar to previous plots for girders 2 and 3, the strands picked up stress at an increasing rate as damage was introduced into the cross section. After the high-strength rods were tensioned, the response of the strands improved. The strands picked up less stress during a static test.

9.6.2.2 Stress Range Data - Girder 4
The stress ranges computed from the strain ranges for the unsevered strands were monitored at different damage levels. Figure 9.15 shows the stress ranges produced in unsevered strands 3 and 4. The stress ranges were collected at different stages of damage and repair under range 3 fatigue loading. Similar to girders 2 and 3, as strands were severed, there was a significant increase in stress range. However, where girder 3 showed the stress range with the repair in place to be almost the same level as it was before damage, strands 3 and 4 of girder 4 recovered only half of the increased
stress range. This was attributed to the repair being "remote" with respect to the location where the strands were severed. Unlike the internal strand splice which physically repaired the damaged strands, this repair restored prestress to the damaged cross section from a distance. The distance was at least 100 strand diameters away from the damaged region. Again, the large drop in stress range after the patch concrete was placed does not imply that such a large drop occurred at crack locations.

9.6.2.3 Strand Severing Data - Girder 4

The software developed to monitor the strain gages on the severed strands of girder 3 was improved before it was used with girder 4. The output of the strain gages installed on the strands is shown in Figure 9.16. Samples were collected at one second intervals. The output of the gages was converted to stress by multiplying the strain by Young’s modulus (28,000 ksi). The observed loss of stress in strands 3 and 4 was 110 ksi.

9.7 Ultimate Load Test - Girder 4

Following the fatigue program, girder 4 was loaded to failure. A compressive failure mode was exhibited. The load-deflection plot is shown in Figure 9.17. Unlike, the previous tests which had been conducted without unloading, girder 4 was tested with several unplanned unloadings.

The first unloading was a result of the microsegment generators being improperly set up before the test began (the microsegment generators span was not aligned with the actuators stroke span). The subsequent three unloadings were a result of tripping hydraulic interlocks which removed hydraulic power to the actuators. However, the unloadings did allow the response of the girder to be monitored during large loading and unloading cycles.

The repair performed well initially. At a deflection of approximately 12 inches, the east corbel popped off and the repair was lost. The hairpins attaching the corbel released as a result of inclined flexural cracking which had propagated through each hairpin
hole. With an enlarged hole, the corbel and a significant portion of girder flange concrete moved transversely and dropped to the floor. The local failure was documented and the test continued. After the repair failed the peeling of the damaged girder flange proceeded in a manner similar to that observed for girders 2 and 3. Large cracks propagated from the damaged region up into the girder web. Overall the specimen carried a peak load of 253 kips at 7 inches, and the failure load at a deflection of 21 inches was 220 kips.

Figure 9.18 shows a comparison of the ultimate test response of girder 4 compared with girder 1. Initially the load deflection curves were very close with girder 4 slightly stiffer. After a peak load of 250 kips, girder 4 lost its load carrying capacity with additional deflection. Initially the severed strands in the damaged region lost load carrying capacity when the patch concrete failed. This reduced the flexural capacity of the section. Another reduction occurred when the corbel failed. After the corbel failed at 12 inches, the flexural capacity was fairly constant until failure.

9.8 Performance of External Post-tensioned Repair - Girder 4

9.8.1 Static Tests

The repair performed well. Figures 9.19 and 9.20 show the performance of the post-tensioned rods during static load tests. Both show a small reduction in load with increased fatigue loading. As a result of the entire fatigue program, the top rod lost approximately 1 kip and the bottom rod 2 kips. Theses losses were attributed to anchorage losses and cracking of the corbel.

9.8.2 Stress Ranges - Girder 4

The stress ranges in the post-tensioned rods at various points in the fatigue program are shown in Figure 9.21. Rod 1, the bottom rod, had a larger stress range than rod 2. This was a result of it being located further from the neutral axis than rod 2. The stress range in rod 1 was approximately 11 ksi under range 3 fatigue loading. The corresponding stress range for rod 2 was 8 ksi. The stress ranges
were obtained by multiplying the strain range obtained from the strain gages by Young's modulus (29,000 ksi).

Two items not examined in the research were the impact of the corbels on the fatigue life of the girder and the fatigue life of the shear friction steel. Near the end of the range 3 fatigue loading for repaired girder 4, movement was noticed at the corbel/girder interface near the anchorage end of both corbels. This movement continued until a flexural crack crossed the interface. After the flexural crack (towards centerline), no interface movement was detected. Figure 9.22 shows a schematic of the corbel region.

Two possible failure modes could be induced by the corbels. First, fatigue of hairpin steel could occur in the corbel region where interface movement was detected. Second, the corbels could restrain the opening of girder flexural cracks along the length of the corbel. As a result, cracks on either side of the corbel would open wider. Wider crack opening would increase the strand stress ranges at crack locations.

9.8.3 Ultimate Load Test - Girder 4

The response of the post-tensioned rods during the ultimate load test of girder 4 is shown in Figure 9.23. Figure 9.23 shows the total live load applied to the girder versus the strain in each of the rods. The strains were not converted to stresses or forces because the rods yielded. The figure shows that the rods began yielding at a live load near 175 kips.

The repair failed when the girder was taken to a deflection of 11.8 inches. The east corbel fell away from the girder, thereby removing one of the anchors for the post-tensioning rods. Inspection of the corbel which fell away, showed that a significant amount of girder concrete was removed by the corbel when it broke away. Also noted was the intersection of inclined flexural cracking with the hairpin holes. The flexural cracking opened the hairpin holes and allowed the corbel to move away from the girder. The use of through-web reinforcement may restrain the corbel from sliding away and thereby preserve the dowel action of the hairpins.

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Figures 9.24 through 9.27 show pictures of the damaged region during the ultimate load test. A view of the west corbel is shown in Figure 9.24. At this point in the loading program the east corbel had failed. Large cracks propagated away from the damaged region. One crack intersects the corbel, while others (under the high-strength rods), propagated around the corbel. Flexural cracks which intersected the dowel rod holes of the east corbel are shown in Figure 9.25. An overall picture of the damage region at 12 inches of deflection is shown in Figure 9.26. The west corbel was intact, the east corbel had failed, and the patch concrete was lost. The damaged region after additional loading is shown in Figure 9.27. The high-strength rods had been removed and the bottom flange of the girder had deteriorated more.

9.8.4 Potential Problem Areas - Girder 4

The importance of potential fatigue failure modes associated with the corbels has not been examined. To minimize fatigue problems, the corbels should be designed as small and as short as possible.

The corbels should be detailed to prevent water from deteriorating the repair. The corbels tested, were constructed with flat top faces. It would be prudent to incline the top faces of the corbels to provide drainage away from the girder.

n measurements indicated concentrations less than 100 ppm. The highest chloride concentrations measured were not found to significantly exceed the corrosion threshold concentration (250 ppm by weight of concrete) with one exception. The exception, point 2-4-3B with 430 ppm chlorides at a depth of 2 in., was considered suspicious. Chloride ion concentrations measured at shallower depths for the same point were consistently lower (i.e. 210 ppm at 1/2 in. depth, 200 ppm at 1 in. depth, and 70 ppm at 1-1/2 in. depth).

The only evidence of steel corrosion observed in the girders was at the end faces where the epoxy coating over the beam ends had been chipped to expose the ends of the prestressing
strands. Rust stains were visible on the concrete at this location, but spalling was not observed.

14) Chloride ion concentrations of the bridge deck were found to be elevated well above the chloride ion concentration threshold for corrosion at the level of the steel. Nevertheless, obvious visual evidence of corrosion was not found.

In general, it was confirmed that the nondestructive tests of this study must be calibrated in order to yield accurate concrete strengths and that these tests are best used as an indication of concrete uniformity. Further work on the break-off tester is needed to further delineate its limitations for testing high strength and/or large aggregate concretes. Core testing gave different results for 2 in. and 4 in. cores which led to the derivation of L/d correction factors that differed from ASTM C42 factors. Whether this was due to core size only or core size/aggregate ratio was not apparent.

Results of chloride ion testing showed that chloride ion penetration into the bridge girders was much less than for the bridge deck. In the girders, chloride ion concentrations measured at the reinforcement level had not reached a critical level after 20 years of service.

9.8.5 Applications - Girder 4

The repair is most appropriately used where fatigue loadings are below decompression. If the fatigue loadings are kept below decompression, the integrity of the corbel attachment is not compromised. While the corbels demonstrated no significant deterioration as a result of the fatigue program, the inclined cracking produced during the ultimate test resulted in failure.

The longer the damaged region, the more attractive this repair becomes. The amount of work associated with the repair is independent of the length of the damage. Only material increases would be needed: longer post-tensioning rods and more patch concrete.
Chapter 10 - PBEAM Modeling

10.1 Introduction

The program PBEAM[38] was used to estimate the response of the test girders. Four areas of girder response were examined with PBEAM:

1) A time-dependent analysis was performed to estimate prestress losses after twenty years. In addition, the effect of composite deck removal and replacement was examined.

2) Assuming the girders were uncracked when they arrived for testing, an estimate of the live load required to crack the specimens was obtained with another analysis.

3) To estimate strand stress ranges, strand stress versus bending moment data was generated. Runs were made with effective strand stresses of 110, 130, and 150 ksi, with and without additional deck shrinkage.

4) Lastly, a failure load analysis was performed. Runs were made with effective strand stresses of 110, 130, and 150 ksi.

The program is based on the discrete element method. The discrete element method is a mechanical model. The model consists of a rigid piston with an axial spring, two rotational springs, and two rigid end blocks. In his thesis, Suttikan listed the following assumptions and limitations of analysis with PBEAM.

1) Beams are straight in their original positions and are of cross-sectional shape having one axis of symmetry.

2) Bernoulli's hypothesis holds, i.e. strain increment distributions vary linearly through the depths of the beams.

3) The deformations (strains and curvatures) are small although the displacements (horizontal, vertical, and rotational) can be any size.

4) Shear deformation is negligible.

5) No out of plane movement is considered, i.e. it is assumed that no lateral or local buckling occurs.
(6) Only statically applied loads in the plane passing through the axis of symmetry of the cross section are considered.

(7) Material properties in the beams can be represented by the uniaxial behaviors of those materials.

(8) Equilibrium equations are written in the deformed state.

(9) Soil and other restraints can be represented by sets of linear or nonlinear springs.

Overall the program is quite general. A user can input nonlinear stress-strain curves as well as creep/relaxation, shrinkage, and aging effects of the materials. For both stress-strain curves and the time-dependent effects, a number of standard internal models are provided, otherwise, a user-defined model can be input by prescribing a set of discrete points.

10.2 Assumptions Made

Experimentally determined material properties were used in conjunction with the girder fabrication records. The fabrication reports indicated that the strands were individually tensioned to 175 ksi. The prestress was transferred to the girders 1 day after the girders were cast. The 28 day girder concrete compressive strength was 6700 psi, and the initial camber at transfer was 3/4 inch.

Coggins[30] determined the girder concrete compressive strength to be 8400 psi after 20 years. The coefficients for the ACI aging function were modified in order that the age function intercepted the measured strength values at 28 days and 20 years. Otherwise, the suggested internal models (ACI or PCI) for both girder and deck concretes and prestressing strand were used.

The girder and both original and lab replacement deck concretes were permitted to age, shrink, and creep. Both decks were modeled with a concrete compressive strength of 4500 psi at 28 days. The prestressing strand was allowed to relax. The characteristics of the mild steel reinforcement (except for the reinforcement in the original deck), was considered to be time-independent.

The concrete model proposed by Suttikan was utilized. This model assumes the peak compressive stress occurs at 0.0022 strain and the
unloading portion of the curve is determined by the amount of confinement. The mild steel reinforcement was modeled as an elastic perfectly-plastic material with a yield stress of 60 ksi. The stress-strain curve for the prestressing strand was described by inputing discrete points along the PCI handbook curve for 250 ksi strands.

10.3 Long-Term Analysis

A long-term analysis of the test girders was conducted with PBEAM. Data was obtained for stresses, strains, forces, and displacements over an 8500 day time period (23.3 years).

The following assumptions were made in the analysis:

1. Prestress was transferred the day following casting of the girder. (Based on the 28-day compressive strength, PBEAM calculated \( f'c_{1\text{-day}} = 4200 \) psi; the measured strength at transfer was \( f'c_{\text{transfer}} = 4905 \) psi according to the construction records.)

2. The concrete modulus was manipulated until the initial camber matched the fabrication reports value of 3/4 inch. (\( E = 6500 \) ksi, with a 28-day compressive stress of \( f'c = 6700 \) psi)

3. The first composite deck was added to the girder at 75 days. This deck was 6 inches thick and 84 inches wide.

4. The first deck was removed at 7000 days (approximately 19 years). The 7000 days represented the approximate period of time the girders were in service. The deck was removed in the program by forcing the strength aging functions for both the deck concrete and reinforcement to zero at 7000 days. Also, at 7000 days the dead load from the original deck was removed.

5. The girder was allowed to sit 500 days without a deck before the second composite deck was added. The second composite deck, applied to the test girders in the lab, was 6 inches thick and 64 inches wide. The 500 day value was chosen to represent an amount of time large enough for the time-
dependent effects to stabilize after the girders were removed from the bridge.

(6) The PBEAM analysis was terminated after investigating the behavior of the girders through 8500 days. The time-dependent effects from the second deck had stabilized at that time.

The discretization of the cross section at both centerline and at the ends is shown in Figures 10.1 and 10.2. The only difference between the figures is the location of the draped prestressing strands. The cross section of the girder was constant in the center 10 feet of the girder. Outside of this 10 foot region the draped strands varied linearly to the ends while the rest of the elements remained constant. Each composite deck was discretized into six 1 inch concrete fibers. The girder was divided into 34 concrete fibers. The span was divided into 20 elements, the maximum PBEAM could accommodate with the number of subrectangles used to describe the cross section.

The centerline bending moment on the cross section is shown in Figure 10.3. The dead load bending moment of the girder alone was 3700 kip-inches. The dead load bending moment of the composite section with deck one in place was 6900 kip-inches. The second deck being narrower produced a smaller bending moment of 6200 kip-inches.

Figure 10.4 shows the variation of centerline deflection with time. After the prestress was transferred, the initial camber was 0.752 inches upward. This deflection increases upward until the composite deck is added at 75 days. Prior to placement of the original deck the centerline position was 1.44 inches upward. The deck lowered the position of the girder to a minimum of 1.03 at a time of 131 days. Thereafter the centerline position increased with time to 1.23 inches. At 7000 days the original composite deck was removed. This caused the centerline position to move upward. At 7500 days the second deck was added to the beam. The additional dead load from this deck and the shrinkage associated with it both caused the centerline position to lower.
The response of the girder was examined at various locations on the cross section at midspan. Stresses and strains were plotted versus time. For the girder, the bottom flange, top flange, and bottom strand layer were examined. For each deck, the middle concrete layer and the top rebar layer were examined.

The stress in the prestressing strands as a function of time is shown in Figure 10.5. Initially the strands were tensioned to 175 ksi. After transfer, elastic shortening reduced the stress to 162 ksi. Thereafter the stress dropped quickly to 142 ksi at 28 days and 131 ksi at 400 days. Little change in strand stress is seen through the rest of the analysis. A small reduction in stress (2 or 3 ksi) occurs at 7000 days when the original deck is removed. At 7500 days when the second deck is added, the stress increases 2 ksi and increases another ksi as shrinkage occurs in the deck.

Figure 10.6 shows the variation of concrete stress at the top of the precast section of the composite girder. Initially, after transfer, the top was in tension, approximately 200 psi. As a result of the addition of the deck, and the shrinkage of the deck the girder top concrete stress became approximately 850 psi compression. After the deck was removed, the stress returned to 300 psi tension. As one would expect, when the second deck was added, the girder top concrete stress returned to compression and stabilized at a magnitude of 900 psi.

Figure 10.7 shows the different strain components for the girder top concrete fiber. Shown are shrinkage, creep, and instantaneous strains. The shrinkage strains increase to a peak of 387 microstrain at 7000 days. However over 300 microstrain had been attained at an age of 200 days. The creep strains follow the trend of the instantaneous strains. When the instantaneous strains are tensile the creep strains move in that direction and when the instantaneous strains are compressive, the creep strains are negative. The relative magnitudes of the different components are such that the creep strains are the smallest, less than 100 microstrain, the instantaneous strains are less than 150 microstrain, and the shrinkage strains are the largest component with values over 350 microstrain.
The stress in the girder bottom concrete fiber is shown in Figure 10.8. Throughout the time history the bottom fiber was in compression. The peak compressive stress, just after transfer, was approximately 2300 psi. Concrete creep and relaxation of the strands reduce the stress. The addition of the composite deck also reduces the bottom fiber compression. After the first deck is added the stress stabilizes at approximately 1000 psi compression. When the original deck dead load was removed, the compressive stresses increased to 1600 psi. The addition of the second deck restored the dead load and the stress returned to 1000 psi compression.

The individual strain components of the bottom concrete fiber are shown in Figure 10.9. The shrinkage strains being independent of stress level are the same as for the top fiber. However, the relative magnitudes of the three components (shrinkage, creep, instantaneous) are different. The instantaneous strains are the smallest after the first deck is placed, about 120 microstrain compression. The shrinkage strains are about 350 microstrain and the creep strains are much larger at 800 microstrain.

The stress versus time plot for the original composite deck concrete is shown in Figure 10.10. The deck stress is in compression initially, 100 psi, and then demonstrates a quick increase in tension as the deck shrinks. The tensile stress levels off near 275 psi. When the deck was removed there was a short spike to 400 psi tension. Within the program input, the dead load was removed at the same time as the aging functions for the deck materials went to zero. The spike to 400 psi tension implies that the aging functions went into effect later than the removal of load. The removal of the load reduced the bending moment on the cross section, this in turn, caused the beam to move upward and induced additional tension in the deck.

Figure 10.11 shows the strains for the original composite deck concrete. The instantaneous strains quickly become positive (tensile), corresponding to the stresses. The instantaneous strains stabilize at a value near 65 microstrain. The creep strains are also small, however they are negative or compressive, and stabilize at 60 microstrain. By far and away, the largest strain component is
Because of the different concrete mix and geometry of the deck, the ultimate shrinkage strains were larger than those for the girder concrete. The shrinkage strain of the deck concrete stabilized at negative 450 microstrain.

The stress in the top layer of reinforcement of the original deck is shown in Figure 10.12. The stress was compressive and stabilized at approximately 13 ksi.

The concrete stresses in the second composite deck are shown in Figure 10.13. Because the time dependent effects had stabilized in the girder, additional tension was introduced by the second deck. The tensile stress leveled off at 360 psi.

Figure 10.14 shows the individual strain components of the second composite deck concrete. Similar to the first deck, the shrinkage strains level off at -450 microstrain, and the instantaneous strains are a small positive value, 85 microstrain. The creep component is also tensile at 130 microstrain.

The stress in the steel reinforcement of the second deck is shown in Figure 10.15. Similar to the first deck, the reinforcement is in compression. However, the stress of 6500 psi is only half of the value to which the reinforcement in the first deck stabilized.

10.4 Estimate of Cracking Load

PBEAM was used to estimate the cracking load of the test specimens. The rupture stress of the girder concrete was obtained from split cylinder tests conducted on cores taken from the girder. A tensile strength of 580 psi was obtained from the tests and used in the analysis. Load was applied incrementally in the analysis until the bottom of the girder cracked (bottom fibers reached $f_c$).

Figure 10.16 shows two PBEAM runs compared with the cracking static test performed on girder 4. Shown are runs with an effective stress in the strands of 110 and 130 ksi. The initial strains in the strands in the input file were modified until the appropriate effective stress was achieved under dead load of 6160 kip-inches. The output from PBEAM indicates that an effective stress of 130 ksi
matched response better than the 110 effective strand stress output. From the figure, the cracking load test indicates that the effective stress for the specimen was near 130 ksi.

10.5 Stress Ranges of Strands in an Undamaged Girder

Stress ranges were estimated for the test girders using two scenarios. For the first, the experimentally determined material properties were used without any time-dependent effects. For the second, the composite deck elements were given an initial tensile strain of 100 microstrain. This initial strain was used to simulate composite deck shrinkage. The 100 microstrain produced a redistribution of the internal stress field. Additional compression was induced in the girder top flange and additional tension induced in the bottom flange. The additional bottom flange tension opened cracks at a smaller live load.

The output is shown in Figure 10.17. Six runs were made; three with deck shrinkage and three without. For both cases, runs were made at effective prestress levels of 110, 130, and 150 ksi. The 130 ksi level represents the value obtained from using the lump sum prestress losses. The 110 and 150 ksi levels represented lower and upper bounds of the effective prestress. Figure 10.17 shows that the effect of the deck shrinkage on strand stress was small.

10.6 Failure Load Analysis

PBEAM runs were made at three different effective stress levels to determine failure load. Once again, runs were made at effective stresses of 110 ksi, 130 ksi, and 150 ksi. Figure 10.18 shows the total load versus centerline deflection plots for the three runs. All show similar behavior until decompression. As the load reaches decompression, the girder softens and deflections increase at an increasing rate. The difference in the effective stress is most pronounced in the 1 to 5 inch deflection range. After 5 inches, the geometry of the cross section governs the response more than the initial effective stress. At this level the strand is yielding and
the lever arm to the compression block is the critical factor in resisting load.

Figure 10.19 shows the output of the ultimate load test performed on girder 1 (undamaged specimen), compared with the PBEAM test runs. The experimental test compares reasonably well with the 110 and 130 runs. The test reached a larger total load (293 kips). The difference was attributed to the PBEAM analysis not allowing the strand material to reach a stress above 250 ksi. The strand in the girders most likely reached a value somewhat higher.
Chapter 11 - Method to Determine Stress Ranges

11.1 Introduction

Traditionally prestressed concrete bridge members have been designed for fatigue loadings by specifying a maximum allowable concrete stress. The AASHTO code prescribes an allowable service load stress assuming an uncracked section. The indirectness of the current design method is demonstrated by the fatigue failure mode of prestressed beams. In the tests which have been performed, fatigue failures have resulted predominantly from fractures of prestressing steel[18,21,22,23,29]. In short, a concrete stress, based on an uncracked section, is used to limit a steel fatigue failure. This criteria has worked in the past when the allowable concrete stress was small, 0 or 3√fc nominal tension. However, tests conducted at the University of Texas at Austin on prestressed concrete bridge girders indicated a small number of overloads can significantly reduce fatigue life at a 6√fc load level.

If an element remained uncracked during service, the stress ranges in the strands would be small and no fatigue problems would be anticipated. However, if the element is overloaded and cracked, the stress in the strands will increase significantly for loadings above decompression. For loadings above decompression, the strands at crack locations must also carry the tensile stress carried by the concrete in the uncracked regions. Instead of specifying a concrete stress, a more rational fatigue design procedure would be based on the stress ranges in strands at cracks under various loadings.

A fatigue model for prestressing strand has been proposed by Paulson at the University of Texas[15]. Based on tests reported in the literature, and tests Paulson performed himself, he published the following design equation:

\[ \log N = 11.0 - 3.5 \times \log Sr \]

Where \( N \) is the total number of loading cycles and \( Sr \) the stress range in the strands in ksi. The difficulty in designing with this method is that the stress in the strands needs to be determined as a function
of loading. The University of Texas used two methods; first, the stress ranges were determined with a large discrete element program PBEAM, and second, from a time consuming manual calculation.

This chapter describes a simple procedure developed to determine the stress in the strands of a prestressed element at various levels of flexural loading. The procedure was extended to estimate the stress ranges in damaged elements. In which case, the damage can be either symmetric or nonsymmetric. The components of damage included in the analysis were: loss of concrete section, loss of prestressing, rotation of principal axes, nonsymmetric strand pattern, and concrete strut. While not completely rigorous (the basis of the procedure is two-dimensional), the procedure results in a rational estimate of strand stress ranges.

11.2 2D_SECT

The procedure to estimate stress ranges in damaged girders uses two small computer programs: 2D_SECT and NEWAXES. The first, 2D_SECT, is used to estimate the planar response of a cross section. The second, NEWAXES, is used to determine the rotation of principal axes of a damaged element. With information regarding the material and cross-sectional properties of an element, strand stress ranges can be estimated.

2D_SECT is a modified version of PLANE. PLANE, a plane section program, is described and provided in Collins and Mitchell’s Prestressed Concrete Basics Text[1]. PLANE was written in IBM PC Basic. It was translated to Pascal and modified to accommodate U.S. Customary units. PLANE is a simple yet powerful tool to estimate the moment-curvature response of a section. The effects of thermal and shrinkage strains as well as strain discontinuity at a composite deck can be included in its analysis. In addition to moment and curvature, the output of 2D_SECT was expanded to include the stress in the bottom layer of strands and the bottom layer of concrete.

The cross section must be discretized into layers for 2D_SECT. Given a top strain, and a resultant axial load on the cross section, the program varies the curvature on the section until the difference
between the computed axial load and the specified axial load on the section is within a specified tolerance. The computed axial load being the sum of the forces in each of the layers of the cross section. The resultant moment is computed about the neutral axis. As a series of top strains are used as inputs, a moment versus curvature plot can be generated for the cross section.

The method is quite general in that multiple concrete and steel materials can be used. The concrete model assumes a parabolic stress-strain behavior, the stress-strain relation of the prestressing strand is modeled with a Ramberg-Osgood relationship, and mild steel is modeled as an elastic perfectly plastic material. See Figure 11.1 for sketches of the material models and the procedure.

An analysis of the test specimens using experimental material properties will be used as an example. The discretization of the test specimens for analysis with 2D_SECT is shown in Figure 11.2. The girder was discretized into 5 rectangular elements and 2 sets of triangular concrete elements. The centerline strand pattern was modeled as four elements, and the top mild steel bars were modeled as a separate element. The composite deck was modeled as a single concrete element and two mild steel elements. The deck was assumed to be unshored, and hence, a strain discontinuity was included in the analysis. Total dead load moment applied to the girder section was 6160 kip-inches before the composite section was assumed effective.

A sample input file is shown in Table 11.1. The input file consists of stress strain information for each of the materials, the geometry of the cross section via the input of layers, and the strain differentials for each of the layers for shrinkage, strain discontinuity, or temperature.

Figure 11.3 shows the output of 2D_SECT. Shown are the responses of both the girder without a deck and the complete composite section. Assuming prestress losses of 65 ksi, the effective stress in the strands was set to 110 ksi under a dead load moment of 6160 kip-inches. For both sections, the response was nearly piecewise-linear. The initial linear region represents behavior of the section before the neutral axis reaches the bottom flange. The latter linear region
corresponds to the response after most of the cross section has cracked.

11.2.1 2D SECT Parameter Runs

2D SECT was run a number of times to examine the influence of various input parameters on strand stress ranges. The influence of concrete tensile strength, composite deck discontinuity, different concrete strengths, and different effective strand stresses were all examined.

The term effective stress in the strands will be used extensively in the remainder of this chapter. Effective stress is defined as the stress in the prestressing strands with the girder subjected to only dead load moment. The dead load moment for the girders at centerline was 6160 kip-inches.

In Figure 11.4, the response of an uncracked specimen is compared with that of a cracked specimen. The difference between the two being that the concrete in the uncracked specimen still possesses tensile strength. The response is linear up to the cracking moment and then jagged as the individual concrete layers reach their rupture stress. After most of the layers have cracked the response is similar to that of the section with no concrete tensile capacity.

To determine the effect of deck strain discontinuity on the response of a girder, another analysis was performed (Figure 11.5). Because the deck is cast at a later date than the girder, there is a strain discontinuity at the deck/girder interface. The stress range increases earlier in the cross section which included the discontinuity. However, the influence was quite small for this particular cross section, steel layout and prestress level. The influence would be larger for sections where the unshored deck load was larger.

The influence of different girder concrete compressive strengths is shown in Figure 11.6. The analysis was performed with both the experimental compressive strength of 8400 psi, and the nominal value of 5000 psi. The influence is small. The lever arm between the tension steel and the centroid of the compression block is relatively
independent of the compressive strength of the concrete. A slightly softer response is shown for the nominal concrete up to crack opening.

Figure 11.7 shows the response of the section with various assumed effective stress levels. Plots of the response with effective stress levels of 110, 130, and 150 ksi under 6160 kip-inches of bending moment are shown. The change in stress range is quite dramatic for moment fields varying between 10,000 kip-inches and 25,000 kip-inches. Whereas the element with an effective stress of 150 ksi would have a stress range of 5 ksi, the element with 130 ksi effective stress would have a stress range of 20 ksi, and lastly, the element with 110 ksi effective stress would have a stress range of approximately 40 ksi. The magnitude of effective stress within an element is of prime importance in determining the stress range within the strands. The larger the effective stress, the larger the prestress on the cross section. A larger prestress requires a larger external moment to produce decompression. Below decompression the stress ranges in the strands are small.

Lastly, as a means of validating 2D_SECT, its output was compared with output published by Overman, et al.[29]. They compared stress ranges for a Texas Type C girder computed with PBEAM (described in Chapter 10) and a hand computation. Their output is compared with that of 2D_SECT in Figure 11.8. The estimated response of the girder from all three methods is identical before decompression and similar thereafter.

11.3 Damaged Girder Analysis

For a damaged element to be analyzed, a number of additional factors need to be included in the procedure. If the damage is symmetric in nature, loss of concrete section and loss of strands can be directly modeled with 2D_SECT. However, if the damage is nonsymmetric with respect to the plane of bending, additional out-of-plane moments need to be investigated. These out-of-plane bending moments result from: rotation of principal axes, concrete strut, and effective prestress. For the procedure, damage is imposed
symmetrically on the element such that the strand stress ranges can be determined using 2D_SECT. The stresses resulting from the out-of-plane moments can then be computed at the critical strand and added to the stress range obtained from 2D_SECT. The critical strand being located where the out-of-plane moments produce the largest tensile stresses at the appropriate level, the generation of moment-strand stress (and if desired moment-curvature) data is simply a task of running the program with a series of top or bottom strains.

To determine the stress ranges from a moment versus strand stress plot, simply find the maximum (live load + dead load) and minimum (dead load) bending moments on the vertical axis and the corresponding strand stresses on the horizontal axis. The difference in the strand stresses is the stress range.

11.3.1 Symmetrically Damaged Girder Analysis

With an estimate of the condition of the member before damage, a reasonable forecast of the response of the damaged member can be made. Figure 11.9 shows the damaged cross section and how this section was discretized for 2D_SECT. The damage consisted of both concrete removal and the severing of two strands on the right side of the bottom flange. For input into 2D_SECT, it was input as symmetric damage.

After the undamaged element was run the next step was to run 2D_SECT with a reduced cross section. The loss of section is input as symmetric damage. Later the nonsymmetric components of the damage will be considered. The loss of cross section includes two damage effects. First, loss of prestress if strands are severed, and second, the loss of concrete section.

While running 2D_SECT for a damaged element, it is important to use the same strand strain differentials which were determined for the undamaged girder. If this is done there is continuity of the strain field.

The initial strand strain differential indicates the amount of potential energy stored in the cross section. For demonstration purposes 5 digits of precision will be used. The stress in the bottom
layer of strands was computed for different amounts of damage but at a constant bending moment of 6160 kip-inches. The following values were obtained:

<table>
<thead>
<tr>
<th>damage condition</th>
<th>strand stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>no damage</td>
<td>110.36 ksi</td>
</tr>
<tr>
<td>concrete removed</td>
<td>109.84 ksi</td>
</tr>
<tr>
<td>1 strand cut</td>
<td>110.10 ksi</td>
</tr>
<tr>
<td>2 strands cut</td>
<td>110.35 ksi</td>
</tr>
</tbody>
</table>

The values are reasonable. When the concrete is removed additional elastic shortening occurs, and the strand stress drops. As the strands are severed, tension steel is lost and more curvature is put into the system to develop an internal moment equal to 6160 kip-inches. The additional curvature increases the effective stress in the remaining strands. The change in effective stress demonstrates that properly used, 2D_SECT is sensitive to cross-sectional damage.

Figure 11.10 shows the output of 2D_SECT for four different situations: 1) undamaged element, 2) loss of concrete, 3) loss of one bottom layer strand (1-cut), and 4) loss of one second layer strand (2-cut). Essentially, the damage induced in the cross section moves the intersection point of the two segments downward (prestress is lost), thus increasing the stress range in the remaining strands.

Figure 11.11 shows a closeup of the knee region of the previous figure. The loss of three percent of the concrete area slightly softens the response of the element up to the decompression moment. Thereafter the effect is negligible. The loss of a single strand, approximately three percent of the total strand area, has a much more pronounced effect on the stress levels. After each strand is severed, prestress is lost and the knee of the curve lowers.

11.3.2 Out-of-plane Moments

After the response of the symmetrically damaged element has been obtained with 2D_SECT the various out-of-plane components are investigated. Figure 11.12 shows qualitatively how the various components interact. The axes for the figure are unique. The
vertical axis represents opening or closing moments on the damaged half of the bottom flange. The horizontal axis represents bending moments applied to the cross section through the major bending axis of an undamaged element. The three individual components, rotation of axes, nonsymmetric strand pattern, and concrete strut are described below.

11.3.2.1 Concrete Strut

The concrete strut component is shown schematically in Figure 11.13. With no live load on the element, the bottom layer of concrete is in compression. The out-of-plane moment is estimated to be approximately equal to the lost concrete area times the compressive stress in the bottom layer times the lever arm of the lost concrete to the undamaged axis of symmetry. As live loads are applied to the specimen, the compressive stress in the bottom layer is reduced and the moment reduces. After decompression the concrete strut is no longer effective. For the example, the bottom layer concrete stress under dead load is 1.15 ksi, the area of the unsymmetrical remaining concrete region is 27 inches$^2$, and the lever arm is 8.3 inches. This results in a moment of 257.7 kip-inches. Assuming the critical strand is 11 inches away from the bending axis, $I_{yy} = 150,320$ inches$^4$, and the strand stress is multiplied by the modular ratio of 6 which results in a stress of 0.11 ksi.

11.3.2.2 Unbalanced Strand Pattern

An opposite effect is created by the unbalanced prestressing strand pattern. Figure 11.14 gives a schematic of this effect. As live loads are applied to the element, the stress in the unbalanced strands increases. The increase in stress times the unbalanced strand area times the lever arm to the undamaged girder axis of symmetry results in another out-of-plane moment. To get an upper bound on this effect the remaining stress capacity of the strands will be used. $250 \text{ ksi} - 110 \text{ ksi} = 140 \text{ ksi}$. $140 \text{ ksi} \times 2 \times 0.144 \text{ inches}^2 = 40.32 \text{ kips}$. $40.32 \text{ kips} \times 9 \text{ inches} = 362.88 \text{ kip-inches}$. Using, $I_{yy}$ of 150,320
inches^4, and a stress location of 11 inches results in a stress of 0.026 ksi. Multiplied by the modular ratio of 6 gives a maximum effect of 0.16 ksi. This is also negligible relative to the precision with which the material parameters are known.

11.3.2.3 Rotation of Principal Axes

The final step was to account for the rotation of the principal axes of the cross section through the decompression moment level. After decompression, the neutral axis shoots up into the web quickly, and from a concrete section perspective, the section should respond symmetrically. To account for the initial effect of axis rotation, the computer program NEWAXES was developed. The fundamental definitions of the moments of inertias are used in the program.

i.e. \( I_{xx} = \text{integral of } y^2 \, dA \)
\( I_{yy} = \text{integral of } x^2 \, dA \)
\( I_{xy} = \text{integral of } xy \, dA \)

With these three quantities known for a specific cross section with respect to a set of centroidal axes, the principal axes and the orientation of the principal axes with respect to the original centroidal axes can be determined from the following (Mohr circle) identities.

\[
\tan 2\theta = -\frac{2I_{xy}}{I_{xx} - I_{yy}}
\]
\[
I_{\text{max}} = \frac{I_{xx} + I_{yy}}{2} + \sqrt{\left(\frac{I_{xx} - I_{yy}}{2}\right)^2 + I_{xy}^2}
\]
\[
I_{\text{min}} = \frac{I_{xx} + I_{yy}}{2} - \sqrt{\left(\frac{I_{xx} - I_{yy}}{2}\right)^2 + I_{xy}^2}
\]

Using these identities the properties of non-symmetric or damaged elements were determined with NEWAXES.

The user discretizes the cross section into rectangular elements, inputs the lower left and upper right coordinates in addition to the modular ratio of the material. Once the cross section is input, the area and the centroid of the section are computed. Subsequently, the inertias of the section are found with respect to centroidal axes which are parallel to the coordinate system in which the elements were entered. In other words, \( I_{xx}, I_{yy}, \) and \( I_{xy} \) are computed. Finally, the principal inertias and the orientation of the principal axes are
determined. The program was verified with a small hand example and a number of problems in Beer and Johnston's statics book[39] for which solutions were given.

The program was then used to determine the orientation of the principal axes for the damaged girder cross section. The program was also run on the composite test specimen to determine the effect of damage. Figure 11.15 shows the discretization of the composite section using only the concrete elements for the program NEWAXES.

The output from NEWAXES is shown in Table 11.2. Interestingly, the principal axes rotated slightly more for the composite section than the girder section alone. The program was run for both sections with two discretizations. The first discretization used considered only the concrete cross section; the effects of reinforcing steel was neglected. The second discretization included the effects of both the mild steel and prestressing strand. The concrete was assumed to be 7000 psi concrete with a modulus of 4770 ksi. Both steels had a modular ratio of approximately 6, 5.9 for the strand and 6.3 for the rebar. In the discretization, the steel areas were given a modular ration of 5. This was done to account for the effect of concrete area overlapping the steel.

Overall, the influence of the damage on the principal axes of the section was quite small, less than three degrees. The sine of 3 degrees is 0.052 or approximately 5 percent. The out-of-plane moment due to rotation of principal axes was 5 percent of the moment between dead load and decompression. It is fairly accurate and conservative to not reduce the major axis bending moment as a result of the rotation of axes. For small angles the moments are relatively uncoupled. Figure 11.16 shows a plot to demonstrate the interaction between sin(theta) and (1-cos(theta)) for small angles. For this reason the rotation of the principal axes effect on the stress ranges was added to the symmetrically damaged value.
or stress = \((\sin(\theta) \times (M_{\text{decompression}} - M_{\text{dead load}})) \times \frac{\text{dist bd axis}}{I_{\text{min}}})

where:

\(\theta\) = rotation of principal axes

\(M_{\text{decompression}}\) = bending moment on the cross section at decompression

\(M_{\text{dead load}}\) = bending moment on the cross section under dead load

\(\text{dist bd axis}\) = distance from strand in question to weak axis

\(I_{\text{min}}\) = weak axis moment of inertia

The computations for rotation of axes show numbers of similar magnitude as those obtained for the concrete strut and nonsymmetric strands. \(M_{\text{decompression}} = 20000\) kip-inches. \(M_{\text{dead load}} = 6160\) kip-inches. The distance to the bending axis was assumed to be 11 inches and \(I_{\text{min}}\) was 141,049 inches\(^4\). Therefore, the stress was equal to \(0.05234 \times 13,840 \times \frac{11}{141,049} = 0.05649\). Multiplying by the modular ratio results in a final stress change of 0.34 ksi. Once again negligible.

Various steel layouts, prestress levels, geometries, and damages will influence the magnitude to which the out-of-plane effects influence stress ranges. They were negligible for the section and damage studied.

### 11.3.3 Comparison of 2D_SECT with Experimental Data

The results of 2D_SECT for the symmetrically damaged girder were compared with data obtained from test girders 3 and 4. Figure 11.17 shows a plot of static tests performed on girders 3 and 4 with concrete removed, 1 strand cut, and 2 strands cut. Shown in Figure 11.18 is the data from Figure 11.17 and numerical data obtained with 2D_SECT for corresponding damage levels. The experimental curves do not show the same distinct knee found in the numerical model. This is to be expected. The experimental data is an average of the strains in
the strands over the entire damage zone. On the other hand, the model estimates strand response at a crack.

The procedure neglects the effect of time-dependent effects and the influence of shear and torsion.

Three programs are included in the appendices: 2D_SECT, NEWAXES, and Extract_life. As mentioned previously, 2D_SECT is a plane section program which outputs the data to generate a moment versus strand stress plot. NEWAXES is a small routine which determines the orientation of principal axes for a nonsymmetrical cross section. Extract_life utilizes the output file created by 2D_SECT to determine strand stress ranges. Given a lower and upper moment, the program linearly interpolates between the closest 2D_SECT data points to determine the strand stress range. In addition to the stress range, also output are the estimated fatigue lives using Paulson's equation and Tide and Van Horn's equation.
Chapter 12 - Prestress Losses

12.1 Introduction

In the following discussion, initial prestress is defined as the stress in the strands when tensioned and effective stress is defined as the stress in the strands under dead load after losses have occurred. Prestress losses are the difference between initial prestress and effective stress. The components of prestress losses are elastic shortening, creep and shrinkage of concrete, and strand relaxation.

The magnitude of prestress losses is an important design parameter. Knowing the magnitude of prestress losses allows one to estimate the cracking load, the load required to subsequently reopen cracked elements, and to estimate strand stress ranges.

The fabrication records indicated that the strands in the girders were individually tensioned to 175 ksi. The prestress loss values were computed with respect to this initial value.

The calculations used to determine the bottom fiber stress of the girder at centerline for various percentages of prestress losses and loads are given in Appendix A.12. Five components of bottom fiber stress were examined. Axial compression from the prestress, internal bending caused by the prestress, external bending from dead load, external bending from live load, and the redistribution of internal stresses caused by shrinkage of the composite deck.

The effective stress is also used to determine the load to produce different "nominal" tensile stresses on the gross cross section.

This chapter describes the procedures used to determine the effective stress within the test girders. The effective stress was initially determined indirectly with girder 1. Later with girders 2 through 4, strands near centerline were exposed and instrumented with strain gages. The output of these strain gages was monitored during static and fatigue loadings. Many were monitored during strand severing.
The indirect prestress loss measurements made on girder 1 assumed the cross section to have no tensile capacity at locations where the girder had cracked. The losses were measured by determining the load at which cracks reopened with crack detection gages. After cracks had formed, instrumentation was installed across the cracks and monitored during subsequent static tests. The output from the crack instrumentation was then plotted versus applied load. Tangents were then drawn to both the initial portion of the curve and to the final portion of the response. The intersection of the two lines was used as an estimate of the crack reopening load. With an estimate of the decompression load, the effective stress in the strands was determined.

The results obtained from the indirect methods used on girder 1 were used to determine the fatigue load levels. Upon examining the data obtained from the strand strain gages installed on girders 2 through 4, it became apparent that the losses were larger than originally computed with the indirect data obtained from girder 1.

The best data was obtained from the strain gages attached directly to the strands. The data from the strand gages included response during static and fatigue loadings. Data was also collected as the strands were severed. The best strand severing data was obtained with the strands cut in girder 4. In girders 2 and 3 the strands were severed quickly. This resulted in snapping of the individual wires of the strand and shock waves sent through the strand. The purpose in quickly severing the strands was to prevent the strain gages from being damaged by heat. For girder 4, a large section of the strand was heated and allowed to become plastic. This eliminated the snapping of individual wires. The strain gages were protected from the heat by wrapping wet rags around the strands between the heated region and the strain gages. Upon completion of severing the strands, the strand near the gages was barely warm to the touch. In addition, better sampling software was developed for girder 4.
12.2 Measurements made with Girder 1

The initial estimates of prestress losses were made with girder 1. Indirect measurements were utilized because girder 1 was used to examine the performance of an undamaged girder. Because the girder had previously been in service, it was not instrumented prior to fabrication. Prestress losses could be estimated by determining the load at which the bottom of the girder cracked, or the load which reopened cracks in a cracked girder. Each of the indirect methods was used to estimate the load which opened or reopened cracks.

12.2.1 Crack Gages

Prior to the initial static test conducted on girder 1, Micro Measurement crack detection gages were installed on the bottom flange of the girder. The gages were staggered to capture a crack in the center ten feet of the bottom flange. The gages were wired in series as a single circuit. The resistance of the circuit was monitored during the first two static tests which were conducted on girder 1. After the first test, G1C1, the resistance of the circuit returned to its pretest value. During the second test the circuit was closely monitored, as the resistance increased to infinity, the damaged gage was located, and a jumper soldered across the gage. This permitted the circuit to be monitored for further cracking as additional load was applied to the girder. Figure 12.1 shows the loads at which the gages were jumpered. The gages failed consistently between 90 and 96 kips. A crack reopening load of 91 kips was used. This corresponded to an effective prestress of 135 ksi.

12.2.2 Crack LVDT

After static test G1C2 had been completed and the cracks produced during the test marked with felt tip markers, LVDTs were placed across crack locations. The LVDTs had a full scale of 0.1 inch. Each LVDT was attached to the girder with a distance between attachment of the core and that of the body of approximately 3 inches. The setup resulted in a system with an accuracy of 10 microstrain averaged over the gage length.
The output of an LVDT placed across the bottom flange during static test G1C3 is shown in Figure 12.2. As described earlier, tangents were drawn to both the upper and lower section of the plot and the intersection of the tangents provided the crack opening load value. The crack opening load was estimated to be 88 kips. The calculated effective stress in the strands was 133 ksi.

12.2.3 Crack Strain Gages

In a manner similar to the crack LVDTs, concrete strain gages were crudely installed across bottom flange cracks. The output of the strain gages was also monitored during static test G1C3. The intersection of the tangents resulted in an estimated crack opening load of 90 kips. The response of two gages is shown in Figure 12.3. The computed effective prestress level was 135 ksi.

12.2.4 Load versus Deflection Plots

As another indication of the remaining prestress in the girder, the overall load deflection plot of the girder was compared between G1C2 and G1C3. During G1C2 extensive cracking was introduced into the girder. After the cracking had been introduced, the girder softened during G1C2. Upon subsequent loading in test G1C3, the load at which the behavior of the two tests diverged gave an estimate of the load at which the cracks reopened. Tests G1C2 and G1C3 are shown in Figure 12.4. The load at which the curves diverged was estimated to be 93 kips. A load of 93 kips corresponds with an effective prestress of 137 ksi.

12.3 Measurements made with Girders 2, 3, and 4.

12.3.1 Strand Response during Static Tests

Strand strain gage data was obtained during static tests which were performed on the girder at different phases of the damaging procedure. The output of the strain gages plotted with respect to the bending moment on the cross section is shown in Figure 12.5. Shown are curves for girders 2, 3, and 4 with 0 strands cut.
At a girder crack, the variation of strand versus bending moment on the cross section is essentially a piecewise linear plot with two segments. One segment up to decompression and a second thereafter. The initial segment, before decompression, is steep. There is little change in strand stress for a large change in bending moment. After decompression, the situation is reversed and the slope is small. A small increase in bending moment corresponded with a large increase in strand stress.

Strand stresses were obtained by multiplying the observed strain by 28,000 ksi. This value, suggested by the PCI handbook, was confirmed with strand tension tests.

In Figure 12.6, the output of the program 2D_SECT is superimposed on the strand strain gage data. Output is shown for effective stresses of 100, 110, and 130 ksi. The output of the gages should be similar to 2D_SECT up to decompression. After decompression the response should differ. 2D_SECT should have a smaller slope than the experimental data. The experimental data was obtained from the girders in damaged regions. Four to six feet of concrete was removed around the four instrumented strands. This produced a situation in which the instrumented strands averaged the strains over both cracks and uncracked regions. 2D_SECT on the other hand, predicted the change in strand stress at a crack location. No averaging, or length of damage zone effects were accounted for in the 2D_SECT analysis.

12.3.2 Strand Severing Data

The strand severing data for girder 2, 3, and 4 is shown in Table 12.1. The best data was obtained from girders 2 and 4. The entire strand pattern in girder 3 was placed 1 inch lower than specified on the design details. Because the strands were placed on 2 inch centers, this resulted in direct hits with the drill as the concrete was removed around the strands. The direct hits nicked up the wires on the strands which were subsequently severed.

The raw data from the strands which were severed in girder 1 was lost. An earlier paper reported the strands lost 105 ksi.
The best severing data was obtained with girder 4. The data acquisition software was its fully developed, and the strand was heated over a sufficient region to prevent shock waves from reaching the gages as the strands were severed. The strand lost 110 ksi.

12.4 Estimate of Effective Prestress with PBEAM

A long-term analysis was performed with the program PBEAM. The time-dependent effects of prestress losses were obtained over a 23 year time period. Within the time period, at 7000 days (19.2 years), the original composite deck was removed, and 500 days later, a new composite deck was added. The output of the analysis resulted in an effective stress of 127 ksi at the end of deck 1, and 126 ksi after behavior had stabilized after deck 2 was added. Prestress losses of 48 ksi correspond with losses of 27.4 percent.

12.5 Summary

The prestress losses measured with the direct measurement of strand strain gages indicate that the effective stress in the strands was approximately 110 ksi. The static test response of the strands in girder 2 indicated that the effective prestress may have been slightly smaller for that particular girder. The strand in girder 2 picked up stress quicker than girder 3 or 4.

The indirect measurements (crack detection gages, crack LVDT, and load versus deflection plots) with tangent intercepts used as crack opening loads estimated the effective strand stress to be 130 ksi. However the indirect measurements are subject to the state of internal stresses in the concrete. The influence of deck shrinkage could change the stress in the bottom of the girder by as much as 200 psi. A 200 psi change in bottom fiber stress, was equivalent to a difference in live load of 11.5 kips for the composite girders tested. A 11.5 kip change in decompression load was equivalent to 10 ksi. The best estimate of remaining strand stress was the data from strand severing. It was relatively independent of the distribution internal concrete stresses. For this reason the effective strand stress was estimated to be 110 ksi.
Chapter 13 - Summary and Conclusions

13.1 Introduction

At the beginning of this report, four questions and three main objectives were presented. These served as the focal point for the research project. This chapter begins by addressing the questions and objectives, after which, each of the previous chapters of this report are summarized. Remarks conclude the chapter by providing topics for further research and a suggested procedure to experimentally measure the effective prestress of existing prestressed elements.

1) What were the prestress losses of these girders?

The fabrication records indicated that the initial tension in the strands was 175 ksi. The effective or remaining stress in the strands (determined experimentally) was 110 ksi. Hence, the prestress losses or the difference between the initial tension and the effective stress was 65 ksi.

The AASHTO Code provides two methods to estimate the prestress losses of a prestressed concrete member. One method is a simple lump sum value to account for all loss components (creep, relaxation, and shrinkage). The second is a detailed method where each of the components are individually estimated. Both methods estimated the losses to be 45 ksi. In the lab, the individual components of prestress loss were not measured. Rather, the effective prestress was determined from the response of the prestressing strands. Near centerline, four strands were exposed and instrumented with strain gages. The instrumented strands were monitored during static and fatigue loadings as well as during strand severing. To verify the output of the strain gages, instrumented strand samples were loaded in a testing machine.

The instrumentation on the exposed girder strands was used to determine the effective stress of the strands with two methods. The first method measured the change in strand strain as a result of being severed. As a strand is cut, all of its effective stress is lost. The change in strain from before the strand is cut, to after the
strand is severed, can be correlated to a change in stress. This change in stress is the remaining or effective stress.

The second method consisted of monitoring the response of the instrumented strands during static and fatigue loadings which had peak loads significantly larger than decompression. The variation of strand stress with applied load is a function of effective stress. The larger the effective stress the smaller the stress range. Knowing the geometry of the cross section and the applied load, an estimate of the effective stress can be made from the observed strain ranges.

2) What were the material properties and the load behavior characteristics of the girders?

The girder materials, strand and concrete, were of 1965 construction. The strand was 250 ksi stress-relieved strand manufactured by Roebling. Strand samples were removed from the girders and pulled in tension tests to determine material properties. The tension tests showed the strand to approximate the stress-strain curve in the PCI handbook for 250 ksi strand (Figure 3.6). For the experimental program, the critical material parameter for the strand was the elastic modulus. From the tension tests, the modulus was found to be approximately 28,000 ksi. The girder concrete was previously examined by Coggins[33]. The estimated compression strength after more than 20 years was 8400 psi. The specified transfer strength was 4500 psi, with an average 1 day strength of 4905 psi.

The initial static load tests performed on the girders indicated that the girders were uncracked when they arrived in the lab. During the initial test sequence for each girder, a load above the cracking load was applied. After the initial load response was measured, the girders were subjected to fatigue programs. For girders 2-4 this was accompanied by a damage or damage/repair procedure. The fatigue programs were conducted at a peak load which corresponded to nominal concrete stresses of 0, 3, 6, and 12√f’c nominal tension (assuming the lump sum prestress losses given by the AASHTO code). All four girders
were subjected to at least one million fatigue load cycles, in some cases, as many as three million cycles.

It is difficult to make any significant conclusions about the load carrying response of the girders during the initial static tests. However, the initial slopes of girders 1-3 were approximately the same, while girder 4 was slightly stiffer. The initial stiffness was approximately 150 kips/inch for girders 1-3, and 195 kips/inch for girder 4.

During fatigue loading, strands failed in girder 2 and girder 3. The fatigue life of prestressing strand is highly dependent on the applied stress range. The larger the stress range, the smaller the fatigue life. The stress range in centerline strands in a girder without strand damage, at the nominal $\sigma f'c$ load level was 22-25 ksi.

The stress range in the strands (for loadings above decompression) is dependent on the level of the remaining prestress. As the prestress level is reduced, the decompression load is reduced. When the applied load is above decompression, the strands alone must develop the internal couple. On the other hand, below decompression, both concrete and strand are developing the internal couple to resist the applied loads. As a result, the lower the prestress level, the lower the decompression load. And the lower the decompression load, the larger the stress range produced for an equivalent applied load.

The AASHTO code does not directly limit strand stress ranges under service loads. Rather, it indirectly limits the strand stress range by placing a limit on the maximum nominal concrete tension allowed. This indirect limit keeps strand stress ranges small when prestress losses are closely estimated, and the nominal tension value is small. The code greatly underestimated the prestress losses in the test girders. As a result, the strands experienced large stress ranges when the applied loading produced a nominal concrete tension of $\sigma f'c$ (the actual bottom fiber tension was much greater due to the lower effective prestress assumed).

The girder tests to failure required the application of large loads and the imposition of large deflections. The control girder (no
impact damage) on a 64 foot span failed at a live load of 293 kips (moment of 53,000 kip-inches) and a midspan deflection of 21 inches.

3) What is the behavior of an impact-damaged girder?

For static load tests, the impact-damage naturally resulted in a region of the girder which was "softer" than the adjacent areas. This region reduced the overall stiffness of the girder. The loss of stiffness meant that more deflection needed to be imposed to resist an equivalent applied load. The more deflection imposed, the further cracks propagated and the larger the stress range of the remaining strands in the damaged region.

The strands in girder 2 showed the following change in stress range as a result of simulated impact-damage. A total of thirty strands were in the undamaged cross section. For each of the damage levels, the total centerline bending moments varied between 7,430 kip-inches and 24,500 kip-inches.

<table>
<thead>
<tr>
<th>Stress Range</th>
<th>0 strands cut</th>
<th>25 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2 strands cut</td>
<td>35 ksi</td>
</tr>
<tr>
<td></td>
<td>3 strands cut</td>
<td>40 ksi</td>
</tr>
</tbody>
</table>

A small amount of strand damage (10%) led to a large increase in strand stress range (60%).

The loss of concrete on the bottom flange has a small impact on the load carrying capacity of a girder. It slightly softens the stiffness of the girder below decompression and its effect is negligible above decompression. The loss of strand has negative effects for both service and ultimate load conditions. In the service load range, the stress ranges increase in the remaining strands and their fatigue life is reduced. At ultimate, the loss of strand reduces the amount of the tension steel in the cross section and results in a reduced flexural strength.

4) How do girder repairs perform under static and fatigue loadings?

Both the strand splice and external post-tensioning repairs stiffened the damaged girders. Neither repair technique, however,
enabled the repaired girders to achieve their predamage stiffness. In repairing the girders, the existing cracks were not repaired. In other words, during the repair procedures, the "tension" stiffness of the girder was not restored at crack locations. As a result of the unrepaired damage, new flexural cracks formed and existing cracks lengthened during retesting the repaired girders.

The internal strand splice did a better job of restoring the response of the undamaged strands to their predamage level than the external post-tensioning repair. One of the reasons the internal strand splice was more efficient was that it restored the prestress to the girder directly at the damaged region, while the post-tensioned repair restored the prestress at the corbel level which was located above the damaged region.

Both repairs were sensitive to fatigue loading. Sensitive in the fact that the first elements to fail or show distress during fatigue loading were a repaired strand for girder 3, and the corbels for girder 4.

The internal strand splice, because of its large axial stiffness, magnified the stress ranges in the "repaired" or spliced strands. This reduced the fatigue life of the "repaired" strands. Debonding the splice from the patch concrete would reduce the observed magnification.

The fatigue problems with the external post-tensioned repair centered on the discontinuity of the cross section where the corbels were attached. The corbels should be located in a region where the concrete does not decompress under service loading. This minimizes the potential for fatigue problems near the corbel and ensures the integrity of the corbel attachment to the girder. The post-tensioning rods were subjected to relatively small stress ranges during the fatigue loading program. As a result, the rods were not the critical fatigue component of the repair. A few of the hairpin bars used to attach the corbels to the girder failed during fatigue loading.

Neither repair performed exceptionally well during the girder ultimate load tests. The internal strand splice had a large flexural stiffness compared to the strand. As a result, the repaired strand
was unable to deform the splice in a flexural manner. This increased the deformations in the repaired strand near the splice causing a relatively early failure.

The external post-tensioning repair also completely failed during the ultimate load test for girder 4. Inclined cracking destroyed the effectiveness of the hairpins to attach the corbels to the girder. When enough cracking had occurred, one of the corbels slid away from the girder, and the repair was lost. Methods to improve both of these techniques are described in Sections 8.8 and 9.8.

The objectives of the project were:

1) To examine the viability of reusing prestressed girders which had been removed from bridges for reasons other than structural integrity.

The question of reusing girders is not a simple one to answer. The major questions are: 1) will the girder perform adequately under service loads, and 2) does the girder possess sufficient overload capacity and ductility. It is not easy to quantify how much of the "fatigue life" was used in the previous installation. It is difficult to make sweeping conclusions regarding the reuse of girders because of the variables of fabrication (materials and practices) and loading histories.

Looking back at the performance of the girders in the experimental program, the following comments can be made:

- The girders were apparently uncracked when they arrived in the lab. If the girders had been cracked when they arrived, they may not have performed as well during the fatigue program.
- The prestress losses were larger than anticipated, 65 ksi instead of the 45 ksi predicted by the AASHTO Code.
- All of the girders withstood severe fatigue programs.
- Small amounts of strand damage severely reduced the fatigue lives of the test girders.
- The repaired girders were sensitive to fatigue loading. The repairs were the fatigue critical elements.
During the ultimate load tests, each girder deflected at least 7 inches before the peak load was attained.

It would appear reasonable to reuse prestressed girders under the following constraints:
- Girders removed from previous installations showing no signs of distress or deterioration.
- Service loads for the new installation are less than decompression. This constraint is specified for three reasons: 1) the uncertainty of the prestress losses, 2) the additional tension caused as a result of a new deck being cast on the girder and shrinking (presumably the strength aging and the shrinkage of the girder concrete has stabilized), and 3) the uncertainty associated with the past loading history of the bridge.

2) To assess the amount of damage that prestressed concrete girders can sustain before major repairs are required.

From a purely structural viewpoint, the question of when repairs are necessary is a function of the loading. The required information is the magnitude and the number of load cycles which the element is expected to withstand. With the loads known, an estimate of the cycles the girder could carry in its current damage state can be made. Quite simply, if this value is less than the number of loads required, repair or replacement is necessary.

The number of cycles the damaged girder can sustain can be determined by following the procedure described in Chapter 11. Basically, the stress range in the strands under service load is determined, and this stress range is put in a strand fatigue model to estimate the number of cycles the damaged element can withstand.

3) To study two repair techniques for restoring prestress to girders which had severed strands due to simulated impact damage.

The repair techniques, (internal strand splices and external post-tensioning) were experimentally examined. Both repairs improved the response of the damaged girders, the stiffnesses were increased
slightly, and more importantly, the stress ranges under service loads in the unsevered strands reduced after the repairs were put in place. However, both repairs were fatigue sensitive and failed relatively early during the girders ultimate load tests.

The repair techniques were simple to design and to install. No special equipment (other than large wrenches) was needed for the internal strand splice, while the external post-tensioning required the use of a standard hydraulic jack to tension the rods.

At the present time, (until more research is conducted on the repairs), the repairs should not be considered effective for ultimate strength calculations. Rather, they should be used to extend the service live of a girder.

Summary Table

A summary of the experimental program is shown in Table 13.1. The table identifies the damage imposed and repair technique (if any) used for each of the four girders. The fatigue loading for each girder is described by the number of cycles at each load level at each damage/repair condition. The final two columns list the peak load and deflection obtained during the flexural test to failure.

13.2 Background

The test series conducted for this research project has been the first to examine impact damage of prestressed concrete girders under static, fatigue, and ultimate loadings. The impact damage implemented with the test girders was nonsymmetric. Previously, Shanafelt and Horn [40] tested a single symmetrically damaged girder under static loads. The nonsymmetrical damage and the fatigue loading employed in this test series, more accurately portrays actual field damage.

13.3 Description of Test Girders

The girders tested were of an AASHTO Type III cross section. The Type III girder cross section is a standard I-shape. It is typically used in bridge construction with a composite deck cast on the girder.

The girders were designed (1964) as fully prestressed members.
Decompression was the largest tensile stress under service loads. The girders were pretensioned with thirty 0.5 inch diameter, 250 ksi, stress-relieved, 7-wire strands. During fabrication the strands were individually tensioned to 175 ksi.

A new composite deck was cast on each girder. The deck was 64 inches wide and 6 inches thick. It was reinforced in the same manner as the bridge deck from which the girders were removed. In the original bridge the girders were spaced at 7 foot centers.

13.4 Test Setup

The girders were 64 feet 8 inches long. The girders were individually tested and loaded with two point loads. The point loads were each 5 feet away from centerline. The girders were supported on a span of 63 feet 5 inches.

13.5 Background of Repairs

Two damage repair schemes were examined as part of the research project. The repairs, an internal strand splice and an external post-tensioning scheme, were previously tested statically[4].

Both repairs were slightly modified from the repairs reported in NCHRP 226 and 280. Two changes were made for the internal strand splice. First, a load cell was incorporated into the turnbuckle. Second, the anchorages (strand chucks) were detailed to be confined by the turnbuckle.

Two changes were also made to the external post-tensioning scheme. First, instead of one high-strength rod, two rods were tensioned between each corbel. Second, because the corbels were cast on only one side of the girder there was no through web anchorage steel.

13.6 Testing of Girder 1

Girder 1 was utilized as the control girder for the test series. No impact damage or repair methods were examined. The loading program used on girder 1 had three phases. Initially a set of static tests were conducted to crack the girder and to subsequently measure the
load at which the cracks reopened. A 2.8 million cycle fatigue loading program was then conducted on the girder. Lastly, the girder was loaded to failure.

The initial static tests indicated that the girder arrived in the lab uncracked. This was determined from load deflection curves. For the first cycle, the initial stiffness was fairly constant up to 130 kips. After 130 kips there was a significant loss of stiffness. This was due to the cracks forming in the bottom flange concrete. As the cracks formed and propagated, the effective moment of inertia reduced, and hence the softening. Subsequent reloadings showed the softening to begin at about 90 kips of live load.

The girder exhibited stable behavior through the first three fatigue loading levels. During the fourth level, range 4, the girder exhibited increasing as the loading progressed. A total of 60,000 cycles of range 4 loading were applied. The fatigue loadings are described in Section 6.3.

A compressive failure of explosive nature was demonstrated during the test to failure. The composite girder failed at a peak load of 293 kips and a midspan deflection of 21 inches.

13.7 Testing of Girder 2

Girder 2 was used as the damage control specimen. After initial static tests and a predamage fatigue program, the damaging sequence commenced.

After each increment of damage was introduced, approximately 500,000 cycles of fatigue loading were applied. This was performed with 0 strands cut and 2 strands cut. Response was unstable with 4 strands cut.

The stress ranges increased significantly in the remaining strands as strands were cut. With 0 strands cut, stress ranges were approximately 25 ksi. With 2 strands cut, under the same load, the stress range increased to 40 ksi.

Girder 2 exhibited a compressive failure at 24 inches of deflection and a peak live load of 209 kips. During the test to failure, the damaged side of the bottom flange developed large cracks
propagating from the damaged region. The large cracks were inclined up into the web and towards the supports. The portion of the flange beyond the web thickness peeled away from the rest of the girder.

13.8 Testing of Girder 3

Girder 3 was utilized as a damaged and repaired specimen. Two internal strand splices were used to repair two severed strands. Initial static tests and a predamage fatigue program were performed before the damage/repair sequence was conducted.

The first static test performed after the first turnbuckle splice was installed significantly lowered the remaining or effective load in the repair. Additional anchorage seating loss was attributed to the drop in load from 18 to 13 kips. Splice 1 was retensioned to 18 kips and subsequently the second splice installed.

The larger axial stiffness of the splice relative to the plain strand produced larger stress ranges in the repaired strands compared to the undamaged strands.

The repair procedure described in NCHRP Reports 226 and 280 used the splice as a fully bonded repair. The patch concrete was allowed to bond to both the turnbuckle and the strand. The repairs tested in girder 3 utilized a bonded splice as well. However, after observing the repair behavior in these tests, it is recommended that the splice be unbonded for two reasons. First it would minimize the stress range in the repaired strand due to the axial stiffness differences. Second, "locking" of the girder would be prevented. In other words, the turnbuckle would not lock cracks together on the repaired side and thereby force cracks to open more on the undamaged side as load is applied.

Repaired strand 1 (with the retensioned repair), failed after 1.3 million cycles of fatigue loading. After which the girder was loaded to failure.

The girder performed adequately during the ultimate strength test. The girder failed in a tensile manner. After reaching a peak load of 238 kips at 7 inches the load dropped with additional displacement imposed on the girder. The tension steel completely
fractured at a deflection of 21 inches. Just prior to failure the applied live load was 100 kips.

The turnbuckles were installed without confining steel. This contributed to a relatively early failure of the splices. After both repairs failed completely at a load of 220 kips, the peeling and large cracks which were observed in girder 2 appeared in girder 3.

13.9 Testing of Girder 4

The last girder was used as a damaged and repaired specimen. An external post-tensioning scheme was used to repair two severed strands.

After initial static tests and a predamage fatigue program, a damage and repair sequence was conducted. The repaired girder withstood 500,000 cycles of range 1, range 2, and range 3 fatigue loadings.

The stress ranges in the post-tensioning rods were significantly smaller than those in the prestressing strands. Two reasons: first, the rods were located higher in the cross section which meant that they were closer to the neutral axis of the section. Second, because the rods were not bonded to the girder throughout their length, the rods experienced average strains over a long length. While the strands experienced larger strains at discrete crack locations.

One of the corbels failed during the ultimate load test. Inclined flexural cracking intersected the dowels which anchored the corbel to the girder, thus enlarging the holes. Because no through web reinforcement was used, the corbel slid away from the girder.

13.10 PBEAM Modeling

The program PBEAM was used to numerically estimate the response of the prestressed girders. Both short-term and long-term (time) analyses were performed.

The short-term analysis included response for stress ranges and cracking load and ultimate failure load. The long-term analysis was conducted to estimate the prestress losses after an extended period of time. Also examined in the long-term analysis was the effect of
adding a new composite deck to a girder which had been in service previously for 20 years.

The stress ranges were found to be piecewise linear with two segments. One segment before decompression, and a second after. The stress range for a particular fatigue loading is highly dependent on the remaining prestress. A smaller effective stress in the strands results in a smaller decompression moment, and therefore, fatigue loadings which may be below decompression for a larger effective stress, may be above decompression for a smaller effective stress. Once the load is above decompression the stress in the strands picks up at a much faster rate.

The long-term analysis estimated the remaining prestress in the strands to be 126 ksi after 20 years. The removal of the first composite deck and the addition of a second deck reduced the remaining prestress by only 1 ksi. The redistribution of internal stresses caused by shrinkage of the second deck, reduced the bottom fiber compressive stress by approximately 75 psi.

13.11 Method to Estimate Stress Ranges

A procedure was developed to estimate the fatigue life of prestressed girders. A plane section program was to output strand stress versus bending moment. Coupling this output with strand fatigue models, results in an estimate of fatigue life. The method was extended to estimate stress ranges in damaged girders.

Five damage components were examined. Loss of concrete, loss of strand, nonsymmetric strand pattern, concrete strut, and rotation of principle axes. The damage component with the most significant impact on strand stress ranges was loss of strand (which also caused loss of prestress).

For the girders examined in the experimental study, nonsymmetric strand pattern, concrete strut, and axes rotation, contributed less than 0.5 ksi for strand stress range. The strand stress versus bending moment diagram is piecewise linear with two segments. The change in strand stress below decompression is governed primarily by the stress strain relation of the concrete. After decompression, the
change in strand stress with bending moment is more pronounced, and more dependent on the geometry of the cross section.

13.12 Prestress Losses

Initial estimates of the remaining or effective prestress in the prestressing strands were conducted indirectly on girder 1. The indirect methods consisted of determining the load at which flexural cracking occurred and the load at which the cracks reopened during subsequent load tests. The initial estimates indicated an effective stress of 130 ksi.

The measurements made with direct instrumentation of the prestressing strands indicated that the effective stress was lower, approximately 110 ksi. The measurements were obtained using strain gages attached to exposed strands. The gages were monitored during the static test response at various levels of damage, lastly at strand severing. From these data, stress ranges were obtained at various levels of damage, and remaining effective prestress was measured.

13.13 Remarks

The results from this test series indicate that the AASHTO code is unconservative with respect to prestress losses. The experimentally measured prestress losses were 65 ksi, 44 percent larger than estimated by the code.

The only accurate way to determine the effective prestress in strands is with direct instrumentation. The response of strands during loading and strand severing is primarily dependent on the effective or remaining stress in the strands. Observed decompression or crack reopening are dependent on the effective stress in the strands, but these are also dependent on the distribution of internal stresses in the concrete. The influence of composite deck shrinkage on the internal stress field, further complicates the matter.

The following procedure should give accurate estimates of effective strand stress.

1. Crack the member.
2. At a crack location, remove only enough concrete to install strain gages. (This minimizes the length over which the strand averages its strain)

3. Monitor the response of the instrumented strands during loading which proceeds from below decompression to significantly above. (The objective is to try to locate the decompression load)

4. After strand response has been measured during loading, remove additional concrete. Remove enough to prevent the strand from being damaged as it is severed. (Additional elastic shortening takes place when concrete is removed from the cross section.)

Impact damage can significantly increase the stress ranges in the remaining strands. The critical parameter for determining strand stress ranges is the amount of prestress on the cross section. Modest amounts of concrete damage have a relatively small impact on strand stress ranges. However, a loss of strand significantly increases the stress range in the remaining strands for two reasons. First, the prestress on the cross section is reduced. Second the area of tension steel is reduced.

Further research should be directed to determine the effective strand stress of other girders which have been in service for an extended period of time.

Research could be conducted to minimize the fatigue sensitivity of the repair schemes. Ideally, the axial stiffness of the internal strand splice should match the stiffness of the strand which it is repairing. This could be accomplished by incorporating spring washers into the splice to reduce its "stiffness".

A model could be developed to analyze the interaction of a corbel with a girder for loadings above decompression. What happens at cracked regions adjacent to the corbel?

NCHRP Report 226 (1980) reported 23,344 prestressed concrete bridges in service and 160 girders were damaged each year as a result of overheight vehicle impact damage. The number of damaged girders will most likely increase in the future. Currently, half of the new bridge construction in the U.S. is prestressed (approximately 2500 per year). The results form a basis from which further research can more
fully investigate prestress losses, overheight vehicle impact damage, and repair schemes.
REFERENCES


5. Personal communication with Matt Lang, Minnesota Department of Transportation, January 1988.


Tables
## Girder Section Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area</td>
<td>559.5</td>
<td>inches²</td>
</tr>
<tr>
<td>Inertia</td>
<td>125,400</td>
<td>inches⁴</td>
</tr>
<tr>
<td>Modulus</td>
<td>4,031</td>
<td>ksi</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>5,000</td>
<td>psi</td>
</tr>
<tr>
<td>$Y_{bottom}$</td>
<td>20.27</td>
<td>inches</td>
</tr>
<tr>
<td>$S_{bottom}$</td>
<td>6,186</td>
<td>inches³</td>
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</table>
Nondestructive Test Results

<table>
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<tr>
<th>Test</th>
<th>Average</th>
<th>Coeff. of Var.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Schmidt</td>
<td>7,960 psi</td>
<td>0.042</td>
</tr>
<tr>
<td>Windsor Probe</td>
<td>6,500 psi</td>
<td>0.102</td>
</tr>
<tr>
<td>Pulse Velocity</td>
<td>10,700 psi</td>
<td>0.053</td>
</tr>
<tr>
<td>Break-off</td>
<td>3,704 psi</td>
<td>0.215</td>
</tr>
<tr>
<td>4 inch cores</td>
<td>8,615 psi</td>
<td>0.058</td>
</tr>
<tr>
<td>2 inch cores</td>
<td>8,147 psi</td>
<td>0.074</td>
</tr>
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</table>
Composite Deck Concrete Mix

MnDOT Mix Number 3X46

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
<th>Unit</th>
</tr>
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<tbody>
<tr>
<td>Type I Cement</td>
<td>732</td>
<td>pounds</td>
</tr>
<tr>
<td>Sand</td>
<td>1,266</td>
<td>pounds</td>
</tr>
<tr>
<td>3/4&quot; Rock</td>
<td>1,583</td>
<td>pounds</td>
</tr>
<tr>
<td>Water</td>
<td>307</td>
<td>pounds</td>
</tr>
<tr>
<td>Air entrainment</td>
<td>5.5</td>
<td>percent</td>
</tr>
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</table>

water/cement = 0.42
**Composite Section Properties**

3000 psi deck / 5000 psi girder

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area</td>
<td>852.3</td>
<td>inches(^2)</td>
</tr>
<tr>
<td>Inertia</td>
<td>275,800</td>
<td>inches(^4)</td>
</tr>
<tr>
<td>Modulus</td>
<td>3,100</td>
<td>ksi (deck)</td>
</tr>
<tr>
<td>(f'_c)</td>
<td>3,000</td>
<td>psi (deck)</td>
</tr>
<tr>
<td>Weight</td>
<td>983</td>
<td>lb/ft</td>
</tr>
<tr>
<td>(Y_{\text{top}})</td>
<td>21.19</td>
<td>inches</td>
</tr>
<tr>
<td>(Y_{\text{bottom}})</td>
<td>29.81</td>
<td>inches</td>
</tr>
<tr>
<td>(Y_{\text{top-gird}})</td>
<td>15.19</td>
<td>inches</td>
</tr>
<tr>
<td>(S_{\text{bottom}})</td>
<td>9,252</td>
<td>inches(^3)</td>
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## Corbel / Patch Concrete Mix

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Amount</th>
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<tbody>
<tr>
<td>Cement</td>
<td>100 lbs</td>
</tr>
<tr>
<td>Sand</td>
<td>175 lbs</td>
</tr>
<tr>
<td>Pea Rock</td>
<td>225 lbs</td>
</tr>
<tr>
<td>High Range Water Reducer</td>
<td>16 oz</td>
</tr>
</tbody>
</table>

Water to produce a slump between 2 and 5 inches
## Load Tests Conducted

<table>
<thead>
<tr>
<th>Test</th>
<th>Label</th>
<th>Fat. Cycles</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>G1C1</td>
<td>0</td>
<td>initial cracking</td>
</tr>
<tr>
<td>2</td>
<td>G1C2</td>
<td>0</td>
<td>recracking test</td>
</tr>
<tr>
<td>3</td>
<td>G1C3</td>
<td>0</td>
<td>cracks instrumented</td>
</tr>
<tr>
<td>4</td>
<td>G1C4</td>
<td>0</td>
<td>prior to fatigue</td>
</tr>
<tr>
<td>5</td>
<td>G1F1</td>
<td>447,000</td>
<td>47,000 - range 2</td>
</tr>
<tr>
<td>6</td>
<td>G1F2</td>
<td>973,000</td>
<td>573,000 - range 2</td>
</tr>
<tr>
<td>7</td>
<td>G1F3</td>
<td>1,083,000</td>
<td>cracks in slab</td>
</tr>
<tr>
<td>8</td>
<td>G1F4</td>
<td>1,292,000</td>
<td>static check</td>
</tr>
<tr>
<td>9</td>
<td>G1F5</td>
<td>1,400,000</td>
<td>range 2/range 3</td>
</tr>
<tr>
<td>10</td>
<td>G1F6</td>
<td>1,451,000</td>
<td>static check</td>
</tr>
<tr>
<td>11</td>
<td>G1F7</td>
<td>1,742,000</td>
<td>static check</td>
</tr>
<tr>
<td>12</td>
<td>G1F8</td>
<td>1,885,000</td>
<td>static check</td>
</tr>
<tr>
<td>13</td>
<td>G1F9</td>
<td>2,321,000</td>
<td>static check</td>
</tr>
<tr>
<td>14</td>
<td>G1F10</td>
<td>2,486,000</td>
<td>static check</td>
</tr>
<tr>
<td>15</td>
<td>G1F11</td>
<td>2,710,000</td>
<td>range 3/range 4</td>
</tr>
<tr>
<td>16</td>
<td>G1F12</td>
<td>2,770,000</td>
<td>end range 4</td>
</tr>
<tr>
<td>17</td>
<td>G1U1</td>
<td>2,770,000</td>
<td>load to failure</td>
</tr>
</tbody>
</table>

Load Tests Conducted
Table 6.1

- 136 -
### Test/Damage Sequence - Girder 2

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Cracking Static Tests</td>
</tr>
<tr>
<td>2.</td>
<td>Predamage Fatigue Program</td>
</tr>
<tr>
<td>3.</td>
<td>Static Test</td>
</tr>
<tr>
<td>4.</td>
<td>Expose Four Strands</td>
</tr>
<tr>
<td>5.</td>
<td>Static Test</td>
</tr>
<tr>
<td>6.</td>
<td>500,000 cycles (0 cut damage level)</td>
</tr>
<tr>
<td>7.</td>
<td>Static Test</td>
</tr>
<tr>
<td>8.</td>
<td>Sever Strands 1 and 2</td>
</tr>
<tr>
<td>9.</td>
<td>Static Test</td>
</tr>
<tr>
<td>10.</td>
<td>300,000 cycles - Strand 3 broke</td>
</tr>
<tr>
<td>11.</td>
<td>Static Test</td>
</tr>
<tr>
<td>12.</td>
<td>200,000 cycles (finish 500,000 cycles at 2 cut damage level)</td>
</tr>
<tr>
<td>13.</td>
<td>Static Test</td>
</tr>
<tr>
<td>14.</td>
<td>Sever Strand 4</td>
</tr>
<tr>
<td>15.</td>
<td>Static Test</td>
</tr>
<tr>
<td>16.</td>
<td>Fatigue loading unstable</td>
</tr>
<tr>
<td>17.</td>
<td>Static Test</td>
</tr>
<tr>
<td>18.</td>
<td>Test to Failure</td>
</tr>
</tbody>
</table>

---

Test/Damage Sequence - Girder 2

Table 7.1

- 137 -
Strand Severing Data - Girder 2

<table>
<thead>
<tr>
<th>Strain Gage</th>
<th>1a</th>
<th>1b</th>
<th>2a</th>
<th>2b</th>
<th>3</th>
<th>4</th>
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</thead>
<tbody>
<tr>
<td>init. read.</td>
<td>3</td>
<td>5</td>
<td>0</td>
<td>-3</td>
<td>x</td>
<td>0</td>
</tr>
<tr>
<td>final read.</td>
<td>-3316</td>
<td>-2960</td>
<td>-4287</td>
<td>-3818</td>
<td>x</td>
<td>-4164</td>
</tr>
<tr>
<td>stress change</td>
<td>93</td>
<td>83</td>
<td>120</td>
<td>107</td>
<td>x</td>
<td>117</td>
</tr>
</tbody>
</table>

units: strain gages - microstrain, stresses - ksi
<table>
<thead>
<tr>
<th>Step</th>
<th>Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Static test (end of predamage fatigue loading)</td>
</tr>
<tr>
<td>2</td>
<td>Expose 4 strands</td>
</tr>
<tr>
<td>3</td>
<td>Instrument exposed strands with strain gages</td>
</tr>
<tr>
<td>4</td>
<td>Static test/strain ranges in strands</td>
</tr>
<tr>
<td>5</td>
<td>Sever strand 2</td>
</tr>
<tr>
<td>6</td>
<td>Static test/strain ranges in strands</td>
</tr>
<tr>
<td>7</td>
<td>Sever strand 1</td>
</tr>
<tr>
<td>8</td>
<td>Static test/strain ranges in strands</td>
</tr>
<tr>
<td>9</td>
<td>Repair strand 1</td>
</tr>
<tr>
<td>10</td>
<td>Static test/strain ranges in strands</td>
</tr>
<tr>
<td>11</td>
<td>Retighten strand 1 splice</td>
</tr>
<tr>
<td>12</td>
<td>Repair strand 2</td>
</tr>
<tr>
<td>13</td>
<td>Static test/strain ranges in strands</td>
</tr>
<tr>
<td>14</td>
<td>Apply preload/replace missing concrete</td>
</tr>
<tr>
<td>15</td>
<td>Static test/strain ranges in strands</td>
</tr>
</tbody>
</table>
### Damage/Repair Sequence - Girder 4

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Static test (end of predamage fatigue loading)</td>
</tr>
<tr>
<td>2.</td>
<td>Expose 4 strands</td>
</tr>
<tr>
<td>3.</td>
<td>Instrument exposed strands with strain gages</td>
</tr>
<tr>
<td>4.</td>
<td>Static test/strain ranges in strands</td>
</tr>
<tr>
<td>5.</td>
<td>Sever strand 2</td>
</tr>
<tr>
<td>6.</td>
<td>Static test/strain ranges in strands</td>
</tr>
<tr>
<td>7.</td>
<td>Sever strand 1</td>
</tr>
<tr>
<td>8.</td>
<td>Static test/strain ranges in strands</td>
</tr>
<tr>
<td>9.</td>
<td>Tension rods between corbels</td>
</tr>
<tr>
<td>10.</td>
<td>Static test/strain ranges in strands</td>
</tr>
<tr>
<td>11.</td>
<td>Apply preload/replace missing concrete</td>
</tr>
<tr>
<td>12.</td>
<td>Static test/strain ranges in strands</td>
</tr>
</tbody>
</table>

---

**Damage/Repair Sequence - Girder 4**

Table 9.1

- 140 -
<table>
<thead>
<tr>
<th>material phi factors</th>
<th>number of concretes</th>
</tr>
</thead>
<tbody>
<tr>
<td>concrete #1's properties</td>
<td>concrete #2's properties</td>
</tr>
<tr>
<td>number of mild steels</td>
<td>mild steel #1's properties</td>
</tr>
<tr>
<td>number of strand steels</td>
<td>strand #1's properties</td>
</tr>
<tr>
<td>depth, n.a., #conc. elements</td>
<td>concrete elements</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>number of mild steel elements</th>
<th>mild steel elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>bottom,area,material</td>
<td></td>
</tr>
</tbody>
</table>

| number of strand elements | bottom,area,strain differential,mat. |

<table>
<thead>
<tr>
<th>thermal strain indicator</th>
<th>initial strain indicator</th>
</tr>
</thead>
<tbody>
<tr>
<td>conc. top &amp; bot. initial strains</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>mild steel initial strains</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>strand initial strains</th>
</tr>
</thead>
</table>

Sample Input File for Composite Section 2D_SECT
Table 11.1

- 141 -
## Output of program "Newaxes.pas"

### Parameters:
- **Date:** July 11, 1990

#### Table: Element Properties

<table>
<thead>
<tr>
<th>Element</th>
<th>Area</th>
<th>x Centr</th>
<th>y Centr</th>
<th>Imax</th>
<th>Imin</th>
<th>Rot. dg</th>
<th>Ixx</th>
<th>Iyy</th>
<th>Ixy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undamaged girder - concrete only</td>
<td>559.5</td>
<td>0</td>
<td>20.284</td>
<td>125189</td>
<td>12150</td>
<td>0</td>
<td>125189</td>
<td>12150</td>
<td>0</td>
</tr>
<tr>
<td>Undamaged girder - concrete/steel</td>
<td>601.2</td>
<td>0</td>
<td>20.425</td>
<td>140334</td>
<td>13255</td>
<td>0</td>
<td>140334</td>
<td>13255</td>
<td>0</td>
</tr>
<tr>
<td>Damaged girder - concrete only</td>
<td>532.5</td>
<td>-0.403</td>
<td>21.209</td>
<td>115874</td>
<td>10093</td>
<td>-2.217</td>
<td>115716</td>
<td>10251</td>
<td>4085</td>
</tr>
<tr>
<td>Damaged girder - concrete/steel</td>
<td>571.3</td>
<td>-0.413</td>
<td>21.384</td>
<td>129894</td>
<td>11001</td>
<td>-2.183</td>
<td>129721</td>
<td>11174</td>
<td>4529</td>
</tr>
<tr>
<td>Undamaged composite - concrete only</td>
<td>943.5</td>
<td>0</td>
<td>31.565</td>
<td>301261</td>
<td>143222</td>
<td>0</td>
<td>301261</td>
<td>143222</td>
<td>0</td>
</tr>
<tr>
<td>Undamaged composite - concrete/steel</td>
<td>999.5</td>
<td>0</td>
<td>31.401</td>
<td>323229</td>
<td>150320</td>
<td>0</td>
<td>323229</td>
<td>150320</td>
<td>0</td>
</tr>
<tr>
<td>Damaged composite - concrete only</td>
<td>916.5</td>
<td>-0.234</td>
<td>32.434</td>
<td>277320</td>
<td>141049</td>
<td>-2.733</td>
<td>277010</td>
<td>141359</td>
<td>6493</td>
</tr>
<tr>
<td>Damaged composite - concrete/steel</td>
<td>969.7</td>
<td>-0.244</td>
<td>32.304</td>
<td>297065</td>
<td>147939</td>
<td>-2.733</td>
<td>296725</td>
<td>148279</td>
<td>7108</td>
</tr>
</tbody>
</table>
### Strand Severing Data (average)

<table>
<thead>
<tr>
<th>Effective strand stress (ksi)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder 1</td>
<td>105</td>
</tr>
<tr>
<td>Girder 2</td>
<td>114</td>
</tr>
<tr>
<td>Girder 3</td>
<td>111</td>
</tr>
<tr>
<td>Girder 4</td>
<td>110</td>
</tr>
</tbody>
</table>

Strand Severing Data - Girders 1 through 4
Table 12.1
<table>
<thead>
<tr>
<th>Impact Damage</th>
<th>Repair Method</th>
<th>Fatigue Load Cycles</th>
<th>Total Cycles</th>
<th>Ultimate Load Test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>range 1</td>
<td>range 2</td>
<td>range 3</td>
</tr>
<tr>
<td>Girder 1</td>
<td>NONE</td>
<td>1-7,430</td>
<td>1-7,430</td>
<td>1-7,430</td>
</tr>
<tr>
<td></td>
<td>NONE</td>
<td>2-7,430</td>
<td>2-7,430</td>
<td>2-7,430</td>
</tr>
<tr>
<td></td>
<td>NONE</td>
<td>3-7,430</td>
<td>3-7,430</td>
<td>3-7,430</td>
</tr>
<tr>
<td></td>
<td>NONE</td>
<td>4-7,430</td>
<td>4-7,430</td>
<td>4-7,430</td>
</tr>
</tbody>
</table>

Centerline Bending Moments (including dead load)
range 1 - 7,430 to 20,500 kip-inches
range 2 - 7,430 to 22,500 kip-inches
range 3 - 7,430 to 24,500 kip-inches
range 4 - 7,430 to 26,500 kip-inches

AASHTO Type III cross section with a 60 inch wide by 6 inch thick composite deck
Figures
adapted from Collins & Mitchell

<table>
<thead>
<tr>
<th>Conditions at Midspan</th>
<th>Strains</th>
<th>Stresses</th>
<th>Stress Resultants</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Non-prestressed</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No external loads</td>
<td></td>
<td></td>
<td>H = 0</td>
</tr>
<tr>
<td>Just prior to cracking</td>
<td>0.0001</td>
<td>1.16 ksi</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.003</td>
<td>0.29 ksi</td>
<td></td>
</tr>
<tr>
<td>Just prior to failure</td>
<td>-0.003</td>
<td>0.59 ksi</td>
<td></td>
</tr>
<tr>
<td><strong>Prestressed</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No external loads</td>
<td></td>
<td>174 ksi</td>
<td>P, P, M = 0, M = 0</td>
</tr>
<tr>
<td>Just prior to cracking</td>
<td>0.0001</td>
<td>180 ksi</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.003</td>
<td>201 ksi</td>
<td></td>
</tr>
</tbody>
</table>

Prestressed Concrete versus Nonprestressed adapted from Collins and Mitchell[1]
Figure 1.1
Breakdown of Sections Used
reported in NCHRP 226

23,344 Girders in service

- I-sections (54.0%)
- Box sections (26.0%)
- Slab sections (8.0%)
- T sections (12.0%)
Prestressed Concrete Bridge Construction in Minnesota

Figure 1.3
Bridge Construction 1954-1989
State of Minnesota

Cumulative Built

Year


Prestressed

Total
Breakdown of Damages reported in NCHRP 226

- Overheight Loads (80.6%)
- Fire (2.5%)
- Other (16.9%)
Hognestad Concrete Stress-Strain Model
Figure 2.1
Fatigue Design Figure for Concrete
ACI Committee 215

Minimum Stress as a Percent of Static Strength

Maximum Stress as a Percent of Static Strength

one million cycles
Strength Aging of Concrete

ACI Committee 209

Figure 2.3
Suggested Shrinkage Strains
Prestressed Concrete Institute
Figure 2.4

- 154 -
ACI 209 - Basic Creep Curve

Basic Creep Curve
ACI Committee 209
Figure 2.5
MILD STEEL
Grades 40, 60, & 75

STRESS (ksi)

STRAIN

Elastic-Plastic Model for Mild Steel
Figure 2.6
Mild Steel Fatigue Test Data
ACI Committee 215
Figure 2.7
Prestressing Strand Stress-Strain Curves
Prestressed Concrete Institute
Figure 2.8
Paulson Strand Fatigue Design Curve

Log N = 11.0 - 3.5 * Log Sr

Number of Cycles

Strand Stress Range - ksi

Paulson Fatigue Design Curve for Strand
Figure 2.10

- 160 -
Relaxation Curves
Stress-relieved and Low-relaxation

Relaxation Curves for Prestressing Strand
Figure 2.11
Girder on a Truck
Figure 3.1
Girder being Lowered into the Lab
Figure 3.2

- 163 -
Test Girder Cross Section

Test Specimen
AASHTO Type 3 Girder
Conventional Concrete Deck
Length - Out-Out = 64'-8"

Units are inches
Strand Pattern
Figure 3.4

- 165 -
Composite Deck Reinforcement

# 4 Hairpins

Units are Inches
All Bars Grade 60
Transverse Bars No. 5's
Top Longitudinal Bars No. 4's
Bottom Longitudinal Bars No. 5's
Figure 4.1
Cross Section of Loading Frame

LOAD FRAME

- Cross Bracing Rods
- Test Stubs

- Columns
- Tie Rods
- Actuator Tube
- Actuator
- Actuator Restraining Beam
- Loading Tube
- Test Specimen
- Strong Floor

Actuator Bracket
Elevation of Girder with Instrumentation
Figure 4.2

- 169 -
Bar gages are labelled from West to East and are on three inch centers.

Bottom longitudinal bars rest on #5 transverse bars which sit on 1-inch chairs.

Top longitudinal bars rest on 3.5' chairs and are #4 bars.

Girder 4 Deck Instrumentation
Figure 4.3
Figure 5.1 - Splice No. 7

**Splice No. 7 (Cont.)**

Working strength per strand =
\[ 0.7 \left( 270,000 - 25,000 \right) (0.153) = 22,000 \text{ psi/strand} \]

Splicing rod = 1 in. threaded rod ASTM A722 Grade 150, f_s = 150,000 psi

\[ f_s = 0.551 \text{ sq. in.} \]

Allowable = 0.6 (150,000) = 90,000 psi

40,000 psi required

Ultimate strength of 1/2 in. 210K strand = 270,000 (0.153) = 41,300 lb.

Ultimate strength of splicing rod = 150,000 (0.551) = 82,600 lb

1/2 in. ultimate load = 41,300/0.551 = 75,000 psi

75,000 psi = 150,000 psi.

---

**ELEVATION**

Splice No. 7

2 in. strand spacing

1 in. high strength steel rod & splice

Face of concrete

Existing strand grip

See Detail "A"

Right hand thread

Left hand thread

Octagonal high strength steel splice sleeve

1/8 in.

Flattened two sides for gripping

Check or equal

Baill

Special splice

Threaded inside for rod coupling

Alternate strand grip

(1 inch = 25.4 mm)

(1 foot = 0.305 m)
To Assemble and Tension Splice:
1. Place barrels of splice chucks on strands.
2. Screw one steel splice onto threaded coupling.
3. Screw one rod into threaded coupling to bottom out.
4. Screw other steel splice approximately 2½" onto other rod.
5. Then screw assembly 4, one inch into turnbuckle.
6. Then back steel splice off rod while screwing onto the other threaded coupling.
7. Tension splice by turning the turnbuckle approximately 2-5° prior to torquing.

**Completed Splice**

**Threaded Coupling**

Hexagonal Sleeves, σ₃ = 150 ksi

6 Threads Engaged Prior to Torquing

Threaded to Stops

1" Heat Treated Rods 6½" Long

ASTM A322: f₅ = 127 ksi, f₇ = 110 ksi

**Splice**

Threaded Rod

Supreme Splice Chuck or Approved Equal

Barrel

**Wedge Assembly**

Hexagonal High Strength Steel Splice (Inside Threads for Rod and Coupling)

Springs

Spring Cap

**Threaded Coupling**

**Note:**

Vary rod lengths as necessary for required splice length.

**Strand Grip Details**
Figure 5.4
Tensioning of Internal Strand Splices

Strand Stress Change (ksi)

Unsevered Strands and Turnbuckles
NCHRP 226 External Post-tensioning Detail
Figure 5.5

175
Splice No. 2

$f_{c'} = 5000$ psi. $E_{c'}$ assumed to be $0.8 \times E_{c'}$.

Section properties Girder: $A = 789$ in$^2$, $I = 260,700$ in$^4$, $S_{ax} = 10,540$ in$^3$.

Section Properties Girder and Slab: $A = 1257$ in$^2$, $I = 260,700$ in$^4$, $S_{ax} = 15,570$ in$^3$.

Live Load: Assume Span Length to be 85 ft.; Girder Spacing = 7.5 ft.

Moment (AASHTO 1977) = 1255 Kip-ft. w/o Imp.; Distribution = 5/5.5 = 1.36; $M_{u,li} = 1058$ Kip-ft. HS 20

Assume 4 severed strands in bottom layer, working stress of severed strands = $0.7(270,000) - 45,000 = 88$ kips

$P_{u} = P/A + M/S = 88,000/789 + 88,000(22.23)/15,570 = 111 + 186 = 297$ psi.

Prestress Loss

$P_{u} = 297 - 2$ - in. $= 275$ psi. $= 2$ - in. Grade 160 rods (smooth)

Use approximate working load = 0.6 $f_{c'} = 0.6 (160)$ = 96 ksi

Assume conduit supports @ 10 ft. centers, bending stress = 5.5 ksi.

$M_{u,li} = 1255(1/1000)/15,570 = 815$ psi.

Prestress Gain

$P_{u} = 297 - 2$ in. $= 275$ psi. $= 2$ - in. Grade 160 rods will replace four severed 1/2 in. strands. Strands are assumed to be in bottom layer.

Calculate approximate ultimate strength:

$a$ = depth of equivalent compression zone; $a = As f_{y}/0.85 f'c$, $f'c = 4,000$ psi, $b = 90$ in.

As 2 - in. $f_{c'} = 1.57$ in$^2$; $As = 1.57(160)(0.85)$ = 213 kips

Assume 34 strands originally, with 4 severed. Area Strands = $30(0.153)$ = 4.59 in$^2$

$As f_{y} = 4.59(270)(0.85)$ = 1053 kips

$M_{u,li} = 213 (41.5)/12 = 736$ kip-ft.

$M_{u,li}$ = $1053 (52)/12 = 4563$ kip-ft.

Total = 5299 kip-ft. Approx.

$D_{l,li} = 822$ lb./ft., $D_{l,li} = 650$ lb./ft.; $M_{l,li} = 1329$ kip-ft.;

$M_{u,li}$ required = 1.3 $(D_{l} + 5/3 LL + g) = 1.3 (1329 + 5/3(1058)) = 4020$ kip-ft.

Ultimate strength of splice is adequate.

Calculate Corbel reinforcing:

Area bearing plate = 41 in$^2$; Area corbel = 67 in$^2$.

Pu = 0.785 (160)(0.95) = 119 kips (See AASHTO Art. 1.6.17)

Working load per corbel = $142/2 = 71$ kips

$fc_{u,li} = 71/41 = 1730$ psi under plate; $fc_{u,li} = 3000$ psi (See AASHTO Art. 1.6.6)

Bearing on corbel at Pu = $119/67 = 1780$ psi (Safe)

Calculate ties required:

$A_{v} = Vu/f_{y}$, AASHTO Art. 1.5.35; $\mu = 1.2$, concrete interface to be roughened.

$A_{v} = 119,000(0.85) (40,000) (1.2) = 2.8$ in$^2$.

Strength Required (at yield) = 2.9(40,000) = 116 kips

5 - #5 ties = $5 \times 0.31 \times 40 = 62$ kips

Expansion Bolts, 5/8 $. Use 80 % of allowable for reduced spacing in one direction.

$10 \times 7000$ lb. x 0.80 = 56 kips

Total ea. side = 118 kips $\geq 116$ kips

Use Grade 60 #5 ties for added strength across interface.

Also check shear along interface per AASHTO Art. 1.5.35E; $vu = allowable = 350$ psi.; $116,000/12(48) = 201$ psi $\leq 350$ psi.

NCHRP 226 External Post-tensioning Detail Calculations

Figure 5.6 - 176 -
NCHRP 280 External Post-tensioning Detail
Figure 5.7

- 177 -
Figure 5.8

EXTERNAL POST-TENSIONED REPAIR

damage

high-strength rods

corbel
Tensioning of High-Strength Rods

Figure 5.9

Tensioning of Post-tensioning Rods

- 179 -
Initial Cracking Tests
Girder 1

Figure 6.1
Fatigue Loading - Peak Live Load
Girder 1
Figure 6.2
Fatigue Loading - Actuator Deflection Span
Girder 1
Figure 6.3
AASHTO Code Allowable Stresses

Nominal Concrete Stresses

First U.S. Prestressed Bridge

First Minnesota Prestressed Bridge

Year

Intermediate Static Tests
Girder 1

Figure 6.5
Ultimate Load Test
Girder 1

Figure 6.6
Average Jack versus Centerline Deflection
Ultimate Load Test Girder 1
Figure 6.7
Ultimate Load Test Crack Patterns
Girder 1
Figure 6.8
Initial Cracking Test
Girder 2

![Graph showing net deflection vs. total live load for Girder 2.](image-url)

Initial Cracking Test
Girder 2
Figure 7.1

- 188 -
Initial Cracking Tests
Girders 1 and 2

Initial Cracking Test Comparison
Girders 1 and 2
Figure 7.2
Concrete Encasing Removed
4 Strands Exposed

Two Exposed Strands Severed

Remaining Exposed Strands Severed

Damaging Scheme
Girder 2
Figure 7.3
Fatigue Loading - Peak Live Load
Girder 2
Figure 7.4
Fatigue Loading - Girder 2

Cumulative Cycles (Millions)

Actuator Span (inches)

- Range 2
- Range 3
- 3rd Strand Broke
- 4th Strand Cut
- 2 Strands Cut
- Concrete Removed

Fatigue Loading - Actuator Deflection Span

Figure 7.5

- 192 -
Figure 7.6

Predamage Static Tests
Girder 2

Total Live Load (kips)

Net Deflection (inches)
Intermediate Static Tests
Girder 2

Figure 7.7

Intermediate Static Tests
Girder 2
Figure 7.7

- 194 -
Predamage Static Tests
Girders 1 and 2

Figure 7.8
Instrumented Strands
Figure 7.9
Girder 2, Strand Response
Static Test Prior to Severing

![Graph showing the change in strand stress vs total live load for Girder 2. The graph includes two lines representing the 2nd layer and the bottom layer.](image)
Stress Ranges in Strands 3 and 4
Girder 2
Figure 7.11
Ultimate Load Test
Girder 2

Figure 7.12
Ultimate Load Tests
Girders 1 and 2

Figure 7.13
Cracking Test
Girder 3
Figure 8.1
Cracking Tests
Girders 1, 2, and 3
Figure 8.2
Turnbuckle Splices Installed
Figure 8.3

- 203 -
Turnbuckle Splice Repair Completed
Figure 8.4

- 204 -
Fatigue Loading - Actuator Deflection Span
Girder 3
Figure 8.6
Intermediate Static Tests
Girder 3
Figure 8.7
Girder 3
Predamage Static Tests

Predamage Static Tests
Girder 3
Figure 8.8
Predamage Static Test Comparison
Girders 1 and 3
Figure 8.9

- 209 -
Girder 3
Damage Static Tests

Figure 8.10

Girder 3
Damage Static Tests

Net Deflection (inches)

Total Live Load (kips)

1. No damage
2. 0 cut
3. 1 cut
4. 2 cut

Damage Static Tests
Girder 3
Figure 8.10

- 210 -
Repair Static Tests
Girder 3
Figure 8.11

- 211 -
Girder 3
Static Tests during Fatigue Loading

Net Deflection (inches)

Total Live Load (kips)

1. No damage
2. Patched
3. end range 1
4. end range 2
5. end range 3

Static Tests During Fatigue Loading
Girder 3
Figure 8.12
Girder 3
Strand Strain Gage Layout

Strand Strain Gage Layout
Girder 3
Figure 8.13

- 213 -
Girder 3
Strand 3 Response

Static Test Response of Strand 3
Girder 3
Figure 8.14
Static Test Response of Strand 4
Girder 3
Figure 8.15
Girder 3
Unsevered Strands - Range 3

Stress Ranges in Strands 3 and 4
Figure 8.16

- no concrete
- 1 strand cut
- 2 strands cut
- 1 repair
- 2 repair
- patch concrete

Stress Range (ksi)

Strand Number

3 4
Girder 3
Stress Ranges in Unsevered Strands

Stress Ranges in Strand Gages 9 - 14

Figure 8.17

<table>
<thead>
<tr>
<th>Gage Number</th>
<th>Stress Range - ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td></td>
</tr>
</tbody>
</table>

Legend:
- No concrete
- 1 strand cut
- 2 strands cut
- 1 repair
- 2 repair
- Patched
Girder 3
Strand Severing

Strand 1

Strand 2

Strand 3

Strand 4

Strand Severing Data - Strands 1-4
Girder 3
Figure 8.18

- 218 -
Girder 3
Ultimate Load Test

![Graph showing ultimate load test for Girder 3. The x-axis represents average jack deflection (inches) ranging from 0 to 30. The y-axis represents total live load (kips) ranging from 0 to 300. The graph shows a peak at around 250 kips with a steady decrease to 0 as the deflection increases.](image-url)
Cracks Propagating from Damaged Region
Ultimate Load Test - Girder 3
Figure 8.20
Damaged Region at Completion of Ultimate Test
Girder 3
Figure 8.21
Ultimate Load Tests
Girders 1 and 3
Figure 8.22
force/ratio = \( \frac{7/9 \times 3}{1/9} = 21/9 \) undamaged

\[
\frac{\text{force strand}}{\text{force repair}} = \frac{7/9 \times 3}{1/9} = 21/9
\]

\[
\frac{\text{delta strand}}{\text{delta repair}} = \frac{P_{\text{strand}}}{P_{\text{repair}}} = \frac{3.5}{\text{delta}}
\]

\[
\frac{\text{delta strand}}{\text{delta repair}} = \frac{3.5}{\text{delta}}
\]

\[
\frac{p_{\text{strand}}}{p_{\text{repair}}} = \frac{3}{\text{delta}}
\]

\[
\frac{p_{\text{strand}}}{p_{\text{repair}}} = \frac{3}{\text{delta}}
\]

\[
\frac{p_{\text{strand}}}{p_{\text{repair}}} = \frac{3}{\text{delta}}
\]

\[
\frac{p_{\text{strand}}}{p_{\text{repair}}} = \frac{3}{\text{delta}}
\]
Magnification of Strand Stress
Repaired versus Undamaged

Percent Undamaged Stress

Percent of length which is repair

repair-5
repair-7
repair-9
Turnbuckle 1 – Static Response
(cycle 1 missing)

Static Test Response of Turnbuckle 1
Figure 8.25
Static Test Response
Turnbuckles 1 & 2

Total Live Load (kips)

Turnbuckle Load (kips)

Static Test Response of Turnbuckles 1 and 2
Figure 8.26
Static Test Response of a Repaired and an Undamaged Strand Girder 3
Figure 8.27
Girder 3 - Strand Response
Range 3 Fatigue Loading

Stress Ranges in Strands 1 and 4
Figure 8.28

Gird.er 3 - Strand Response
Range 3 Fatigue Loading

Strand Response
Range 3 Fatigue Loading

Damage/Repair State

Stress Range (ksi)

0 cut 1 cut 2 cut 1 rep. 2 rep. patched

strad 1
strad 4
Girder 3
Turnbuckle Force Ranges
Range 3 - Fatigue Loading

Turnbuckle Force Ranges
Figure 8.29

Turnbuckle Force Ranges
Figure 3

Repair/Fatigue Condition
1 fixed 2 fixed patch end 1 end 2 end 3

Force Range (kips)
0 2 4 6 8 10 12 14 16 18

turnbuckle 1 turnbuckle 2
Figure 8.30

Turnbuckle Load Cells
Ultimate Load Test

Turnbuckles 1 and 2
Ultimate Load Test
Figure 8.30
Girder 4
Cracking Test

Cracking Test
Girder 4
Figure 9.1

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Cracking Tests
Girders 1 - 4
Figure 9.2
Fatigue Loading - Girder 4

![Graph showing cumulative cycles vs. live load peak (kips).]

- Range 1
- Range 2
- Range 3
- Range 4
- Post-tensioned Repair

Cumulative Cycles (Millions)

Live Load Peak (kips)
Fatigue Loading - Actuator Deflection Span
Girder 4
Figure 9.4
Intermediate Static Tests
Girder 4
Figure 9.5
Figure 9.6

Girder 4
Predamage Static Tests

Predamage Static Tests
Girder 4
Figure 9.6

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Girders 1 and 4
Predamage Static Tests

Predamage Static Tests
Girders 1 and 4
Figure 9.7
Girder 4
Damage Static Tests

Figure 9.8

Total Live Load (kips)

Net Deflection (inches)

1 - predamage
2 - no concrete
3 - 1 cut
4 - 2 cut
Girder 4
Repair Static Tests

Repair Static Tests
Girder 4
Figure 9.9
Girder 4
Static Tests during Fatigue
After Repaired

Figure 9.10

Static Tests During Fatigue Loading After Being Repaired
Girder 4
Figure 9.10

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Figure 9.11

Reinforcing Bar Strain Gage Output
Girder 4
Figure 9.11
Figure 9.13

Static Test Response of Strand 3
Girder 4

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Static Test Response of Strand 4
Girder 4
Figure 9.14
Girder 4
Unsevered Strands - Range 3

Stress Ranges in Strands 3 and 4
Figure 9.15

Stress Range (ksi)

<table>
<thead>
<tr>
<th>Stress Range (ksi)</th>
<th>1 strand cut</th>
<th>2 strands cut</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

no concrete
rods tensioned
patched
Girder 4 – Strand Severing

Strand 1

Strand Severing Data – Strands 1 and 2
Girder 4
Figure 9.16
Ultimate Load Test
Girder 4
Figure 9.17
Figure 9.18

Ultimate Load Tests
Girders 1 and 4

Total Live Load (kips)

Average Jack Deflection (inches)

--- girder 1
--- girder 4
Post-tensioned Repair
Top Rod
Static Test Response

Static Test Response of Top Post-tensioned Rod
Girder 4
Figure 9.19
Static Test Response of Bottom Post-tensioned Rod
Girder 4
Figure 9.20

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Stress Ranges in Post-Tensioned Rods

Range 3 - Fatigue Loading

Stress Range (ksi)

<table>
<thead>
<tr>
<th>Rod</th>
<th>Repair/Fatigue Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rod 1</td>
<td>rods</td>
</tr>
<tr>
<td></td>
<td>patch</td>
</tr>
<tr>
<td></td>
<td>end 0</td>
</tr>
<tr>
<td></td>
<td>end 3</td>
</tr>
<tr>
<td></td>
<td>end 6</td>
</tr>
<tr>
<td>Rod 2</td>
<td></td>
</tr>
</tbody>
</table>
Girder - Corbel Interface Schematic

Girder 4

Figure 9.22

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Ultimate Load Test - Post-tensioned Rods
Girder 4
Figure 9.23
Cracks Intersecting Epoxyed Hairpin Holes
Girder 4
Figure 9.25
Damaged Region at 12 Inches of Deflection
Ultimate Load Test - Girder 4
Figure 9.26
Damaged Region After Additional Loading
Ultimate Load Test - Girder 4
Figure 9.27
PBEAM DISCRETIZATION

Centerline Cross Section
Figure 10.1

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PBEAM DISCRETIZATION

END VIEW

PBEAM Discretization
Support Cross Section
Figure 10.2

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LONG-TERM ANALYSIS
Centerline Moment

Figure 10.3
LONG-TERM ANALYSIS

Centerline Position

Deck 1 Added

Deck 2 Added

Centerline Position (inches)

Time Since Girder Cast (days)
LONG-TERM ANALYSIS

Prestressing Strand Stress

Long-term Analysis

Figure 10.5

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LONG-TERM ANALYSIS

Girder Top Concrete Stress

Figure 10.6
LONG-TERM ANALYSIS

Girder Top Concrete Strains

Figure 10.7
LONG-TERM ANALYSIS
Girder Bottom Concrete Stress

Girder Bottom Concrete Stress
Long-term Analysis
Figure 10.8

Stress (psi)

-2400 -2200 -2000 -1800 -1600 -1400 -1200 -1000 -800

0 1000 2000 3000 4000 5000 6000 7000 8000 9000

Time Since Girder Cast (days)
LONG-TERM ANALYSIS
Girder Bottom Concrete Strains

Strain

Time Since Girder Cast (days)

-1E-04
-0.0002
-0.0004
-0.0006
-0.0008
-0.001

-1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000

shrink
creep
instan.
Deck 1 Concrete Stress
Long-term Analysis
Figure 10.10

LONG-TERM ANALYSIS

Deck 1 Concrete Stress

Stress (psi)

Time Since Girder Cast (days)

400 350 300 250 200 150 100 50 0 -50 -100

1000 2000 3000 4000 5000 6000 7000 8000 9000
LONG-TERM ANALYSIS
Deck 1 Concrete Strains

Strain

Time Since Girder Cast (days)

-0.0005
-0.0004
-0.0003
-0.0002
-0.0001
0

0.0001

-1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000

shrink
creep
instan.
LONG-TERM ANALYSIS

Deck 1 Steel Stress

Figure 10.12

Deck 1 Steel Stress
Long-term Analysis
Figure 10.12

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LONG-TERM ANALYSIS
Deck 2 Concrete Stress

Figure 10.13
LONG-TERM ANALYSIS
Deck 2 Concrete Strains

- shrink
- creep
- instan.
LONG-TERM ANALYSIS

Deck 2 Steel Stress

Deck 2 Steel Stress
Long-term Analysis
Figure 10.15
Comparison of PBEAM With Experimental Data For Specimen Cracking

PBEAM Estimates of Cracking Load versus Girder 4
Figure 10.16
Effect of 100 microstrain Composite Deck Shrinkage Strain

Bending Moment (kip-inches)

150 ksi
130 ksi
110 ksi

w/ shrinkage
w/o shrinkage

Strand Stress (ksi)

PBEAM Generated Strand Stress versus Bending Moment with and without Deck Shrinkage
Figure 10.17
PBEAM Modeling
Failure Load Analysis

Figure 10.18

PBEAM Failure Load Analysis

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PBEAM Modeling
Failure Load Analysis

PBEAM Failure Load Analysis versus Girder 1
Figure 10.19
Material Models and Procedure for 2D_SECT

Figure 11.1
Discretization of Cross Section for 2D_SECT
Figure 11.2
Example Problem

Moment versus Strand Stress Plot
Girder and Composite Cross Sections
Figure 11.3
Example Problem

Influence of Concrete Tension on Strand Stress
Figure 11.4
Example Problem

Influence of Deck Strain Discontinuity on Strand Stress

Figure 11.5
Example Problem

Influence of Girder Concrete Strength on Strand Stress

Figure 11.6

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Example Problem

Influence of Effective Prestress on Strand Stress
Figure 11.7
Comparison of 2D_SECT with PBEAM and Hand Computations
Figure 11.8
Discretization of Girder for Damaged 2D_SECT Analysis

Figure 11.9
Symmetric Damage Analysis Curves
Figure 11.10
Knee of Symmetric Damage Analysis Curves
Figure 11.11
Moment Interaction

Out-of-plane as a function of In-plane

- M,oop-ten
- M,ip
- M,oop-com
- M,decom

nonsymmetric strand pattern
rotation of axes
concrete strut
Concrete Strut Effect

Undamaged Axis of Symmetry

bottom layer

stress = bottom layer, section under dead load
area = A * B
lever_arm = C + B/2
moment = stress * area * lever_arm

Concrete Strut Schematic
Figure 11.13

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Nonsymmetric Strand Pattern

Undamaged Axis of Symmetry

bottom layer

area = strand area
stress = bottom layer strand stress at live load moment - dead load moment
lever_arm = D
moment = stress * area * lever_arm

Nonsymmetric Strand Pattern Schematic
Figure 11.14
Discretization for NEWAXES
Figure 11.15
Variation of Sine and Cosine

Figure 11.16
Girders 3 and 4
Static test response of
Strand 4

Static Test Response of Strands
Girders 3 and 4 - 0, 1, and 2 Strands Cut
Figure 11.17
Girder 4

Experimental versus 2D_SECT Results

2D_SECT Estimate versus Girder 4
0, 1, and 2 Strands Cut
Figure 11.18
Figure 12.1
Crack Detection Gages

Crack Detection Gages
Girder 1
Figure 12.1
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Figure 12.2
Bottom Flange LVDT
Girder 1

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Girder 1
Bottom Flange Strain Gages

Bottom Flange Strain Gages
Girder 1
Figure 12.3
Crack Reopening Static Test
Girder 1

Figure 12.4
Girders 2, 3, and 4
Static Test Response

Bending Moment (kip-inches)

Strand Stress Change (ksi)

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Figure 12.6 - 2D_SECT with 110 ksi versus Girders 2, 3, and 4
APPENDIX A

Sample Calculations
Calculations for Flexural Strength

AASHTO Method - Section 9.17

\[ A_s' = \text{area of prestressing steel} \]
\[ = 4.32 \text{ square inches} \]

\[ f'_c = \text{compressive strength of concrete} \]
\[ = 5000 \text{ psi} \]

\[ b = \text{width of compressive block} \]
\[ = 64 \text{ inches} \]

\[ d = \text{depth of strands from top of composite section concrete to center of gravity of strands} \]
\[ = 47.17 \text{ inches} \]

\[ f = \text{nominal strength of strands} \]
\[ = 250,000 \text{ psi} \]

\[ \rho^* = \text{strand ratio} \]
\[ = \frac{A_s'}{bxd} \]
\[ = 0.001431 \]

\[ f_{wu}' = \text{stress in strand at failure} \]
\[ = f_s' \left[ 1 - 0.5 \rho^* \frac{f_s'}{f'_c} \right] \]
\[ = 241,100 \text{ psi} \]

\[ M_u = \text{flexural capacity of section} \]
\[ = A_s' f_{wu}' xd \left[ 1 - 0.6 \rho^* \frac{f_{wu}'}{f'_c} \right] \]
\[ = 47,100 \text{ kip-inches} \]
ACI / PCI Method

\[ A_{ps} \] = area of prestressing steel
\[ = 4.32 \text{ square inches} \]

\[ f'_c \] = compressive strength of concrete
\[ = 5000 \text{ psi} \]

\[ b \] = width of compression block
\[ = 64 \text{ inches} \]

\[ d \] = depth of prestressing strands
\[ = 47.17 \text{ inches} \]

\[ \rho_p \] = strand ratio
\[ = \frac{A_{ps}}{bxd} \]
\[ = 0.001431 \]

\[ \beta_i \] = whitney stress block factor
\[ = 0.80 \]

\[ \gamma_p \] = strand type factor
\[ = 0.40 \]

\[ f_{ps} \] = nominal strength of strand steel
\[ = 250,000 \text{ psi} \]

\[ f_{ps} \] = stress in strands at failure
\[ = F_{su} \left[ 1 - \frac{V_p}{\beta_i} \frac{\rho_p}{f^{'c}} \right] \]
\[ = 241,100 \text{ psi} \]

\[ a \] = depth of whitney stress block
\[ = \frac{A_{ps} f_p}{\rho_p} \left[ 0.85 \frac{f'_c}{b} \right] \]
\[ = 3.83 \text{ inches} \]

\[ M_a \] = nominal flexural capacity
\[ = \frac{A_{ps} f_p}{\rho_p} \left[ \frac{d - a}{2} \right] \]
\[ = 47,140 \text{ kip-inches} \]
Assume the strands yield

\[ A_{\text{strands}} = 4.32 \text{ inches}^2 \]
\[ \sigma_y = 250 \text{ ksi} \]
\[ T_f = \text{tension force} \]
\[ = A_{\text{strands}} \times \sigma_y \]
\[ = 1,080 \text{ kips} \]
\[ f'_c = \text{compressive strength of concrete} \]
\[ = 5 \text{ ksi} \]
\[ b = \text{width of compression block} \]
\[ = 64 \text{ inches} \]
\[ a = \text{depth of whitney stress block} \]
\[ = T_f/[0.85 x f'_c x b] \]
\[ = 3.97 \text{ inches} \]
\[ d = \text{depth of strands} \]
\[ M_n = \text{flexural capacity} \]
\[ = T_f [d - \frac{a}{2}] \]
\[ = 48,800 \text{ kip-inches} \]
Girder Bottom Fiber Stress Calculations

Girder Section Properties

\[ A_{\text{gir}} = \text{girder area} \]
\[ = 559.5 \text{ inches}^2 \]

\[ S_{\text{gb}} = \text{girder bottom section modulus} \]
\[ = 6186 \text{ inches}^3 \]

\[ e_s = \text{eccentricity of strands from girder centroid} \]
\[ = 16.14 \text{ inches} \]

\[ \omega_{\text{gir}} = \text{weight of girder} \]
\[ = 0.0486 \text{ kips per inch} \]
\[ = 0.583 \text{ kips per foot} \]

\[ c_{\text{gbf}} = \text{distance of girder centroid from bottom fiber} \]
\[ = 20.27 \text{ inches} \]

Composite Section Properties

\[ A_{\text{comp}} = \text{composite section area} \]
\[ = 852 \text{ inches}^2 \]

\[ S_{\text{cb}} = \text{composite section bottom section modulus} \]
\[ = 9252 \text{ inches}^3 \]

\[ \omega_{\text{comp}} = \text{weight of composite section} \]
\[ = 0.0819 \text{ kips per inch} \]

\[ c_{\text{cbf}} = \text{distance of comp. centroid from bottom fiber} \]
\[ = 29.81 \text{ inches} \]

\[ l = \text{span} \]
\[ = 776 \text{ inches} \]
Stress in the bottom fiber of the girder is due to five different components:
- axial compression due to the effective prestress;
- bending compression due to the eccentricity of the strands;
- bending tension as a result of the dead load moment;
- tension as a result of the composite deck shrinking;
- bending tension caused by the live load imposed by the actuators.

\[ M_{dl} = \omega_{comp} l^2 / 8 \]
\[ = 6,160 \text{ kip-inches} \]

\[ \sigma_{dl} = \frac{M_{dl}}{S_{pf}} \]

losses = percent of prestress force lost

\[ P_{axial} = 756 \times \left[ 1 - \frac{\text{losses}}{100} \right] \]

\[ \sigma_{ax} = \frac{P_{axial}}{A_{gird}} \]

\[ \sigma_{cc} = \frac{P_{axial} x e_s}{S_{gb}} \]

\[ \sigma_{shrink} = \text{redistribution of internal stresses due to the shrinkage of the new composite deck} \]

Assume the span is simply supported and load with two point loads. Each point load is located 320 inches from a support.

\[ M_i = 0.5 \times \text{Total Load} \times \frac{320}{S_{cb}} \]

\[ \sigma_i = \frac{M_i}{S_{cb}} \]

Bottom fiber stress = \[ \sigma_{dl} + \sigma_i + \sigma_{shrink} + \sigma_{cc} + \sigma_{ax} \]
APPENDIX B

Prestress Loss Estimates
AASHTO Method Prestress Loss Estimate

Total_loss = shrinkage + elastic_shortening + concrete_crep 
+ steel_relaxation

Shrinkage Component

relative_humidity := 70
shrinkage := 17000 - 150 * relative_humidity = 6500 psi

Elastic Shortening Component

Strand_modulus := 28,000,000 psi
Concrete_modulus := 3,605,000 psi
(using 4000 psi at transfer)

Now find the stress at the centroid of strand due to initial prestress force and the dead load of the beam at transfer.

prestress_stress := 756,000 / 559.5 = 1351 psi compression

dead_bending_moment := 0.125 * 582.8 * 64.8 * 64.8 * 12
= 3,671,000 lb-in

strand_eccentricity := 16.14 inches
inertia := 125,400 inches^4

bending_stress := dead_bending_moment * 
    strand_eccentricity/inertia
= 472.499 psi tension

net_stress := prestress_stress - bending_stress
= 879 psi compression

Elastic_shortening := net_stress * strand_modulus / 
concrete_modulus
= 6825 psi
Concrete Creep Component

deck_weight := 84 \times 6 / 144 \times 150 = 525 \text{ lbs/ft}

neglect dead load from diaphragms at third points

deck moment := 0.125 \times 525 \times 64.44 \times 64.66 \times 12
= 3,292,000 \text{ lb-in}

deck moment stress := deck moment \times \text{strad_eccentricity} / \text{inertia}
= 424 \text{ psi tension}

Concrete creep := 12 \times \text{net stress} + 7 \times \text{deck moment stress}
= 13511 \text{ psi}

Steel Relaxation Component

Relax := 20,000 - 0.4 \times \text{Elastic shortening} - 0.2 \times (\text{shrinkage} + \text{concrete creep})
= 13,268 \text{ psi}

Total Losses = Relax
+ Concrete creep
+ Elastic shortening
+ Shrinkage
= 40,104 \text{ psi}

* Note - AASHTO suggests a lump sum loss of 45,000 psi for this type of construction.
PCI METHOD

\[
\begin{align*}
Kes & := 1.0 & \text{for pretensioned members} \\
Es & := 28,000,000 & \text{strand modulus} \\
Eci & := 3,605,000 & \text{initial concrete modulus} \\
Kcir & := 0.9 & \text{value from PCI handbook} \\
Pi & := 756,000 & \text{initial prestress force} \\
e & := 16.14 & \text{eccentricity, inches} \\
\text{area} & := 559.5 & \text{girder cross-sectional area} \\
inertia & := 125,390 & \text{girder moment of inertia} \\
dead\_bending\_moment & := 3,670,801 \text{ lb-in} \\
\end{align*}
\]

\[
\begin{align*}
fcir & := Kcir*Pi*(1/\text{area} + e*e/\text{inertia}) - \\
& \quad \text{dead\_bending\_moment}*e/\text{inertia} = 2,157
\end{align*}
\]

\textbf{Elastic shortening} := Kes * fcir * Es / Eci

\[
= 16,754 \text{ psi}
\]

\[
\begin{align*}
Kcr & := 2.0 & \text{coefficient for normal weight concrete} \\
Ec & := 4,631,000 & \text{28 day strength modulus} \\
\text{deck\_moment} & := 3,292,000 \text{ lb-in} \\
\text{fcds} & = \text{deck\_moment}*e/\text{inertia} = 423.801 \text{ psi} \\
\text{Creep} & := Kcr*(fcir-fcds)*Es/Ec = 24,080 \text{ psi}
\end{align*}
\]

RH := 70
Ksh := 1.0
V/S := 5.09

\textbf{Shrinkage} := 0.0000082 * Ksh * Es * (1 - 0.06 * V/S) * (100 - RH)

\[
= 4,784 \text{ psi}
\]

Kre := 18,500 value from table
J := 0.14 value from table
C := 1.00 value from table
Relaxation := \( (Kre - J \times (Shrinkage + Creep + Elastic short.)) \times C \)
= 12,113 psi

<table>
<thead>
<tr>
<th>Total Losses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic shortening</td>
</tr>
<tr>
<td>Creep</td>
</tr>
<tr>
<td>Shrinkage</td>
</tr>
<tr>
<td>Relaxation</td>
</tr>
</tbody>
</table>
= 57,732 psi

PCI places an upper bound of 50,000 psi as the maximum amount of prestress loss that needs to be used in design.
T. Y. Lin and N. H. Burns

\[
\begin{align*}
Eci &= 3,605 \text{ ksi} \\
Es &= 29,000 \text{ ksi} \\
fcir &= 0.9 \times \frac{F_i}{\text{area}} + 0.9 \times \frac{e}{\text{inertia}} - \text{dead_bending_moment} \times \frac{e}{\text{inertia}} \\
&= 2.157 \\
\text{Elastic shortening} &= 2.157 \times Es / Eci = 17.35 \text{ ksi}
\end{align*}
\]

\[
\begin{align*}
Kcr &= 2.0 \\
Ec &= 4,031 \text{ ksi} \\
fcds &= \text{deck_moment} \times e / \text{inertia} = 0.4237 \text{ ksi} \\
\text{Creep} &= Kcr \times (fcir - fcds) \times Es / Ec = 24.94 \text{ ksi} \\
V/S &= 4.056 \text{ ignoring the ends ( Area / perimeter )} \\
RH &= 70 \\
Ksh &= 1.0 \\
\text{Shrinkage} &= 0.0000082 \times Ksh \times Es \times (1 - 0.06 \times V/S) \times (100 - RH) \\
&= 5.398 \text{ ksi} \\
Kre &= 18,500 \\
J &= 0.14 \\
C &= 1.0 \\
\text{Relaxation} &= (Kre - J \times (\text{Shrinkage} + \text{Creep} + \text{Elastic shortening})) \times C \\
&= 11.82 \text{ ksi}
\end{align*}
\]

Total Losses = Elastic shortening + Creep + Shrinkage + Relaxation

= 59,510 psi
**Collins and Mitchell Method**

\[ \text{initial\_stress\_ratio} \ := \ 0.70 \]
\[ \text{th} \ := \ 17,500 \text{ hours} \ = \ \text{approximately} \ 20 \text{ years} \]
\[ \text{initial\_stress} \ := \ \text{initial\_stress\_ratio} \times 250 = 175 \text{ ksi} \]

\[ \text{remaining\_ratio} \ := \ 1 - (\log(\text{th})/10) \times (\text{initial\_stress\_ratio} - 0.55) \]
\[ = \ 0.921 \]

\[ \text{Relaxation} \ := \ (1 - \text{remaining\_ratio}) \times \text{initial\_stress} \]
\[ = \ 13.763 \text{ ksi} \]

\[ Ksh \ := \ 0.75 \]
\[ Kh \ := \ 1.00 \]
\[ \text{td} \ := \ 7,300 \text{ days} \ = \ \text{approximately} \ 20 \text{ years} \]

\[ \text{shrink\_strain} \ := \ Ks \times Kh \times (0.00056) \times \text{td} / (55 + \text{td}) \]
\[ = \ 0.000417 \]

\[ \text{Shrinkage} \ := \ 28,000 \times \text{shrink\_strain} = 11.672 \text{ ksi} \]

\[ \text{phi} \ := \ 1.615 \ - \ \text{start loading at} \ 60 \text{ days and finish at} \ 20 \text{ years} \]
\[ \text{Ecl} \ := \ 57,000 \times \sqrt{5000} = 4,031,000 \text{ psi} \]
\[ \text{Eceff} := \text{Ecl} / (1 + \text{phi}) = 1,541,000 \text{ psi} \]

\[ \text{creep\_strain} \ := \ 424 / \text{Eceff} = 0.000275 \]

\[ \text{Creep} \ := \ 28,000 \times \text{creep\_strain} = 7.703 \text{ ksi} \]

Elastic shortening is not described. Use the average of the previous three elastic shortening values.

\[ \text{Elastic\ shortening} \ := \ 17.31 \text{ ksi} \]

<table>
<thead>
<tr>
<th>Total Losses =</th>
<th>Relaxation</th>
<th>+ Creep</th>
<th>+ Shrinkage</th>
<th>+ Elastic shortening</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>= 50,488 psi</td>
</tr>
</tbody>
</table>

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APPENDIX C

Repair Details
Turnbuckle Fabrication Details

1045 ROUND STOCK REQUIRED

SIX FEET OF 2" DIAMETER
SIX FEET OF 1 1/4" DIAMETER

Fy = 59,000   Fu = 98,000 psi

Steve Olson       January 11, 1989

WORK TO BE DONE

THREAD 6 CHUCK BARRELS
TURN 6 CHUCK CAPS
FABRICATE 6 CHUCK COUPLES
FABRICATE 3 R.H. -R.H. 1 1/4" RODS
FABRICATE 3 R.H. -L.H. 1 1/4" RODS
FABRICATE 3 TURNBUCKLES FOR 1 1/4" RODS
THREAD 6 CHUCK BARRELS

1 5/8" - 18 TPI - UNEF - RH
TURN DOWN 6 CHUCK CAPS

TURN DOWN CHUCK CAPS FROM
1 11/16" TO 1 1/2"
CHUCK - 1 1/4" CONNECTORS - 6 PIECES

MADE FROM 2" DIAMETER 1045 CARBON STEEL STOCK

1 5/8" - 18 TPI - UNEF - RH
TO MATCH CHUCK BARRELS

1 1/4" - 18 TPI - UNEF - RH
TO MATCH 1 1/4" ROD

- C4 -
CHUCK - COUPLE FLAT DETAIL

INCLUDED ANGLES ARE 90 DEGREES

ϕ2.00
<table>
<thead>
<tr>
<th>Threaded Rod from 1 1/4&quot; 1045 Round Stock</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1 1/4&quot;-18TPI-UNEF-RH</td>
<td>4.00</td>
</tr>
<tr>
<td>TURNED TO A DIAETER OF 1.128&quot;</td>
<td>4.00</td>
</tr>
<tr>
<td>1 1/4&quot;-18TPI-UNEF-LH</td>
<td>4.00</td>
</tr>
<tr>
<td>THREE RIGHT HAND - LEFT HAND PIECES</td>
<td></td>
</tr>
<tr>
<td>THREE RIGHT HAND - RIGHT HAND PIECES</td>
<td></td>
</tr>
<tr>
<td>1 1/4&quot;-18TPI-UNEF-RH</td>
<td>4.00</td>
</tr>
<tr>
<td>TURNED TO A DIAETER OF 1.128&quot;</td>
<td>4.00</td>
</tr>
<tr>
<td>1 1/4&quot;-18TPI-UNEF-RH</td>
<td>12.00</td>
</tr>
<tr>
<td>C6</td>
<td></td>
</tr>
</tbody>
</table>
1 1/4" ROD TURNBUCKLE

3 PIECES REQUIRED

OUTSIDE DIMENSIONS SAME AS HEAVY HEX NUT

CONSTRUCTED FROM 2" DIAMETER 1045 ROUND STOCK
Post-tensioned Repair Calculations

Corbel Design

Use shear friction method

Each corbel must be able to develop the ultimate force of two 5/8 inch diameter Dywidag bars. Ultimate load for each bar is 43.5 kips. Use a load factor of 1.1.

Therefore,

\[ \text{required capacity} = 2 \times 43.5 \times 1.1 = 95.7 \text{ kips} \]

From a pullout test, the hairpins in Hilti epoxy are capable of developing at least 12.96 kips. Interface was roughened with a grinder, therefore coefficient of friction was 1.0. Use a phi of 0.85.

\[ \text{required number of hairpins} = \frac{95.7}{(0.85 \times 12.96 \times 1)} = 8.69 \]

Use 9 hairpins per corbel.

Use 9 hairpins at 5.5 inch spacing. Corbels are 48 inches long, 8 spaces \( \times \) 5.5 inches = 44 inches. Embedment depth of hairpins as tested was 4.25 inches.

Centerline face of corbels was located at least 100 strand diameters away from the damaged strand. This allows the strand to redevelop its effective stress before the corbels.
Calculate load to put into post-tensioned rods to restore bottom fiber stress

area of each severed strand = 0.144 inches^2
total area lost = 0.288 inches^2

assume 130 ksi effective in strands when severed

force lost = 130 * 0.288 = 37.44 kips
centroid of lost strands = 3 inches above bottom flange
centroid of post-tensioned rods = 12.25 inches above b.f.

Work with composite section because strands were severed when the section was fully composite.

area = 852 inches^2
inertia = 275,800 inches^4
Y_top = 21.19 inches
Y_bottom = 29.81 inches

axial stress lost = \( \frac{37.44}{852} = 0.04393 \) ksi
bending stress lost = \( 37.44 \times 26.81 \times 29.81 / 275,800 = 0.1085 \) ksi

total bottom fiber stress lost = 0.1085 + 0.0439 = 0.1524 ksi

\[ 0.1524 \text{ ksi} = \frac{2P}{852} + \frac{2P \times (17.56) \times (29.81)}{275,800} \]

P = load in each rod
P = 24.81 kips

single rod ultimate capacity = 43.5 kips
0.6 \times 43.5 = 26.1 kips

24.81 kips per rod - O.K.
Check loss of ultimate capacity

Assume at ultimate, neutral axis 3 inches below top of deck
Assume each rod yields, max force/rod = 43.5 kips
lever arm = (51 - 3 - 12.25) = 35.75 inches

Repair develops 87 * 35.75 = 3,110 kip-inches

strand area lost = 0.288 inches^2
force lost = 250 ksi * 0.288 = 72 kips
lever arm = (51 - 3) = 45 inches

Strands would develop 72 * 45 = 3,240 kip-inches

Difference = 3,240 - 3,110 = 130 kip-inches. At failure, section develops more than 45,000 kip-inches. Difference is insignificant.

Check transverse moments

Repair force = 87 kips
Repair eccentricity = 9 inches

Repair Moment = 87 * 9 = 783 kip-inches

Strand force = 72 kips
Strand eccentricity = 9 inches

Strand Moment = 72 * 9 = 648 kip-inches

Difference = 783 - 648 = 135 kip-inches. Difference is negligible.
## Post-tensioned Repair Hardware and Details

<table>
<thead>
<tr>
<th>Item Description</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Twenty foot, 5/8 inch diameter, 157 ksi, dywidag bars</td>
<td>2</td>
</tr>
<tr>
<td>5/8 inch dywidag rod nuts, (4 for repair, 1 for jacking)</td>
<td>5</td>
</tr>
<tr>
<td>Anchor plates</td>
<td>1</td>
</tr>
<tr>
<td>Center hole stressing jack</td>
<td>1</td>
</tr>
<tr>
<td>Grout tubes</td>
<td>4</td>
</tr>
<tr>
<td>Grout sleeves</td>
<td>4</td>
</tr>
<tr>
<td>20' sheathing for 5/8&quot; rods</td>
<td></td>
</tr>
<tr>
<td>No. 4 hairpins</td>
<td>18</td>
</tr>
<tr>
<td>No. 3 stirrups</td>
<td>16</td>
</tr>
<tr>
<td>No. 3 longitudinal bars (46&quot; long)</td>
<td>6</td>
</tr>
<tr>
<td>Hammer drill</td>
<td>1</td>
</tr>
<tr>
<td>5/8 inch diameter masonry bit</td>
<td>1</td>
</tr>
<tr>
<td>Hole cleanout kit</td>
<td>1</td>
</tr>
<tr>
<td>Hilti mixer/dispenser</td>
<td>1</td>
</tr>
<tr>
<td>Hilti Hit C-100 cartridges</td>
<td>2</td>
</tr>
<tr>
<td>Formwork for corbels</td>
<td></td>
</tr>
<tr>
<td>Concrete for corbels and patching</td>
<td></td>
</tr>
<tr>
<td>Grout for post-tensioned rods</td>
<td></td>
</tr>
</tbody>
</table>
5/8" 157 ksi
Dywidag bars

bearing plates
Grade 60
Number 4 rebar

Total length
21 inches

- C15 -
Grade 80
Number 3 rebar

Total Length
= 22.25"

Lap bars on 3.6" face for the full length
APPENDIX D

Computer Input and Programs
Sample PBEAM Input File

University of Minnesota Test Specimens - AASHTO Type III w/ deck
Start of Long-term losses analysis, Time-dependent response, November 28, 1990

START
1Strands initially tensioned to 175 ksi, superposition creep model

13
12 33 12 52 5 3
38 38 2.000e+00 8.500e+03
2.010e+00 4.000e+00 7.000e+00 1.400e+01 2.800e+01 5.600e+01 7.400e+01 7.500e+01
7.600e+01 8.200e+01 8.900e+01 1.030e+02 1.310e+02 2.000e+02 4.000e+02 7.000e+02
1.000e+03 1.500e+03 2.500e+03 4.000e+03 5.500e+03 6.999e+03 7.000e+03 7.007e+03
7.028e+03 7.500e+03 7.508e+03 7.515e+03 7.529e+03
7.557e+03 7.601e+03 7.801e+03 8.000e+03 8.500e+03
2.010e+00 4.000e+00 7.000e+00 1.400e+01 2.800e+01 5.600e+01 7.400e+01 7.500e+01
7.600e+01 8.200e+01 8.900e+01 1.030e+02 1.310e+02 2.000e+02 4.000e+02 7.000e+02
1.000e+03 1.500e+03 2.500e+03 4.000e+03 5.500e+03 6.999e+03 7.000e+03 7.007e+03
7.028e+03 7.500e+03 7.508e+03 7.515e+03 7.529e+03
7.557e+03 7.601e+03 7.801e+03 8.000e+03 8.500e+03
5 20 1.000e+00 1.000e+00 1.000e+00 1.000e+00

1 1 1 1 1 1 8.681e-02
1.000e+00 1.000e-06 1.110e-04 -0.700e-02
0.70 0.7976
0.70 0.70
0.60 1.00 1.13 -0.095

-389.7e-06 55.0
2 1 1 1 1
1.000e+00 1.000e+00 1.302e+00 -7.000e-02
0.3509 0.6061 0.8955 1.007 1.085 1.176 0.0 0.0
2.0 5.0 15.0 28.0 56.0 7000.0 7001.0 10000.0
0.5923 0.7785 0.9463 1.042 1.084 1.084 1.084
2.0 5.0 15.0 28.0 56.0 7000.0 7001.0 10000.0
1.958 0.70 0.60 10.0 1.25 -0.118

-453.1e-06 35.0
3 1 3 1 2.836e-01
1.000e+02 1.000e-04 6.530e-02 -6.530e-02
2.125e+05 10.0 0.55 0.60
4 4 2.836e-01
1.000e+02 1.000e-05 6.530e-02 -6.530e-02
5 5 12 2.836e-01
1.000e+02 1.000e-05 6.530e-02 -6.530e-02
1.0 1.0 1.0 1.0 1.0 1.0 0.0 0.0
2.0 5.0 15.0 28.0 56.0 7000.0 7001.0 10000.0
1.0 1.0 1.0 1.0 1.0 1.0 1.0 1.0
2.0 5.0 15.0 28.0 56.0 7000.0 7001.0 10000.0
6 1 1 1 1 1 8.681e-02
1.000e+00 1.000e-04 -7.000e-02
4.00 0.85
1.958 0.70 0.60 10.0 1.25 -0.118

-453.1e-06 35.0
1 -6700-3350-6500
-2200-2877-1000
-4500-2250-3824
-2200-3429-1000
3 8 1 0 2240 2364 2408 2427 2439 2450 2500
0 80 110 140 170 200 253 650
4 3 1 0 600 600
0 207 6530
5 3 1 0 600 600
0 207 6530
6 -4500-2250-3824
-2200-3429-1000
20 7.610e+02
19 *number of sub rectangles*
1 4 1 1.0 1.0 2.0 *start grid cone*
22.0 3.5 -28.06 0.0

- D1 -
Sample PBEAM input file

```
1 4 1 1.0 1.0 2.0
1 4 1 1.0 1.0 2.0
18.25 3.75 -21.06 0.0
1 4 1 1.0 1.0 2.0
10.75 3.75 -17.40 0.0
1 4 1 1.0 1.0 2.0
7.0 19.0 -6.81 0.0
1 4 1 1.0 1.0 2.0
11.5 4.5 6.23 0.0
1 4 1 1.0 1.0 2.0
16.0 7.0 11.69 0.0
2 6 1 75.0 75.0 75.0 *start deck 1*
2 6 1 75.0 75.0 75.0 *start deck 1*
6 6 1 7500.0 7500.0 7501.0 *start deck 1*
3 1 1 1.0 1.0 2.0
3.168 1.0 -25.99 6.250e-03
3 1 4 2.0 2.0
0.000e+00 2.880e-01 1.000e+00 4.190e+00 6.250e-03
3.205e+02 2.880e-01 1.000e+00-2.781e+01 6.250e-03
4.405e+02 2.880e-01 1.000e+00-2.781e+01 6.250e-03
7.610e+02 2.880e-01 1.000e+00 4.190e+00 6.250e-03
3 1 4 2.0 2.0
0.000e+00 2.880e-01 1.000e+00 6.190e+00 6.250e-03
3.205e+02 2.880e-01 1.000e+00-2.581e+01 6.250e-03
4.405e+02 2.880e-01 1.000e+00-2.581e+01 6.250e-03
7.610e+02 2.880e-01 1.000e+00 6.190e+00 6.250e-03
3 1 1 2.0 2.0
0.000e+00 2.880e-01 1.000e+00 8.190e+00 6.250e-03
3.205e+02 2.880e-01 1.000e+00-2.381e+01 6.250e-03
4.405e+02 2.880e-01 1.000e+00-2.381e+01 6.250e-03
7.610e+02 2.880e-01 1.000e+00 8.190e+00 6.250e-03
3 1 4 2.0 2.0
0.000e+00 2.880e-01 1.000e+00 1.019e+01 6.250e-03
3.205e+02 2.880e-01 1.000e+00-2.181e+01 6.250e-03
4.405e+02 2.880e-01 1.000e+00-2.181e+01 6.250e-03
7.610e+02 2.880e-01 1.000e+00 1.019e+01 6.250e-03
4 1 1 1.0 1.0 1.0
4.0 1.0 12.19 0.0
5 1 1 75.0 75.0 75.0 *l. dck mild*
5 1 1 75.0 75.0 75.0 *l. dck mild*
1.0 1.0 18.69 0.0
4 1 1 7501.0 7501.0 7501.0 *u. dck mild*
4 1 1 7501.0 7501.0 7501.0 *u. dck mild*
1.0 1.0 18.69 0.0
number of lds
2
1 75.0 -43.75
1 7000.0 43.75
2
0.000e+00 -1.000e+30
7.610e+02 -1.000e+30 -1.000e+30
CEASE
```

- D2 -
Sample PBEAM input file

Program 2D_SECT

program MS_EZ ( input , output ) ;

{ input units are kips and inches}

This program utilizes the layer approach described by COLLINS and MITCHELL in their book "PRESTRESSED CONCRETE BASICS". It is a converted version of the program PLANE at the back of their book. Created 9/5/89 by S.A.O.

modified 7/19/90 to output the stress of the first tendon layer when a position of equilibrium is found,

modified 7/26/90 to write the output to a text file to be picked up by Quattro.

a series of strains are used by the program to generate output curves)

VAR
tendon_stress : real ;
bottom_concrete_stress : real ;
fileit : integer ;
fileit_ok : integer ;
out_file : text ;
out_file_name : string[80] ;
F_0,S_0,F_1,O_1,E_1,
E_2,Y_0,A_4,E_3,C_3,
E_3,U_3
Y_1,E_1,H_1,K_1
z_1,k_9
z_2,x_3
Y_2,A_2,Y_3,A_3,D_3,
T_2,T_3,D_2,D_4
K,T_1,D_1
f0,s7,s8,f3,f4,f5,
q1,y3,y4,y5,y6
f6,e2,y0,s9,m,a3,
b3,e3,e3,c1
s,e1,f1,f0,s0,q5,
q2,q3,y9,a1,n5
.s1,s2,s5,s6,n
m1,n1,n8,m3,n3
z1,z2,z3,b9,n2,
k4,t0,d0,t,c,b9,q
loop : integer ;
problem_string : string[80] ;
dum_char,continue : char ;
input_text_file : TEXT ;
file_name_string : string[80] ;
input_strains : ARRAY[1..100] OF REAL ;

procedure define_input_strains ;
begin
input_strains[ 1 ] := 0.005 ;
input_strains[ 2 ] := 0.001 ;
input_strains[ 3 ] := -0.005 ;
input_strains[ 4 ] := -0.010 ;
input_strains[ 5 ] := -0.015 ;
input_strains[ 6 ] := -0.020 ;
input_strains[ 7 ] := -0.025 ;
input_strains[ 8 ] := -0.030 ;
Sample PBEAM input file

input_strains[  9] := -0.035
input_strains[10] := -0.040
input_strains[11] := -0.045
input_strains[12] := -0.050
input_strains[13] := -0.060
input_strains[14] := -0.070
input_strains[15] := -0.080
input_strains[16] := -0.090
input_strains[17] := -0.100
input_strains[18] := -0.110
input_strains[19] := -0.120
input_strains[20] := -0.130
input_strains[21] := -0.140
input_strains[22] := -0.150
input_strains[23] := -0.160
input_strains[24] := -0.170
input_strains[25] := -0.180
input_strains[26] := -0.190
input_strains[27] := -0.200
input_strains[28] := -0.220
input_strains[29] := -0.240
input_strains[30] := -0.260
input_strains[31] := -0.280
input_strains[32] := -0.300
input_strains[33] := -0.320
input_strains[34] := -0.340
input_strains[35] := -0.360
input_strains[36] := -0.380
input_strains[37] := -0.400
input_strains[38] := -0.420
input_strains[39] := -0.440
input_strains[40] := -0.460
input_strains[41] := -0.480
input_strains[42] := -0.500
input_strains[43] := -0.550
input_strains[44] := -0.600
input_strains[45] := -0.650
input_strains[46] := -0.700
input_strains[47] := -0.750
input_strains[48] := -0.800
input_strains[49] := -0.850
input_strains[50] := -0.900
input_strains[51] := -0.950
input_strains[52] := -1.000
input_strains[53] := -1.100
input_strains[54] := -1.200
input_strains[55] := -1.300
input_strains[56] := -1.400
input_strains[57] := -1.500
input_strains[58] := -1.600
input_strains[59] := -1.700
input_strains[60] := -1.800
input_strains[61] := -1.900
input_strains[62] := -2.000
input_strains[63] := -2.100
input_strains[64] := -2.200
input_strains[65] := -2.300
input_strains[66] := -2.400
input_strains[67] := -2.500
input_strains[68] := -2.600
input_strains[69] := -2.700
input_strains[70] := -2.800
input_strains[71] := -2.900

- D4 -
Sample PBEAM input file

```
input_strains[72] := -3.000 ;
input_strains[73] := -3.100 ;
input_strains[74] := -3.200 ;
input_strains[75] := -3.300 ;
input_strains[76] := -3.400 ;
input_strains[77] := -3.500 ;
input_strains[78] := -3.600 ;
input_strains[79] := -3.700 ;
input_strains[80] := -3.800 ;
input_strains[81] := -3.900 ;
input_strains[82] := -4.000 ;
input_strains[83] := -4.100 ;
input_strains[84] := -4.200 ;
input_strains[85] := -4.300 ;
input_strains[86] := -4.400 ;
input_strains[87] := -4.500 ;
input_strains[88] := -4.600 ;
input_strains[89] := -4.700 ;
input_strains[90] := -4.800 ;
input_strains[91] := -4.900 ;
input_strains[92] := -5.000 ;
input_strains[93] := -5.100 ;
input_strains[94] := -5.200 ;
input_strains[95] := -5.300 ;
input_strains[96] := -5.400 ;
input_strains[97] := -5.500 ;
input_strains[98] := -5.600 ;
input_strains[99] := -5.700 ;
input_strains[100] := -5.800 ;

end;

procedure calc_conc_stress ;
begin
  if (s>-0.1) and (s<e1/e1) then f := e1*s ;
  if (s>e1/e1) then f := a9*q1*f1/(1+sqrt(0.5*s)) ;
  if (s<0) and (s>2*s0) then f:=(2*s/s0-sqr(s/s0))*f0 ;
  if (s<2*s0) or (s=2*s0) then f := 0 ;
end ;

procedure estimate_new_strain ;
begin
  if n=n8 then s7 := s+c/5 ;
  if n<>n8 then s7 := s+(n5-n)*(s-s8)/(n-n8) ;
end ;

BEGIN
  define_input_strains ;
  for i := 1 to 6 do writeln ;
  writeln( What is the name of the input file? ' ) ;
  readin(file_name_string) ;
  writeln ; writeln ;
  ( write( Do you want output written to a file? (1=yes, 0=no) ' ) ;
    readin(fileit_ok) ; )
  fileit_ok := 1 ;
  if fileit_ok=1 then begin
    writeln ;
    write( What is the name of the output file? ' ) ;
    readin(out_file_name) ;
    assign(out_file, out_file_name) ;
```

-D5-
Sample PBEAM input file

```pascal
rewrite(out_file);
writeln(out_file,'Et Eb N M CV TS TC');
end;
assign(input_text_file, file_name_string ) ;
reset ( input_text_file ) ;
readin(input_text_file,problem_string);
writeln('**********************************');
writeln(problem_string);
writeln('**********************************');
{input the material properties}
readin(input_text_file,QS,dum_char,Q2,dum_char,Q3) ;
writeln('MATERIAL RESISTANCE FACTORS');
writeln('Fee cone=' ,q5:4:2,' Fee rebar=',q2:4:2,' Fee tendon=',q3:4:2);
writeln('CONCRETE PROPERTIES');
readln(input_text_file,Z1);
for i := 1 to z1 do begin
  readln(input_text_file,F_O[i],dum_char,S_O[i],F_1[i],dum_char,Q_1[i] );
  writeln('l=' ,i:1,' F'ec'=' ,F_O[i]:8:2,' ec'=' ,S_O[i]:8:2,
   ' Fcr'=' ,F_1[i]:8:2,' Q1=' ,Q_1[i]:8:2 );
  F_0[i] := - F_0[i] - .E-Hil := 2*F omis om end;

writeln('REBAR PROPERTIES');
readin(input_text_file,Z2) ;
for i := 1 to z2 do begin
  readln(input_text_file,E_2[i],dum_char,Y_O[i]);
  writeln('l=' ,i:1,' E/1000=' ,E_2[i]:8:2,
   'YIELD=' ,Y_O[i]:8:2) end;
readin(input_text_file,Z3) ;
writeln('TENDON PROPERTIES');
for i := 1 to z3 do begin
  readln(input_text_file,a_4[i],dum_char,b_3[i],dum_char,c_3[i],dum_char,e_3[i],dum_char,u_3[i] ) ;
  writeln(' l=' ,i:1,' -A=' ,a_4[i]:8:2,' __B=' ,b_3[i]:8:2,
   'I C=' ,c_3[i]:8:2,' E/1000=' ,e_3[i]:8:2,' Fpu=' ,u_3[i]:8:2); end;
writeln('******************');
writeln('SECTION COMPONENTS');
writeln('********************');
readin(input_text_file,H,dum_char,y9,dum_char,c9);
writeln('Height=',h:8:3,' Moment axis=',y9:8:3);
writeln('No. of concrete components=',c9:4);
ai := 0;
for i := 1 to c9 do begin
  readin(input_text_file,k_9[i],dum_char,y_1[i],dum_char,
   b_1[i],dum_char,h_1[i],dum_char,z_1[i] ) ;
  writeln('l=' ,i:1,' Shape']=' ,k_9[i]:1,' Y=' ,y_1[i]:8:3,
   ' B=' ,b_1[i]:8:3,' H=' ,h_1[i]:8:3,' Type']=' ,z_1[i]:1 ) ;
if k_9[i]=1 then begin
  k[i,1] := 1 ;
  k[i,2] := 4 ;
  k[i,3] := 1 ;
  k[i,4] := 1 ;
end ;
if k_9[i]=2 then begin
  k[i,1] := 0 ;
end ;
```
Sample PBEAM input file

\[
\begin{align*}
  k[i,2] &:= 2; \\
  k[i,3] &:= 1; \\
  k_1[i] &:= 0.5; \\
  \text{if } k[i,1] = 3 \text{ then begin} \\
    k[i,1] &:= 1; \\
    k[i,2] &:= 2; \\
    k[i,3] &:= 0; \\
    k_1[i] &:= 0.5; \\
  \text{end;}
\end{align*}
\]

\[
\begin{align*}
  a_1 &:= a_1 + k_1[i] \cdot b_1[i] \cdot h_1[i]; \\
end; \\
\text{writeln('CONC AREA=', A1:8:2, ' in^2');} \\
\text{writeln;}
\end{align*}
\]

\[
\begin{align*}
  \text{readln(input_text_file, n2);} \\
  \text{writeln('No. of rebar layers=', n2:1);} \\
  \text{for } i := 1 \text{ to } n2 \text{ do begin} \\
    \text{readln(input_text_file, y_2[i], dum_char, a_2[i], dum_char, z_2[i]);} \\
    \text{writeln('I=', i:1, ' Y=', y_2[i]:8:3, ' Area=', a_2[i]:8:3, ' Type=', z_2[i]:1);} \\
  \text{end;}
\end{align*}
\]

\[
\begin{align*}
  \text{readln(input_text_file, n4);} \& \text{ writeln('No. of tendon layers=', n4:1);} \\
  \text{for } i := 1 \text{ to } n4 \text{ do begin} \\
    \text{readln(input_text_file, y_3[i], dum_char, a_3[i], dum_char, d_3[i], dum_char, z_3[i]);} \\
    \text{writeln('I=', i:1, ' Y=', y_3[i]:8:3, ' Area=', a_3[i]:8:3, ' Strain diff.=', d_3[i]:8:3, ' Type=', z_3[i]:1);} \\
  \text{end;}
\end{align*}
\]

\[
\begin{align*}
  \text{readln(input_text_file, t0);} \& \text{ if t0<>0 then begin} \\
    \text{writeln('*************************');} \\
    \text{writeln('THERMAL+SHRINKAGE STRAINS');} \\
    \text{writeln('*************************');} \\
    \text{for } i := 1 \text{ to } c9 \text{ do begin} \\
      \text{readln(input_text_file, t_1[i,1], dum_char, t_1[i,2], dum_char, t_1[i,3]);} \\
      \text{writeln('I=', i:1, ' Bot. strain=', t_1[i,1]:8:3, ' Mid. strain=', t_1[i,2]:8:3, ' Top strain=', t_1[i,3]:8:3);} \\
    \text{end;}
\end{align*}
\]

\[
\begin{align*}
  \text{for } i := 1 \text{ to } (n2-1) \text{ do begin} \\
    \text{readln(input_text_file, t_2[i], dum_char);} \\
    \text{writeln('I=', i:1, ' Rebar thermal strain=', t_2[i]:8:3);} \\
  \text{end;}
\end{align*}
\]

\[
\begin{align*}
  \text{for } i := 1 \text{ to } (n4-1) \text{ do begin} \\
    \text{readln(input_text_file, t_3[i], dum_char);} \\
    \text{writeln('I=', i:1, ' Tendon thermal strain=', t_3[i]:8:3);} \\
  \text{end;}
\end{align*}
\]

\[
\begin{align*}
  \text{readln(input_text_file, d0);} \& \text{ if d0<>0 then begin} \\
    \text{writeln(' hay***********');} \\
    \text{writeln('INITIAL STRAINS');} \\
\end{align*}
\]
Sample PBEAM input file

```pascal
writeln('***************') ;
for i := 1 to c9 do begin
  readln(input_text_file,
    d_1[i,1],dum_char,d_1[i,3] ) ;
  d_1[i,2] := (d_1[i,1]+d_1[i,3])/2 ;
  writeln('i=',i:1,' Bottom strain=','d_1[i,1]:8:3,' Top strain=','d_1[i,3]:8:3');
end;
for i := 1 to (n2-1) do begin
  read(input_text_file,d_2[i],dum_char);
  writeln('l=',i:1,' Initial strain=',d_2[i]:8:3);
end;
readln(input_text_file,d_2[n2]);
writeln('l=',n2:1,' Initial strain=',d_2[n2]:8:3);
for i := 1 to (n4-1) do begin
  read(input_text_file,d_4[i],dum_char);
  writeln('I=',i:1,' Initial concrete strain at tendon=','d_4[i]:8:3');
end;
readln(input_text_file,d_4[n4]);
writeln('I=',n4:1,' Initial concrete strain at tendon=','d_4[n4]:8:3');
end;
writeln('********************');
write('N kips (TENSION+)=');
{readln(n5) ;}
r5 := 0.0 ;
writeln('N=',n5:8:2,' kips');
writeln('**************************');
write('Account for tension stiffening (y/n) ');
( readline(dum_char ) )
  dum_char := 'n' ;
if (dum_char='y') or (dum_char='Y')
  then a9:=1 else a9:=0 ;
if a9=1 then writeln
  ('POST-CRACKING TENSION IN CONCRETE ACCOUNTED FOR');
if a9=0 then writeln
  ('POST-CRACKING TENSION IN CONCRETE NEGLECTED');
writeln('**************************');
write('Which strain constant, top or bottom (t/b) ');
( readline(dum_char ) )
dum_char := 't' ;
if (dum_char='t') or (dum_char='T')
  then t:=1 else t:=2 ;
writeln;
continue := 'y' ;
loop := 1 ;
repeat
  if loop <> 1 then begin
    write('continue2 (y/n) ');
    readline(continue) ;
  end
  if (continue='y') or (continue='Y') then begin
    if t=1 then begin
      write('Top strain * 1000= ');
      readln(s1) ;
    end
    if t=2 then begin
      write('Bottom strain * 1000= ');
      readln(s2) ;
    end
  end
  loop := loop + 1 ;
end;```

Sample PBEAM input file

s1 := -s2 * 3;
end;
q := 1;
c := 1;
n := 0;
for i := 1 to n do begin
s3 := s2+(s1-s2)*y_1[i]/h;
s4 := s2+(s1-s2)*(y_1[i]+h_1[i])/h;
s5 := s2+(s1-s2)*(y_1[i]+h_1[i])/h;
if t0=1 then begin
s3 := s3*t_1[i,1];
s4 := s4*t_1[i,2];
s5 := s5*t_1[i,3];
end;
if d0=1 then begin
s3 := s3+d_1[i,1];
s4 := s4+d_1[i,2];
s5 := s5+d_1[i,3];
end;
f0 := f_0[i];
s0 := s_0[i];
f1 := f_1[i];
q1 := q_1[i];
e1 := e_1[i];
s := s3;
calc_conc_stress;
f3 := f*q5;
s := s4;
calc_conc_stress;
f4 := f*q5;
s := s5;
calc_conc_stress;
f5 := f*q5;
y_3 := y_1[i];
y_5 := y_3 + h_1[i];
y_4 := (y_3+y_5)/2;
f6 := (k[i,1]*f3+k[i,2]*f4+k[i,3]*f5)/6;
if i=1 then bottom_concrete_stress := f6;
y_6 := (k[i,1]*f3+y_3+k[i,2]*f4+y_4+k[i,3]*f5*y_5)/6;
n1 := n1+f6*b_1[i]*h_1[i];
m1 := m1+y_6*b_1[i]*h_1[i];
end;
m3 := 0;
for i := 1 to n do begin;
s := s2+(s1-s2)*y_2[i]/h;
if t0=1 then s := s-t_2[i];
if d0=1 then s := s+d_2[i];
e2 := e_2[i];
y_6 := y_0[i];
s9 := y_0/e2;
if (s<s9) and (s>-s9) then f := e2 * s;
if (s>s9) or (s=-s9) then f:=y_0;
if (s<-s9) or (s=s9) then f:=-y_0;
n3 := n3 +f*q2*a_2[i];
m3 := m3+f*q2*a_2[i]*y_2[i];
end;
Sample PBEAM input file

for i := 1 to n4 do begin
  s := s2*(s1-s2)*y_3[i]/h+d_3[i] ;
  if t0=1 then s := s-t_3[i] ;
  if d0=1 then s := s+d_4[i] ;
  a3 := a_4[i] ;
  b3 := b_3[i] ;
  c3 := c_3[i] ;
  e3 := e_3[i] ;
  u3 := u_3[i] ;
  f := e3*s*(a3+(1-a3)/
           exp(1/c3*ln((1+exp(c3*ln(b3*s/1000))))));
  if f>u3 then f:=u3 ;
  if f<-u3 then f:=-u3 ;
  if i=1 then tendon_strength := f ;
  n3 := n3 + f*q3*a_3[i] ;
  m3 := m3 + f*q3*a_3[i]*y_3[i] ;
end;

n := (n1+n3) ;

m := -(m1+m3)+n*y9 ;
c := c+1 ;
writeln(s1:10:6,s2:10:6,n:10:2,m:10:2);
if (abs(n-n5)>0.05) then
  begin
    q := q + 1 ;
    if (q=2) and (t=1) then begin
      s8 := s2 ;
      n8 := n ;
      s2 := s2 + 9 * (n5-n)/(a1*e1) ;  \{9 works\}
    end;
    if (q=2) and (t=2) then begin
      s8 := s1 ;
      n8 := n ;
      s1 := s1 + 9 * (n5-n)/(a1*e1) ;
    end;
    if (t=1) and (q>2) then begin
      s := s2 ;
      estimate_new_strain ;
      s8 := s2 ;
      s2 := s7 ;
    end;
    if (t=2) and (q>2) then begin
      s := s1 ;
      estimate_new_strain ;
      s8 := s1 ;
      s1 := s7 ;
    end;
  end;
until (abs(n-n5)<0.05) ;
loop := loop + 1 ;
writeln;'**********

cl := (s2-s1)*1000/h ;
writeln('M=',M:10:4, 
'k-in CV=',c1:10:4,' mRad/kiloin',
'TS = /', tendon_strength:10:4);
writeln;'**********
end ;
if fileit_ok=1 then begin
write('Send this data to the file? (1=yes, 0=no) ');
readln(fileit) ;
fileit := 1 ;
if fileit=1 then begin
write(out_file,s1,' ',s2,' ',n,' ',
- D10 -
Sample PBEAM input file

m,'c1','tendon_stress,'bottom_concrete_stress);
end;
end;
until (loop = 100);
if fileit_ok = 1 then Close(out_file);
END.
Sample PBEAM input file

Program NEWAXES

Program axes;

(This program receives cross section information from the user and uses it to find the centroid of the cross section, the area of the cross section, the principal axes of the cross section, and lastly the principal moments of inertia. The information from this program is then utilized to determine the stress ranges in the strands. Created by Steve Olson May 28, 1990.

The algorithm for the program is the following:
1) input the section about horiz. & vert. axes
2) compute the centroid of the cross section
3) compute Ixx, Iyy, Ixy about the centroid
4) compute principal axes from Mohr's circle
5) compute Imax, and Imin
6) compute the principal coordinates of each element
7) store element data in an output file)

type material_set = ( concrete, strand, rebar );

element_record = record
  material : material_set;
  area : real;
  m_area : real;
  m_ratio : real;
  x1,x2 : real;
  y1,y2 : real;
  x_centr : real;
  y_centr : real;
  princ_u : real;
  princ_v : real;
end;

Var
  data_from : integer;
  data_out : integer;
  localxx,localyy : real;
  width, height : real;
  dum_char : char;
  Input_file : TEXT;
  input_file_name : string[50];
  number_of_elements : integer;
  counter : integer;
  element : array [1..100] of element_record;
  area : real;
  this_area : real;
  x_area : real;
  y_area : real;
  centroid_x : real;
  centroid_y : real;
  Ixx,Iyy,Ixy : real;
  Imax,inmin,theta : real;
  element_file : TEXT;
  inertia_file : TEXT;
  element_file_name : string[50];
inertia_file_name : string[50];

Procedure INPUT_ELEMENTS ;
Sample PBEAM input file

```pascal
begin
repeat
writeln; writeln;
writeln(' Select the type of input ');
writeln(' 1 = existing file ');
writeln(' 2 = input new data ');
readln(data_from);
until (data_from = 1) or (data_from = 2);
if data_from = 2 then begin
write(' How many elements to be input? ');
readln(number_of_elements);
writeln; writeln;
for counter := 1 to number_of_elements do begin
  write(' What type of element? (Concrete,Strand,Rebar) ');
  readln(element[counter].material_set);
  writeln(' Working on element ',counter:1);
  write(' Lower left corner, x coordin = ');
  read(element[counter].x1);
  write(' y coordin = ');
  readln(element[counter].y1);
  write(' Upper right corner, x coordin = ');
  read(element[counter].x2);
  write(' y coordin = ');
  readln(element[counter].y2);
  write(' What is the modular ratio of the material? ');
  readln(element[counter].m_ratio);
  element[counter].area :=
    (element[counter].x2-element[counter].x1) *
    (element[counter].y2-element[counter].y1);
  element[counter].m_area :=
    element[counter].m_ratio *
    element[counter].area;
  writeln;
end;
writeln;
end;
else begin
  What is the name of the input file to use? '
  readln(input_file_name);
  assign(input_file, input_file_name);
  reset(input_file);
  number_of_elements := 0;
  while not EOF(input_file) do begin
    readln(input_file,
      element[counter].m_ratio ,dum_char,
      element[counter].area ,dum_char,
      element[counter].m_area ,dum_char,
      element[counter].x1,dum_char,
      element[counter].y1,dum_char,
      element[counter].x2,dum_char,
      element[counter].y2);
    number_of_elements := number_of_elements+1;
  end;
end;
end;
```

Procedure COMPUTE_AREA_AND_CENTROID;
begin
  area := 0;
```

- D13 -
Sample PBEAM input file

x_area := 0;
y_area := 0;
for counter := 1 to number_of_elements do begin
  this_area := element[counter].area*
              element[counter].m_ratio;
  area := area + this_area;
  x_area := x_area + this_area *
             0.5*(element[counter].x1+
                  element[counter].x2); 
y_area := y_area + this_area *
             0.5*(element[counter].y1+
                  element[counter].y2);
end;
centroid_x := x_area / area;
centroid_y := y_area / area;
writeln; writeln;
writeln ('The area of the cross-section is ',area:10:2);
writeln('X coordinate of the centroid is ',
 centroid_x:8:3);
writeln('Y coordinate of the centroid is ',
 centroid_y:8:3);
end;

Procedure Compute_x_and_y_from_centroid
begin for counter := 1 to number_of_elements do begin
  element[counter].x_centr :=
    0.5*(element[counter].x1+
        element[counter].x2) - centroid_x;
element[counter].y_centr :=
    0.5*(element[counter].y1+
        element[counter].y2) - centroid_y;
end;
end;

Procedure Compute_inertias
begin
localxx := 0;
localyy := 0;
                    (**********************)
                    (begin by computing local moments of inertia )
                    for counter := 1 to number_of_elements do begin
width := element[counter].x2 -
         element[counter].x1;
height := element[counter].y2 -
         element[counter].y1;
localxx := localxx +
          1/12*width*height*height*height;
localyy := localyy +
          1/12*height*width*height*height;
end;
lxx := 0;
lxy := 0;
lxy := 0;
for counter := 1 to number_of_elements do begin
  lxx := lxx + element[counter].m_area *
        element[counter].y_centr *
        element[counter].y_centr;
lxy := lxy + element[counter].m_area *
        element[counter].x_centr *
        element[counter].y_centr;
lxy := lxy + element[counter].m_area *
        element[counter].x_centr *
end;

- D14 -
Sample PBEAM input file

```
element[counter].y_centre;
Iyy := Iyy + element[counter].m_area *
element[counter].x_centre *
element[counter].x_centre;
end;
Ixx := Ixx + localxx;
Iyy := Iyy + localyy;
end;

Procedure Compute_Imax_Imin_theta;
begin
theta := 0.5 * arctan(-2*Ixy/(Ixx-Iyy));
(in radians)
imax := 0.5*(Ixx+Iyy)+
       sqrt(sqr(Ixx/2-Iyy/2)+sqr(lxy)) ;
imin := 0.5*(Ixx+Iyy)-
       sqrt(sqr(Ixx/2-Iyy/2)+sqr(lxy)) ;
end;

Procedure Display_inertias_and_theta;
begin
writeln; writeln;
writeln('Imax = ',Imax:10:2);
writeln('imin = ',Imin:10:2);
writeln('theta = ',theta:10:4,' radians');
writeln;
writeln('Ixx = ',Ixx:10:2);
writeln('Iyy = ',Iyy:10:2);
writeln('Ixy = ',Ixy:10:2);
writeln;
end;

Procedure File_work;
begin
repeate
writeln;
writeln( 'Do you want to write the data to files? 1 );
writeln( ' 1 = exit without writing data 2 );
write (' 2 = write data and exit 1 );
readln(data_out); untill (data_out=1) or (data_out=2);
if data_out = 2 then begin
writeln;
write('What is the name of the element file? ');
readln(element_file_name);
writeln;
write('What is the name of the inertia file? ');
readln(inertia_file_name);
assign(element_file, element_file_name);
rewrite(element_file);
assign(inertia_file, inertia_file_name);
rewrite(inertia_file);
for counter := 1 to number_of_elements do begin
writeln(element_file,
    element[counter].m_ratio,' ',
    element[counter].area ,',
    element[counter].m_area ,',
    element[counter].x1 ,',
    ...
```
Sample PBEAM input file

element[counter].y1, ' ',
element[counter].x2, ' ',
element[counter].y2);
end;
close(element_file);
writein(inertia_file,
'Data generated using the data file ',
element_file_name);
writein(inertia_file);
writein(inertia_file,'Imax = ',imax:10:2);
writein(inertia_file,'Imin = ',imin:10:2);
writein(inertia_file,'Theta = ',Theta:10:4,
' radians');
writein(inertia_file);
writein(inertia_file,'lxx = ',lxx:10:2);
writein(inertia_file,'lyy = ',lyy:10:2);
writein(inertia_file,'lxy = ',lxy:10:2);
close(inertia_file);
end;

BEGIN {main program}
Input_elements;
compute_area_and_centroid;
compute_x_and_y_from_centroid;
compute_inertias;
compute_Imax_Imin_theta;
display_inertias_and_theta;
file_work;
END.
Program Extract_life

program extract_life ( input , output ) ;

(This program extracts stress ranges from a file generated MS_EZ. The user inputs the name of an input file, and values for the upper and lower moments on the cross section. The program then opens the data file and then searches through the file for moments larger and smaller than the ones input. The stress corresponding to the input moment is determined by proportioning the stress in the same manner as the moments. Created Jan 10, 1991 by Steve Olson. Once the stress range is determined, the expected life is computed using Paulson's fatigue model. Also output is the mean fatigue life from Tide and Van Horn. )

VAR

    input_file : TEXT ;
    input_file_name : string[80] ;
    counter : integer ;
    i : integer ;
    lower_trig : integer ;
    upper_trig : integer ;
    dum_char : char ;
    dum_val : real ;
    this_moment : real ;
    this_stress : real ;
    max_moment : real ;
    min_moment : real ;
    lower_moment : real ;
    upper_moment : real ;
    lower_stress : real ;
    upper_stress : real ;
    stress_range : real ;
    stress : array[1..200] of real ;
    moment : array[1..200] of real ;
    exit_condition : boolean ;
    loop : integer ;
    ult_stress : real ;
    paulson : real ;
    tide_vanhorn : real ;

function ten_x (x:real) : real ;
begin
    ten_x := exp(x*ln(10)) ;
end;

function log_x_base_10 ( x:real) : real ;
begin
    log_x_base_10 := ln(x) / ln(10) ;
end;

procedure get_inputs ;
begin
    writeln ; writeln ;
    writeln('This program extracts stress ranges from data files') ;
    writeln('created by MS_EZ.' ) ;
    writeln ; writeln ;
    write('What is the name of the input file? ') ;
    readln(input_file_name) ;
    writeln ;
end ;

- D17 -
Sample PBEAM input file

procedure open_file;
begin
assign(input_file,input_file_name);
reset(input_file);
end;

procedure get_max_min_moments;
begin
readln(input_file); {move beyond header line}
min_moment := 1000000;
max_moment := 0;
counter := 1;
while not eof(input_file) do
begin
for i := 1 to 3 do read(input_file,dum_val,dum_char);
readln(input_file,this_moment,dum_char,dum_val,dum_char,
this_stress);
if this_moment > max_moment then max_moment := this_moment;
if this_moment < min_moment then min_moment := this_moment;
moment[counter] := this_moment;
stress[counter] := this_stress;
counter := counter + 1;
end;
counter := counter - 1; {let counter = actual load stages}
end;

procedure get_more_input;
begin
writeln('Input moments can vary between'
,min_moment:8:0, 'and ',
max_moment:8:0, ' kip-inches');
writeln;
write('For the stress range, what is the lower moment? ');
readln(lower_moment);
writeln;
write('What is the upper moment? ');
readln(upper_moment);
end;

procedure get_stress_range;
begin
lower_trig := 0;
upper_trig := 0;
for i := 1 to counter do begin
this_moment := moment[i];
if (this_moment>lower_moment) and (lower_trig=0)
then lower_trig := i;
if (this_moment>upper_moment) and (upper_trig=0)
then upper_trig := i;
end;
lower_stress :=
{(lower_moment - moment[lower_trig-1]) /
(moment[lower_trig]-moment[lower_trig-1]) *
(stress[lower_trig]-stress[lower_trig-1])} +
stress[lower_trig-1];
upper_stress :=
{(upper_moment - moment[upper_trig-1]) /
(moment[upper_trig]-moment[upper_trig-1]) *
(stress[upper_trig]-stress[upper_trig-1])} +
stress[upper_trig-1];
Sample PBEAM input file

```
stress_range := upper_stress - lower_stress;
end;

procedure lives;
begin
  ult_stress := 250;
paulson := ten_x(11.3.5*log_x_base_10(stress_range));
tide_vanhorn := ten_x(10-3.6*log_x_base_10C100*stress_range/ult_stress));
end;

procedure ask_for_more;
begin
  writeln; writeln;
  writeln('lower moment=','lower_moment:B:O','kip-inches');
  writeln('lower moment strand stress = ','lower_stress:6:1);
  writeln;
  writeln('upper moment=','upper_moment:B:O','kip-inches');
  writeln('upper moment strand stress=','upper_stress:6:1);
  writeln;
  writeln('The stress range between these moments is ','stress_range:6:1','ksi');
  writeln;
  writeln('Paulson fatigue life is ','paulson:10:0','cycles');
  writeln('Tide and Van Horn fatigue life is ','tide_vanhorn:10:0','cycles');
  writeln; writeln;
  writeln('Would you like to determine another stress range from this file? ');
  write('('Y/N'));
  readln(dum_char);
  writeln; writeln;
  if (dum_char = 'n') or (dum_char = 'N') then exit_condition := true;
end;
```

procedure close_file;
begin
  close(input_file);
end;

begin
  get_inputs;
  open_file;
  loop := 1;
  exit_condition := false;
  repeat
    if loop = 1 then get_max_min_moments
    get_more_input;
```
Sample PBEAM input file

get_stress_range;
loop := loop + 1;
lives;
ask_for_more;
until exit_condition;
close_file;
end.