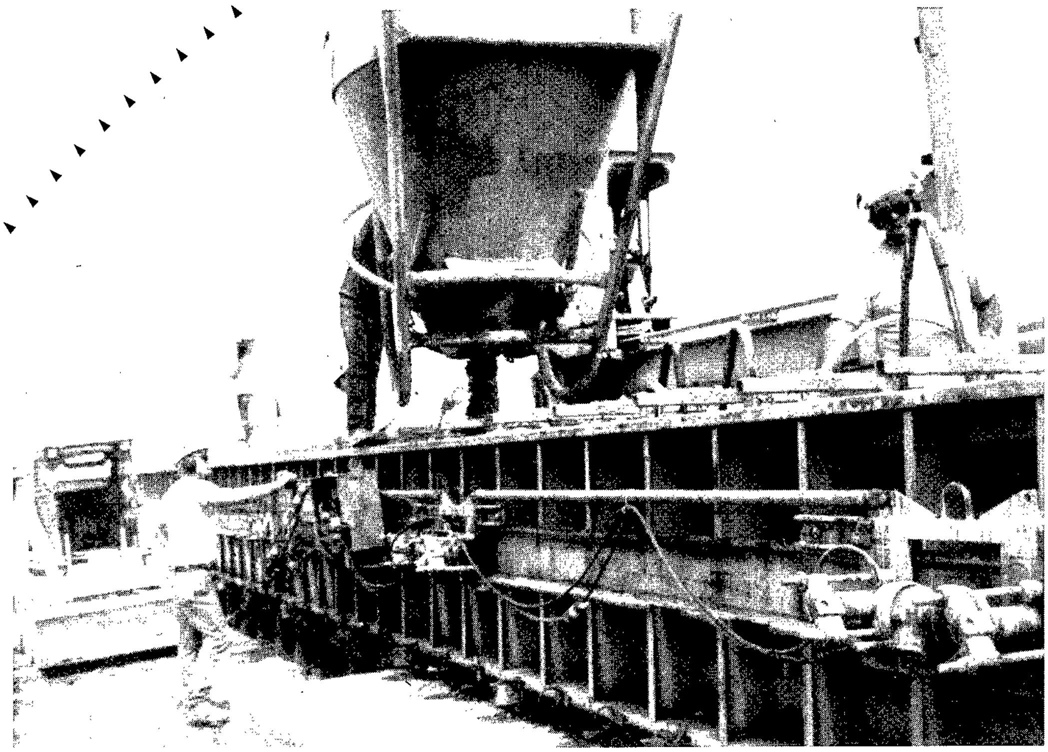
Final Report 1998-09

PB98-157464



Instrumentation and Fabrication of Two High-Strength Concrete Prestressed Bridge Girders

UNIVERSITY OF MINNESOTA
CENTER FOR TRANSPORTATION STUDIES

minnesota prestress association

This research was made possible with the support and contributions from the Minnesota Prestress Association. Many of its members took a role in the fabrication and testing of the girder specimens.

Technical Report Documentation Page

1. Report No. MN/RC - 1998/09	2.	3. Recipient's Accession No.
4. Title and Subtitle INSTRUMENTATION AND FABRICATION OF TWO HIGH-STRENGTH CONCRETE PRESTRESSED BRIDGE GIRDERS		5. Report Date January 1998
		6.
7. Author(s) Jeffrey Kielb, Roberto T. Leon, Catherine E. French, Carol K. Shield		8. Performing Organization Report No.
9. Performing Organization Name and Address Department of Civil Engineering University of Minnesota 500 Pillsbury Dr. S.E. Minneapolis, MN 55455-0220		10. Project/Task/Work Unit No.
		11. Contract (C) or Grant (G) No. (C) 69098 TOC # 83
12. Sponsoring Organization Name and Address Minnesota Department of Transportation 395 John Ireland Boulevard Mail Stop 330 St. Paul, Minnesota 55155		13. Type of Report and Period Covered Final Report 1993-1998
		14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract (Limit: 200 words)

This report describes the design, instrumentation, construction, and test set-up of two high-strength concrete prestressed bridge girders.

The girder specimens were constructed to evaluate prestress transfer length, prestress losses, flexural fatigue, ultimate flexural strength, and ultimate shear strength. Each test girder was a 132.75-foot long, 46-inch deep, Minnesota Department of Transportation (Mn/DOT) 45M girder section reinforced with 46 0.6-inch diameter 270 ksi prestressing strands. The 28-day nominal compressive strength of the girders was 10,500 psi. Each girder was made composite with a 9-inch thick, 48-inch wide composite concrete deck cast on top with a nominal compressive strength of 4000 psi.

Girder I used a concrete mix incorporating crushed limestone aggregate while Girder II utilized round glacial gravel aggregate in the mix with the addition of microsilica. In addition, the two test girders incorporated two different end patterns of prestressing--draping versus a combination of draping and debonding--and two different stirrup configurations--standard Mn/DOT U versus a modified U with leg extensions. More than 200 strain gages were imbedded in each girder during construction. Other reports present flexural and shear testing results.

17. Document Analysis/Descriptors long-span prestressed bridge girders high performance concrete instrumentation fabrication test setup		18. Availability Statement No restrictions. Document available from: National Technical Information Services, Springfield, Virginia 22161	
19. Security Class (this report) Unclassified	20. Security Class (this page) Unclassified	21. No. of Pages 190	22. Price

INSTRUMENTATION AND FABRICATION OF TWO HIGH STRENGTH CONCRETE PRESTRESSED BRIDGE GIRDERS

Final Report

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January 1998

Published by

Minnesota Department of Transportation
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ACKNOWLEDGMENTS

The sponsors for the project were the Minnesota Department of Transportation, the Minnesota Precast Association (MnPA), the Center of Transportation Studies-University of Minnesota, and the Precast Prestressed Concrete Institute. In addition, the following companies provided goods and services: Elk River Concrete Products fabricated the test girders; Lefebvre & Sons Trucking transported the girders to the testing site; and Truck Crane Service Co. and Golden Valley Rigging installed the girders into the off-campus testing laboratory. In addition, Union Wire & Rope, Atlas Foundation Co., Simcote Inc., W.R. Grace & Co., Carl Borg Inc., Progressive Contractors Inc., Midspec Inc., C.S. McCrossan Inc., and Marv Durst at Mn/DOT provided materials and technical expertise. The help of fellow graduate students, especially Steve Hearn, and undergraduate assistants John Ellingson and Eric Leagjeld, and Mike Lins and the machine shop personnel is gratefully acknowledged.

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Executive Summary

This work represents part of an overall project to investigate the applications of high strength concrete to prestressed bridge girders. The entire project addressed the mechanical properties of high strength concrete (Mokhtarzadeh and French, *Mechanical Properties of High-Strength Concrete*), durability issues with high strength concrete (Kriesel, French and Snyder, *Freeze-Thaw Durability of High-Strength Concrete*) and the testing of two high strength prestressed bridge girders (Alhborn, French and Shield *Longterm Behavior and Flexural Tests of High-Strengh Concrete Prestressed Bridge Girders*; and Cumming, French and Shield, *Shear Capacity of High-Strength Concrete Prestressed Bridge Girders*).

Recent research has shown it feasible to fabricate prestressed concrete bridge girders using concrete compressive strengths in excess of 12,000 psi. The current design provisions for these structures were based on empirical relationships developed from tests on specimens with compressive strengths up to 6000 psi. A major part of the high strength concrete bridge girder project addresses the adequacy of the current design provisions for high strength concrete.

This report describes the design, instrumentation, construction, and test setup of the two high strength concrete prestressed bridge girders. The girder specimens were constructed to evaluate prestress transfer length, prestress losses, flexural fatigue, ultimate flexural strength, and ultimate shear strength. Each test girder was a 132.75-ft. long, 45-in. deep Mn/DOT 45M girder section reinforced with forty-six 0.6-in. diameter 270 ksi prestressing strands. The 28-day nominal compressive strength of the girders was 10,500 psi. Each girder was made composite with a 9-in. thick, 48-in. wide composite concrete deck cast on top with a nominal compressive strength of 4000 psi. Girder I used a concrete mix incorporating crushed limestone aggregate while Girder II utilized round glacial gravel aggregate in the mix with the addition of microsilica. In addition to investigating the two concrete mix designs, two different end patterns of prestressing (draping vs. a combination of draping and debonding), and two different stirrup configurations (standard Mn/DOT U vs. a modified U with leg extensions) were incorporated into the two test girders. Over 200 strain gages were imbedded in each girder during construction. Results of the testing, both flexural and shear, are contained in other reports (Ahlborn et. al. and Cumming et. al.).

No problems were encountered in the construction and transportation of the girders. Concrete placement and curing was performed by Elk River Concrete using standard techniques. The girders were transported using standard trucking from the prestressing yard to a test facility with no signs of stability or handling problems.

Chapter One - Introduction

1.1 Advantages of High Strength Concrete

The use of higher strength concrete (>7,000 psi) is the current trend in the prestressed concrete bridge girder industry. In recent years, the Minnesota Department of Transportation (Mn/DOT) has increased its girder design strength from 4,500 psi to 8,000 psi in special situations. Since the design strength is only a minimum and the strength at release generally governs the design, it is not uncommon to have a 6000 psi concrete design mix achieve 10,000 psi in a year.

Taking advantage of high strength concrete (HSC) would be beneficial to the design of prestressed bridge girders for several reasons. With increased concrete strength, the maximum span length can be increased; this would decrease the number of supports needed and provide greater underpass widths. Figure 1.1 [1] shows example results of a parametric study on Mn/DOT 45M girders conducted by Ahlborn [1]. The plot illustrates the required number of strands versus span length for nominal concrete compressive strengths of 7,000 and 10,000 psi at three girder spacings (4, 7 and 10 ft.). It can be seen from this plot that at a given girder spacing, HSC can be used to increase the maximum span length on the order of 12 to 18 percent. In addition, for a given span length (e.g. 80 ft.), HSC can be used to increase the maximum girder spacing (e.g. from 7 to 10 ft.). The use of HSC is limited, however, by the number of strands that can be placed in the cross section (Figure 1.2 [1]). As more and more strands are placed in the cross section their eccentricity decreases and consequently their effectiveness is reduced. As shown in Figure 1.2 [1], there is little benefit in using 12,000 psi concrete over 10,000 psi concrete.

Figure 1.3 [1] shows an example of the benefits which might be achieved by increasing girder spacing. The example is for a bridge of a given width and length fabricated using 81I girders. The girder design was optimized by reducing the number of girders and strands required by increasing the concrete compressive strength from 7,000 psi to 10,000 psi. From the figure it can be seen that 38 percent fewer girders and 13 percent fewer strands would be required for the bridge design with the HSC. Fewer girders and prestressing strands required would result in an economic benefit through reduced fabrication and transportation costs along with faster erection time.

For a given girder length and spacing, a shallower HSC cross section could be utilized. Figure 1.4 [1] shows the effect of varying concrete strength (7,000 to 12,000 psi) on the maximum span length for five different prestressed bridge girder sections with a 4 ft. girder spacing. It can be seen that the depth of the girder section required for a given span length can be reduced by at least one size (9 in. in depth) with an increase in concrete strength from 7,000 to 12,000 psi. This application could increase overpass clearances or reduce bridge roadway elevation. An economic saving through reduced construction materials and earth grading would be achieved.

1.2 Summary of Preliminary Parametric Study

The parametric study [1] described in the previous section was used to determine the maximum girder span length without violating American Association of State Highway and Transportation Officials (AASHTO) or Minnesota Department of Transportation (Mn/DOT) design specifications [2,3]. Variables considered in the study were concrete strength, girder spacing, prestressing strand diameter, number of strands and strand pattern.

The parametric study showed that minimal girder spacing is needed to achieve maximum length. Typically for the narrow spacing design using HSC, tension in the bottom fiber at midspan will control at final service condition because the design is limited by the amount of prestressing. Compression in the top fiber of the same cross section will control in normal strength concrete since the allowable compressive stress is much smaller. It was found that a narrow girder spacing design is controlled by the stresses at final service conditions rather than at the time of release of the prestressing force because a girder with narrow spacing carries a smaller portion of the bridge dead and live loads. The majority of the narrow-spaced girder load is due to the girder self weight. This means that less prestress force needs to be stored at the time of release and therefore the concrete strength at release does not control. This is beneficial to the fabricator since it takes less curing time for the lower release strength, thereby shortening the turn-around time for the prestressing bed.

A girder design with a wider spacing will be controlled by midspan compressive stress in the bottom fiber at release conditions (mild steel reinforcement is typically installed in the top flange for excessive tensile stresses). A much larger portion of the required prestressing force is needed to offset the stresses due to applied loads at final service conditions, but at release the girder self weight is not sufficient to control the compressive prestress at midspan. For this condition compression in the bottom fiber or tension in the top fiber at midspan controls at release. In both cases (i.e. narrow and wide girder spacings) the stresses at the girder ends are controlled by either draping or debonding of the strands.

The parametric study focused on Mn/DOT I-girders; these types of cross sections have the same flange dimensions and web widths; the only difference is the web heights. Because only a given number of strands will fit in the lower flange, once the flange is filled any additional strands placed in the web decrease the prestress eccentricity. This decreases the effectiveness of the HSC.

As noted with reference to Figure 1.2, a large amount of prestressing force at a large eccentricity is required in the girder cross section to take full advantage of HSC. The use of the industry standard (1/2 in. diameter 270 ksi prestressing strand) would quickly limit the maximum girder length because of the limited number of strands that would fit into the cross section. Although not commonly accepted in the prestressed bridge girder industry yet, the use of 0.6 in. diameter 270 ksi low-relaxation strand would allow a 49% increase in prestressing force for the same number of strands.

The shallowest I-girder Mn/DOT typically uses in bridge construction is the 45M. By using forty-six 0.6 in. diameter 270 ksi strands and 10,500 psi concrete at 28 days, the span of the 45 in. deep girder was maximized to 132 ft. 9 in. for a 4 ft. girder spacing. This length is 30% longer than the typical 45M girder design utilizing 7,000 psi concrete and 0.5 in. diameter 270 ksi strands.

1.3 Organization of Report

Chapter Two of the report describes the objective of the girder testing program along with the design of the test girders. Items discussed are girder design loads, prestress losses, flexural strength and shear capacity.

The materials used for the construction of the test girders are discussed in Chapter Three. The design and test strengths of the concrete, prestressing strands and mild steel reinforcement are reported in this chapter.

Included in Chapter Four is a description of the instrumentation and data acquisition system used for the test girders. Instrumentation used to investigate the transfer length and other release conditions, concrete creep and shrinkage, flexure and fatigue, and shear are described.

Chapter Five presents the methods by which the test girders were fabricated. It includes the installation of the instrumentation, placing the reinforcement and formwork, casting and curing of the concrete, and the release of the prestress force.

The transportation of the test girders from the fabricator's facility to the testing site is discussed in Chapter Six. This chapter also includes an explanation of how the girders were handled and the installation of them into the off-campus testing laboratory.

The fabrication of the composite concrete bridge deck is described in Chapter Seven. The materials used, the formwork, the instrumentation and the casting and curing processes are discussed.

The test setup is described in Chapter Eight. The load frames and hydraulic loading system are discussed. A brief description of the future static, fatigue, ultimate flexure and ultimate shear tests are given.

Chapter Nine presents conclusions based on the experience of designing, instrumenting and fabricating the two full sized high strength concrete prestressed bridge girder test specimens.

The body of the report is followed by six appendices. The computer output regarding the design of the girders is given in Appendix A. Sample calculations regarding the ultimate flexural and shear strengths are given in Appendix B. Appendix C contains the batch data for the high strength concrete mixes used in the girders. The description and locations of the instrumentation for the precast girders are given in Appendix D. Appendix E contains the information on the instrumentation used in the composite deck. A copy of the field log kept during the construction process can be found in Appendix F.

Chapter Two - Design of Bridge Girders

2.1 Objective

The primary objective of the overall research program was to investigate applications of HSC to the prestressed bridge girder industry. The research program comprised two main parts: HSC materials research and tests of two HSC prestressed bridge girders. The objective of the HSC materials research, conducted by Alireza Mokhtarzadeh, was to investigate the effect of varying materials available from local sources on the properties of HSC [4]. The second part of the program was to evaluate the adequacy of present AASHTO [2] design specifications for prestressed concrete bridge girders utilizing HSC. To achieve the goals of the latter objective, two full-sized HSC prestressed bridge girders were constructed to investigate the effect of HSC on prestress losses, transfer length, camber, flexural fatigue, and the ultimate strengths in flexure and shear. This report describes the design, fabrication, instrumentation, and test setup of the two girders.

2.2 Assumptions Used in Design

2.2.1 Girder Dimensions

It was the intent in the designing the girders to maximize the girder span without violating the analysis and design parameters given by AASHTO [2] and Mn/DOT [3] design specifications. The design of the two test girders was intended to take full advantage of HSC and 0.6 in. diameter 270 ksi low-relaxation prestressing strand. The type of cross section used for these test girders was a Mn/DOT Type 45M I-girder (Figure 2.1). From the parametric study discussed in Chapter One [1], the minimum girder spacing investigated (4 ft.) was used for the girder design. With this girder spacing, a design using a maximum span length of 132 ft.- 9 in. was achieved.

The test girder design calculations were aided by the use of computer software specifically written to design prestressed concrete bridge girders using the current AASHTO [2] specifications. The computer software was Span v0.1 developed by Leap Software, Inc. Tampa, Florida and modified for this research project to include concrete strengths above 10,000 psi. The final design runs using this software for each test girder can be found in Appendix A. It was determined from the design runs that the length of the girder was governed by the amount of longitudinal prestressing steel in the bottom of the section at centerline. The center-to-center strand spacing was fixed at 2 in. in both the vertical and horizontal directions due to the template on the prestressing bed used. The limiting parameter of the design was tension at the bottom fiber of the cross section and because only a given amount of prestressing steel would fit into the bottom flange the maximum span length could be calculated. Forty-six 0.6 in. diameter, 270 ksi low-relaxation prestressing strands were used in each test girder. See Figure 2.2 for the strand pattern at the midspan of the test girders.

Because the maximum span length design was governed by tension in the bottom fiber at midspan, additional concrete strength would not economically improve the design for a given amount of prestressing. It was found that the optimal design concrete strength for the long span 45M girders was 10,500 psi.

Each of the two test girders had a different concrete mix design. One girder was fabricated using a limestone aggregate mix; the other girder was cast with microsilica in a glacial gravel aggregate concrete mix. Both mixes had a required 28 day compressive strength of 10,500 psi. At the time of release, 18 hours after casting of the concrete, the concrete was required to have 85% of its design 28 day strength or 8,925 psi to satisfy the allowable stresses at this time.

Two different schemes of controlling end stresses were used in the girder specimens. One girder end used draped strand exclusively, while the other three girder ends used a combination of draped and debonded strands. Figure 2.3 shows the strand patterns and the debonded strands used at each of the four test girder ends. A more complete explanation of these two end stress controlling methods can be found in Section 2.3.3.2 of this chapter.

In order to investigate the effect of stirrup configuration on shear strength, two different stirrup configurations were used. In addition to the typical Mn/DOT U-type stirrup, an experimental type of stirrup (modified U stirrup shown in Figure 2.10) with additional anchorage provided by a 90 degree bend at the bottom was installed in 3 of the 4 girder ends. All stirrups were No. 4, epoxy coated reinforcing bars. Figure 2.4 shows the design variables used in the two test girders. Each of the four girder ends were different with respect to one variable only: either the concrete mix, stirrup type, or the method of controlling the end stresses (draping vs. draping/debonding combination).

2.2.2 Bridge Dimensions

The design span length of the prestressed concrete girder bridge for this test was 131 ft.-6 in. center-to-center of the supports. The distance from the end of the girder to the centerline of the support was 7.5 in.; this gave a total girder length of 132 ft.-9 in. The span length at release (supported at its ends due to camber) of the prestressing force was also 132 ft.-9 in.

The width of the design bridge, shown in Figure 2.5, was 52 ft. with the curb to curb distance being 48 ft. The bridge was designed to accommodate two, 12 ft. wide lanes of traffic in each direction. The girders in the bridge system had a center-to-center spacing of 4 ft. that gave a total of 13 girders required for this longitudinal stringer girder system. The design girders represented interior girders from this system.

2.2.3 Composite Concrete Deck

On top of the 45 in. deep precast girder section, a composite 4000 psi concrete deck slab was cast as shown in Figure 2.6. The fabrication of the deck slab was done using unshored construction to simulate actual bridge building techniques. The effective width of the deck, above each 30 in. wide girder top flange, was 48 in. Mn/DOT bridge construction typically uses a variable haunch, as wide as the girder upper flange, in the bottom of the deck to smooth out the girder camber for multiple span bridge systems. The test girders incorporated a constant 1 in. haunch (or stool) to simplify the construction of the composite deck and keep the cross section prismatic.

For the initial girder design, and as typically used in Mn/DOT bridge construction, the slab was to be 8 in. thick above the girder flange and 7 in. thick between girders; on top of this was to be cast a 2 in. thick low-slump 4000 psi concrete wearing course. The low-slump concrete is typically used to inhibit the penetration of harmful de-icing salts into the deck and steel reinforcement. In order to ease in the construction of the test girders it was decided to cast the deck and the wearing course as one continuous slab 10 in. thick over the girders and 9 in. thick in the gap between girders. In actual bridge construction the composite girder/deck would work together in supporting the 2 in. wearing course. Casting the deck and wearing course together would increase the shrinkage of the deck and decrease the girder camber when compared to the actual bridge construction case.

The composite concrete deck was designed using the Mn/DOT Bridge Design Manual [3]. From the design aids given in the bridge design manual, the detailing parameters of the mild steel reinforcing can be determined for a given girder spacing and live load design truck. The concrete slab reinforcement table for the HS25 design truck uses 1993 AASHTO [2] load factor design and assumes design parameters of: 4 ksi concrete in the deck, 60 ksi rebar, impact factor of 30%, allowance for 17 psf future wearing course, and deck continuous over three or more girders. Because the concrete deck of each test girder was relatively narrow, no main reinforcement perpendicular to the axis of the girder was installed. Only shrinkage and temperature steel was installed in the perpendicular direction as prescribed by the ACI Code, Section 7.12 [5]. The mild steel reinforcement installed into the concrete deck of the test girder is shown in Figure 2.7. All mild steel reinforcement in the decks and in the girders was epoxy coated.

2.3 AASHTO/Mn/DOT Test Girder Design

2.3.1 Design Loads

In the design of the test girders the loading schemes as prescribed by the AASHTO Specifications, Section Three [2] were used. It was required by the specifications to check the state of stress in the girders at many stages of loading during their fabrication and service life. Important stages in construction are: the time of prestress transfer, when the girder is lifted from the prestressing bed, when the girder is set onto its final supports, and when the composite deck is cast. Important stages during the service life of the girders are: final service conditions, and factored dead and live loads at critical sections considering final prestress losses. The gravity loads used in the design and the testing of the test girders are shown in Table 2.1.

2.3.1.1 Loads at Release

At the time of prestress transfer (release), the gravity loading on the girder was due to the self-weight of the girder, 671.7 plf was calculated based on the gross area of the precast cross section (624 in^2) and its density (155 pcf). It was assumed at release that the girder would camber up so the girder would only be supported at its ends like a simple beam with a span equal to the total length of the girder, 132 ft.-9 in. Stresses were then calculated at critical sections due to the resultant stresses of the self-weight, the axial prestressing force, and the moment due to the eccentricity of the centroid of the prestressing force from the geometric centroid of the cross section.

2.3.1.2 Dead Loads

When the girders were lifted from the prestressing bed, the state of stress in the girders changed due to the location of the lift hooks from the end of the girders. In determining the location of the lift hooks, the stability of the girder being lifted along with the cross-sectional stresses was considered. The dead load was the same as at transfer.

When the girders were set onto their final supports, the stresses were calculated as for transfer. The only differences were that the unsupported span length was 1 ft.-3 in. shorter because of the location of the final supports (less flexural moment) and the effective prestressing force (less moment due to strand eccentricity and less net compressive force on section) was reduced due to time-dependent losses.

A superimposed dead load of 20 plf was added to the noncomposite design girder section to account for diaphragms between girders before the composite deck was cast. This loading was used to simulate actual design loads and was never actually applied to the test girders. In order to simulate the most typical construction technique used in industry, the composite concrete deck was cast using unshored formwork construction. The loading for this case would include the previous loading plus additional dead load to account for the wet concrete and formwork. In the calculation of stresses, it was assumed that the concrete deck did not act compositely with the girder section until it was fully cured.

The final superimposed design dead load was applied to the composite girder section. This was a gravity load due to bridge guard rails. For the first interior girders, which is what is assumed for the test girders, a J-type guard rail load of 203 plf was used. It was assumed that the fascia girder resists 2/3 of the guard rail weight and the first interior girder resists the other 1/3. The rail dead load was used as a design load and will be applied to the composite test girders before the fatigue testing.

2.3.1.3 Live Loads

The live load that controlled the design of the girders was the moving load of a HS25 truck (a military axle loading was also considered but did not govern). The HS25 design truck is

1.25 times the axle loads of an HS20 truck; the HS25 truck is used by Mn/DOT designs because it closely simulates actual trucks on Minnesota roads today. The HS25 design truck, shown in Figure 2.8, consists of a tractor with a semi trailer and has a total of three axles. The front axle load carries 10,000 lbs. and the two rear axles carry 40,000 lbs. each. A variable axle spacing of 14 to 30 ft. is allowed in the design, but a 14 ft. spacing between all truck axles controlled the design. The load of the truck was assumed to be distributed across a 10 ft. width (AASHTO Specifications, Article 3.7.1.2 [2]) even though the center-to-center distance between the sets of dual tires on each axle was 6 ft. In order to determine the maximum effect of the moving truck on the bending and shear stresses in the girders, shear and moment envelopes were created by moving the HS25 truck across the span and calculating the effect at critical sections: at every 0.1 of the span length and at $h/2$ from the ends.

The AASHTO Specifications, Article 3.23.2.2 [2] requires distribution of the design truck wheel loads (one half the axle loads) to the interior girder for bridges with two or more lanes of traffic as the ratio of the girder spacing divided by 5.5. This gave a distribution factor of 0.727 wheels per girder or 0.364 axles per girder. The load of the design truck was further amplified by an impact factor to account for vertical velocity of the truck mass due to the bouncing of the moving truck on an uneven pavement. The impact factor is given by AASHTO Specifications, Article 3.8.2.1, equation 3-1[2].

$$I = \frac{50}{L+125}$$

I = impact factor (maximum 30 percent)
L = length in ft. of the portion of the span that is loaded to produce the maximum stress in the member

This equation gave an impact factor of 1.195 using the test girder design parameters. A HS25 uniform distributed live load of 800 lbs. per linear foot was used to simulate lane loading, but did not govern the design.

2.3.1.4 Load Factors and Combinations

To determine the total effect of all of the loads on the test girders in the final condition, AASHTO Specifications [2] Group loading combinations for Service Load Design and Load Factor Design given by Article 3.22, equation 3-10 (shown below) and Table 3.22.A were applied to the design loads.

$$Group I = \gamma\left[\beta_D D + \beta_L (L + I)\right]$$

γ = load factor, 1.0 for Service Load Design or 1.3 for Load Factor Design from Table 3.22.A [2]
β_D = coefficient, 1.0 for flexural members from Table 3.22.A
β_L = coefficient, 1.0 for Service Load Design or 1.67 for Load Factor Design from Table 3.22.A
D = dead load
L = live load
I = live load impact, as shown above

Because the test girder design accounted only for gravity loads, Group I coefficients controlled both the Service Load and Load Factor Design.

2.3.2 Prestress Losses

2.3.2.1 Total Losses

It is expected that some of the initial prestressing force in the steel strands is lost over time. This loss adversely affects the performance of concrete bridge girders regardless of the strength of concrete. The total time dependent loss of the prestressing force is made up of several components, including loss due to the elastic shortening of the prestressing strand at release, loss due to shrinkage of the concrete, loss due to creep of the concrete, and loss due to relaxation of prestressing strand. Total losses are found by summing these components as given by equation 9-3 in Article 9.16.2 of the AASHTO Specifications [2] shown below. Table 2.2 shows the estimated losses expected in the test girders. All units of these equations are in psi.

$$\Delta f_s = SH + ES + CR_c + CR_s$$

Δf_s = total loss excluding friction
SH = loss due to concrete shrinkage
ES = loss due to elastic shortening
CR_c = loss due to creep of concrete
CR_s = loss due to relaxation of prestressing steel

$$SH = 17{,}000 - 150RH$$

RH = mean average ambient relative humidity in percent (given in Figure 9.16.2.1.1 [2])

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

E_s = modulus of elasticity of prestressing strand, assumed to be 28,500,000 psi

E_{ci} = modulus of elasticity of concrete in psi at transfer of stress

f_{cir} = concrete stress at the center of gravity of the prestressing steel due to prestressing steel and dead load of beam immediately after transfer; it shall be computed at sections of maximum moment

$$CR_c = 12 f_{cir} - 7 f_{cds}$$

f_{cds} = concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied

$$CR_s = 5{,}000 - 0{,}10ES - 0{,}05(SH + CR_c)$$

for 250 - 270 ksi low relaxation prestressing strand

2.3.2.2 Immediate Losses

The AASHTO Code gives empirical relations to calculate these loss components at the time of release and final conditions. At the time of transfer of the prestressing force, the prestress losses were due to the elastic shortening and relaxation of the strands; there were losses of the initial pull of the strands due to the seating of chucks holding the strands in tension, but this is not taken into consideration. The AASHTO Specifications [2] do not include the relaxation of the prestressing strands before release as a component of the total prestressing force loss. The relaxation of the prestressing strands prior to release is a time dependent loss that occurs before the prestress is transferred to the girder section. This loss, RET, is estimated for low-relaxation prestressing strands by the following equation given by the PCI Committee Report [6]:

$$RET = f_{st}\left[(\log 24t - \log 24t_1)/45\right]\left(\frac{f_{st}}{f_{py}} - 0.55\right)$$

$f_{st}/f_{py} - 0.55 \leq 0.05$

f_{st} = stress in prestressing steel at time t_1, psi

f_{py} = stress at 1% elongation of prestressing steel; assumed to be 0.9 of the ultimate strength of the strand, psi
t = time at end of time interval, days
t_1 = time at beginning of time interval; at the time of anchorage, t_1 is taken as 1/24 of a day so that $\log t_1$ is zero, psi

For the 270 ksi low-relaxation prestressing strand, the initial pull on the strand was assumed to be 75% of the ultimate strength after seating. For the 270 ksi strand used, the initial pull was 202.5 ksi. Of the 202.5 ksi initial stress in the strands, 19.7 ksi was estimated to be lost to elastic shortening and 1.8 ksi to the relaxation of the strand for a total loss of 10.6% at transfer of the prestressing force.

2.3.2.3 Final Losses

At the final condition, the total losses included the losses at release plus the losses due to the concrete shrinkage of concrete (5.8 ksi), concrete creep (37.6 ksi), and strand relaxation (0.9 ksi). This gave a total loss of the prestressing force of 63.9 ksi or 31.5% of the initial stress.

2.3.3 Design Strengths

The required design strengths of the test girders were estimated using equations contained in the AASHTO Specifications, Section 9, Articles 9.17 for flexural strength, 9.18 for ductility limits, 9.20 for shear, and 9.21 for anchorage zones [2]. In the design of composite prestressed concrete bridge girders, capacities must be checked at intermediate construction periods as well as at transfer and final service conditions.

2.3.3.1 Strengths at Transfer

During the fabrication of the prestressed girders, stresses at transfer, caused by the combination of the prestressing load and dead load effects, were not to exceed the allowable stresses given by Article 9.15.2.1 [2]. The maximum allowable compressive stress at any cross section of the girder was $0.6f'_{ci}$. The minimum allowable concrete strength at release, f'_{ci}, was assumed to be 0.85 of the required 28 day strength of the concrete; for the test girders $f'_{ci} = 8925$ psi. This gave a maximum allowable compressive stress of 5355 psi at transfer. In the tension zone, the Specifications [2] allow a maximum tensile stress of 200 psi or $3\sqrt{f'_c}$ for areas with no bonded reinforcement. Although the test girders had reinforcement in the top flange, it was ignored and a 200 psi maximum tensile stress was allowed so no cracking would occur at transfer.

It was required to calculate the stresses in the girder section at critical locations. These locations for the test girders were: at the end transfer length, a distance of half the depth of the girder from the support, at $0.1L$ intervals along the span length, at the draping point, and at midspan. For girders with debonded strands, stresses must be checked at locations where debonding occurs. It was assumed the prestressing force was transferred to the concrete over a distance of 50 diameters of the prestressing strand (Article 9.20.2.4 [2]); the transfer length was assumed to be 30 in. for the 0.6 in. diameter strands used in the test girders. At the free ends of the girder there was no prestressing force, and the force was assumed to be linearly distributed into the girder to the end of the transfer length.

2.3.3.2 Strengths at Final

At service conditions, the allowable stresses in the concrete (given in Article 9.15.2.2 [2]) were calculated using the required 28 day strength of the concrete (10,500 psi). The maximum allowable compressive stress in the concrete was $0.4f'_c$; for the girders that stress was 4200 psi. The maximum allowable tensile stress for cross sections with bonded reinforcement was given by $6\sqrt{f'_c}$, or 615 psi for the test girders. The modulus of rupture for the girders was given by $7.5\sqrt{f'_c}$ or 769 psi in tension for the girders.

Because flexural stresses at both transfer and final conditions were considered, the design process was iterative and was controlled by tensile stresses at the extreme bottom fiber of the girder cross section. The only way to control these stresses without reducing the girder length was to increase the effective prestressing force along the span while maximizing its eccentricity. This situation limited the maximum attainable length of the test girders because of the amount of prestressing strands that could fit in the lower flange.

In the span of the test girders, stresses due to the prestressing force are balanced with the stresses due to flexural bending. At the end regions of the simply supported girders, there is virtually no flexural stress to balance prestress, leading to high compressive stresses in the bottom fibers and large tensile stresses in the top fibers. To control these stresses in the end regions of the girders, two different schemes were used: (1) draping of prestressing strands, and

(2) a combination of draping and debonding of prestressing strands. Of the four ends of the two girders, one end used strand draping exclusively while the other three ends used a draping/debonding combination of prestressing strand. Both of these schemes are shown in Figure 2.3. Draped prestressing strands were used to control end stresses in the test girders by decreasing the effective prestressing force eccentricity. The net compressive force is still the same, the only difference is the net moment induced by the prestressing force. The number of draped strands and their eccentricity was adjusted to balance the prestressing compressive stress and prestress moment so that allowable stresses were not exceeded in any part of the cross section. Debonding the prestressing strands control the end stresses by reducing the effective prestressing force. Debonding was accomplished by shielding some of the prestressing strand with a plastic sheath so that part of the strand is not allowed to bond with the concrete section. A trial and error process was used to optimize the location and number of strands to be debonded and the shielding length required. Debonding of the strands was used along with draping of the strands on test girder ends IA, IB, and IIC. Draping was used exclusively on test girder end IID.

2.3.3.2.1 Flexural Strength

Because Article 9.17 of the AASHTO Specifications [2] does not directly address the issue of the prediction of the ultimate flexural strength for composite concrete prestressed girders, an alternative method was used. An inadequacy of these AASHTO equations is that they do not recognize that two different concrete strengths may be used in the composite section or the top flange and the composite deck may have different widths if the modular ratio is used to transform the composite concrete section into an equivalent section with concrete of the same strength of that of the precast girder. The method that was used to predict the ultimate flexural strength analyzed the composite girder section as a series of concrete layers with different rectangular dimensions and concrete strengths. The analysis assumed full composite action between the HSC precast girder section and the cast-in-place concrete deck. It ignored the area of the nonprestressed bonded reinforcement in the upper flange of the girder and in the concrete deck. Only the gross area of concrete was considered.

The ultimate flexural capacity of the test girders was calculated using the following method:

1. Determine f_{ps} = calculated average stress, psi, in the prestressing steel at ultimate flexure using equation 9-17, Article 9.17.4 [2] shown below

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\frac{\rho f_{pu}}{f'_c}\right)\right]$$

f_{pu} = tensile strength of prestressing strand, psi
γ_p = factor for type of prestressing steel, 0.28 for low-relaxation steel
β_1 = assumed to be 0.85 for concrete with f'_c of 4,000 psi or 0.65 for concrete with $f'_c > 6{,}000$ psi [5]
d = distance from the centroid of the prestressing steel to the extreme fiber in compression, in.
ρ = ratio of prestressing steel, (A_{ps}/bd)
b = width of compression block, in.
f'_c = compressive strength of member, psi

2. Determine the depth of the equivalent rectangular stress block, a

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b}$$

A_{ps} = area of prestressing reinforcement in tensile zone, in^2.

2.1. Assume depth of stress block in deck: use f'_c and b of then deck; if the calculated depth of the stress block is greater than the deck thickness, then the neutral axis is in the precast section.

2.2. Calculate the actual depth of the stress block by equating the ultimate tensile force at the center of gravity of the prestressing with the compressive force in the concrete deck and precast girder flange.

3. Calculate the predicted ultimate flexural capacity

ϕM_n = sum of moments of ultimate compressive strengths of the stress blocks about the center of gravity of the prestressing steel times a flexural strength reduction factor, ϕ; $\phi = 1.0$ for factory produced precast prestressed concrete members, Article 9.14 [2]

From the procedure above, the predicted ultimate flexural strength for the test girders was 8610 ft.-kips (see Appendix B for the numerical calculations). The actual observed ultimate strength of the test girders will be determined after the girders are subjected to a series of flexural fatigue load cycles.

2.3.3.2.2 Shear Strength

The shear capacity of the test girders was predicted using equations given by Article 9.20 of the AASHTO Specifications [2]. The total shear strength was given by

$$V_u = \phi(V_c + V_s)$$

V_u = total shear strength of member, lbs.
V_c = shear strength provided by concrete, lbs.
V_s = shear strength provided by web reinforcement, lbs.
ϕ = capacity reduction factor for shear, 0.90

The shear strength provided by the concrete in the girders is a function of what type of shear failure or shear cracking controls the shear design. There are two types of shear cracking. The first type is flexure-shear concrete cracking, and the second type is web-shear concrete cracking.

A flexure-shear crack originates as a vertical flexural crack in the member and then inclines towards the point of load in the shear span. Flexural-shear cracking that leads to failure of a girder usually begins in the shear span at a distance equal to the effective depth of the cross section away from application of the load. The AASHTO Specifications [2] imply that the shear force required to produce flexure-shear cracking in prestressed concrete members is made up of two components: the shear force that causes a flexural crack to start, $M_{cr}(V/M)$, and the shear force that causes the flexural crack to incline, $0.6\sqrt{f'_c}b_w d$. Therefore, the total shear, V_{ci}, that causes flexural-shear cracks to form is given by equation 9-27 in Article 9.20.2 [2] as:

$$V_{ci} = 0.6\sqrt{f'_c}\,b_w d + V_d + \frac{V_i M_{cr}}{M_i}$$

V_{ci} = total shear causing flexural-shear cracking, lbs. but need not be taken less than $1.7\sqrt{f'_c}b_w d$
f'_c = compressive strength of member, psi
b_w = width of web, in.
d = effective shear depth, in., but not less than $0.8h$ where h is height of the section
V_d = shear due to dead load at location of interest, lbs.
V_i/M_i = shear to moment ratio at the location of interest
M_{cr} = moment that causes flexural cracking, in.-lb., given by equation 9-28 on the following page

$$M_{cr} = \frac{I}{y_t}\left(6\sqrt{f'_c} + f_{pe} - f_d\right)$$

I = moment of inertia of the uncracked section, in.^{4}
y_t = distance from bottom fiber to centroid of section, in.
f'_c = compressive strength of member, psi
f_{pe} = stress at the bottom fiber of the section due to the effective prestressing force, psi
f_d = stress at the bottom fiber of the section due to unfactored dead loads, psi

The second type of shear failure that occurs in prestressed concrete members is web-shear cracking. Web-shear cracks form when the principal tensile stresses from shear and flexure exceed the tensile strength of the concrete. In prestressed concrete members, the resistance to web-shear cracking is due to two components: the tensile strength of the concrete, $3.5\sqrt{f'_c}b_wd$, and the compressive forces in the section due to the vertical component of the prestressing force and the applied loads. These compressive forces help keep the web-shear crack closed which in turn provides added shear resistance through aggregate interlock. The total shear, V_{cw}, that causes web-shear cracks to form is given by equation 9-29 in Article 9.20.2 [2] as:

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3f_{pc}\right)b_w d + V_p$$

V_{cw} = total shear causing web-shear cracking, lbs.
f'_c = compressive strength of member, psi
b_w = width of web, in.
d = effective shear depth, in., but not less than $0.8h$ where h is the height of the section
f_{pc} = resultant compressive stress at the centroid due to loads acting on the precast section only, psi
V_p = vertical component of the effective prestressing force, lbs.

When the cross section of interest in the shear analysis is within the transfer length of the prestressing strands, the effective prestress force must be reduced. It is assumed that the prestressing force is transferred linearly, from zero at the end, to the concrete section over a transfer length of 50 diameters of the strand. The shear strength provided by the prestressed concrete member, V_c, is then taken as the smaller of the values V_{ci} or V_{cw}. From these equations it was found that V_{cw} controlled the predicted concrete shear capacity for both the draped strand and debonded strand designs from the ends of the girders to about 0.1L of the span from the ends. Inside of this interval, V_{ci} controlled the predicted concrete shear capacity. Reference Appendix B for the predicted concrete shear strength along the spans of both test girders.

Even if the shear strength provided by the concrete is greater than the applied shear load, a minimum amount of shear reinforcement must be provided unless the applied shear is less than $V_c/2$. The shear strength provided by web reinforcement is given by equation 9-30 in Article 9.20.3 [2] as:

$$V_s = \frac{A_v f_{sy} d}{s}$$

V_s = shear strength provided by web reinforcement, lbs., but not taken greater than $8\sqrt{f'_c} b_w d$.
A_v = area of web reinforcement, in^2.
f_{sy} = yield strength of web reinforcement, psi, not to exceed 60,000 psi
d = effective shear depth, in., but not less than 0.8h where h is height of the section
s = spacing of the web reinforcement, in., but shall not exceed 0.75h or 24 in.

The minimum area of web reinforcement, given by equation 9-31 in Article 9.20.3 [2], shall not be less than:

$$A_v = \frac{50 b_w s}{f_{sy}}$$

s and b_w are in in. and f_{sy} is in psi.

Since the predicted shear capacity of the composite concrete section was less than twice the applied factored shear throughout the girder spans, at least minimum vertical shear reinforcement was required. No. 4 epoxy-coated, deformed, Grade 60 ksi double leg steel stirrups were used in both test girders. The transverse reinforcement placed at the maximum stirrup spacing, s = 24 in., satisfied both the minimum area and shear strength required from web reinforcement. As noted in the following discussion, however, the horizontal shear requirements governed the design of the transverse reinforcement.

Because the test girders were designed as composite sections, the horizontal shear stress at the interface of the upper flange of the girder and the concrete deck must be considered. The design for horizontal shear in composite flexural members is given by Article 9.20.4 in the AASHTO Specifications[2]. The Specifications state that when minimum ties are provided, the contact surface is clean, free of laitance, and intentionally roughened to an amplitude of approximately 1/4 in., the horizontal shear strength, V_{nh}, cannot be taken greater than $350b_vd$ in pounds; where b_v is the width of the interface and d is the effective composite depth, both in inches. This type of interface condition was provided by the girder fabricator.

It is also stated that all shear reinforcement of the girder shall extend into the cast-in-place concrete deck slab. The specifications allow the stirrups to be used as horizontal shear tie reinforcement. To provide the horizontal shear reinforcement, the No. 4 double leg vertical shear stirrups extended 5-1/2 in. into the composite concrete deck; the loop at the top of the stirrup provided proper anchorage. The minimum area of tie reinforcement required by the specifications was given by $50b_vs/f_y$ with the tie spacing s not exceeding four times the least web width of the support element, nor 24 in. For the No. 4 double leg shear stirrups, the required minimum tie spacing for horizontal shear was 16 in. This 16 in. stirrup spacing controlled the shear stirrup design and was used along the entire length of the test girders, except at the end region as noted in the following discussion. From these equations the predicted horizontal shear strength, V_{nh}, for the test girders was 524 kips. The layout of the vertical stirrups along with other mild steel reinforcement is shown in Figure 2.9 and the reinforcement details in Figure 2.10.

It was also required by the AASHTO Specifications Article 9.21.3 [2] to provide extra reinforcement in the ends of the girders to prevent cracking and bursting of the section during transfer and to also provide confinement of the concrete around the prestressing steel in the bottom flange. It was stated in that article that vertical stirrups acting at a unit stress of 20,000 psi to resist at least 4 percent of the total prestressing force shall be placed within the distance of $d/4$ from the end of the girder. To provide confinement, AASHTO requires nominal reinforcement to enclose the prestressing steel in the bottom flange for at least a distance d from the end of the girder. A detail of this extra reinforcement (three spaces at 3 in. on centers of the typical stirrups and two No. 6 stirrups at 2 in. from the end) in the end of the girders is shown in Figure 2.11.

Chapter Three - Materials

3.1 High Strength Concrete

3.1.1 General

The high strength concrete (HSC) mix used for the girder design required a 28 day compressive strength of 10,500 psi to satisfy the allowable stress limits. Two different mix compositions were used for test Girders I and II, respectively. Girder I used a concrete mix incorporating crushed limestone aggregate while Girder II utilized round glacial gravel aggregate in the mix with the addition of microsilica. The microsilica additive was used to enhance the aggregate - cement paste bond in the glacial gravel concrete mix. These mix designs were derived from laboratory tests conducted in a companion project regarding the mechanical properties of HSC [4]. In that project, tests had shown the use of microsilica in the limestone aggregate mix did not significantly improve the strength of the concrete and no additives were needed to enhance concrete strength using limestone aggregate, but concrete strength was enhanced by microsilica in the glacial gravel mixes.

3.1.2 Design Mixes

The HSC mix design used in the test girders is shown in Table 3.1. The limestone aggregate concrete mix, designated as Mix #1, consisted of Type 3 cement, sand (fine aggregate), 5/8 in. limestone (coarse aggregate), water reducing agent (WRDA -19), and water. The glacial gravel concrete mix designated as Mix #2, had the same contents as Mix #1 except that 3/4 in. round glacial gravel was used for aggregate and liquid microsilica fume was incorporated.

Each gallon of liquid microsilica consisted of 5.4 lbs. microsilica, 5.7 lbs. water and 0.3 lbs. WRDA -19. The absorption capacity of the aggregates as a percentage of their weights were measured as: 2.9% for the limestone, 2.6% for the glacial gravel, and 3.7% for the sand.

After the girders were cast, a sieve analysis of the aggregates used in the concrete mixes was performed. The results of this analysis are shown in Table 3.2.

3.1.3 Concrete Strengths from Samples

During casting of the test girders, samples from each of the concrete batches were taken for strength tests. Each girder used approximately 21 cubic yards of concrete; 7 batches were mixed for the casting (3 cubic yards per batch) of each girder. Since each batch mix of concrete was adjusted slightly to achieve optimum slump and workability (the material quantities for each concrete batch are documented in Appendix C), it was necessary to document each batch location in the test girders (Figure 3.1).

Test cylinders were made with the concrete by both the fabricator and the University of Minnesota research group. The fabricator used 6 x 12 in. and 4 x 8 in. steel molds to cast test cylinders used to determine the compressive strength of the concrete at the time of release (18 hours after casting), at 7 days, and at 28 days. The 4 x 8 in. cylinders were cured using the SURE CURE Cylinder Mould System [7]. The SURE CURE Cylinder Mould System, manufactured by Products Engineering, is a reusable one piece metal cylinder mold that has a built-in heating system that is controlled via a thermocouple temperature sensor. The thermocouple temperature sensors, used by the curing system, (mounted on a steel girder form at midspan) monitor the concrete curing temperature which is used by a temperature/matching controller to adjust the heater in the cylinder mold so the cylinder concrete matches the temperature of the curing girder. The 6 x 12 in. cylinders were cured in laboratory conditions at the fabrication plant.

Additional cylinders were cast by the University of Minnesota research group using 6 x 12 in. plastic molds. Because of the large number of test cylinders and beams needed to determine concrete strength throughout the period of the girder tests, a vibrating table was used to help consolidate the concrete specimens during casting. The concrete specimens were then cured, up to the time of prestress release, next to the girders under the curing tarps before being transported to the University of Minnesota testing laboratory. No additional heat or moisture was applied to the girders or concrete specimens. During the casting, the ambient outside temperature was 89 degrees F (humid) while the temperature of the fresh concrete was 82 degrees F. At release, the ambient outside temperature was 85 degrees F (humid) while the internal temperature of the girders was about 135 - 140 degrees F.

The compressive strength tests were conducted in accordance with ASTM C 39 [8]. Modulus of elasticity tests were conducted in accordance with ASTM C 469 [8]. Modulus of rupture tests were conducted in accordance with ASTM C 78 [8]. Splitting tensile strength tests were conducted in accordance with ASTM C 496 [8]. In addition to these tests, creep, shrinkage, and freeze-thaw tests were conducted in the companion materials study [4] mentioned earlier. An inventory of the concrete samples taken for the test girders and the concrete deck along with the schedule of their testing is shown in Table 3.3. The nominal 4,000 psi concrete used in the composite decks is described in Section 7.3 of this report. The results of these tests to date are summarized in Table 3.4.

3.2 Prestressing Strand

The prestressing strand used in both test girders was 0.6 in. nominal diameter, 7-wire, Grade 270 ksi low-relaxation strand conforming to ASTM Designation A 416 [8]. The strand was supplied using two separate coils for girder fabrication. A total of five 6-ft. strand samples were taken from each coil. Samples were taken from both the beginning and end of each coil. The surface of the strand was clean with very little noticeable corrosion.

A load-strain curve for the production lot of strand was provided by the manufacturer. This load-strain curve is reproduced in Figure 3.2. Additional tensile tests were performed on the 10 strand samples at the Mn/DOT Material Testing Laboratory in Maplewood, Minnesota. Four of the tensile tests, two from each coil, were conducted in accordance with ASTM Designation A 370 [8]. The strains for these tests were determined using an extensometer attached to the strand sample, and the load was given by a calibrated load cell in a Riehle tensile testing machine.

The objective of the remaining six strand tensile tests was to determine the relation between the stress in the strand and the strain measured on the surface of the individual wires around the perimeter of the strand. The strain in the wires was determined by the use of electrical resistance type strain gages bonded to the steel surface. The orientation of each gage was parallel to the longitudinal axis of the wire it was bonded to. Of the six tests, two different strain gage configurations were used and are illustrated in Figure 3.3. The first configuration employed a strain gage on each of the six outside wires at the midlength of the strand sample (these three test samples are designated with a M and gages 1 through 6 around the strand perimeter). The second configuration used a strain gage on two of the six strand wires, each of these two wires were opposite each other. This was done to the same wires at three separate locations along the length of the strand sample. The first location was at the midlength of the strand sample and the other two locations were at one lay of the wire (about 8-3/4 in.) above and below the midlength location (these three test samples are designated with a L and gages at the midlength designated B, the gages below C and the gages above A on the wires 1 and 2 of the two wires gaged on the perimeter of the strand sample). An extensometer, that was monitored by Mn/DOT testing personnel, was also attached to the midlength of each strand sample. The samples were loaded and unloaded at 10 kip intervals up to 50 kips and then taken to ultimate. The strain in the individual wires of the strand were recorded after each load interval.

A typical stress-strain plot for the strands tested for each of the strain gage schemes is shown in Figure 3.4 (type L gage scheme) and Figure 3.5 (type M gage scheme). It can be seen from these figures that there is very little variation of strain in each of the individual perimeter wires of the prestressing strand. Also, there appears to be very little effect on the individual wire strain due to the cycling of the tensile load. In general, the strain measured from having only one wire of the prestressing strand instrumented with a strain gage is a good approximation for the total average strain in the strand, and that any minor error in mounting the strain gage so its axis is parallel to the axis of the individual strand wire is negligible.

The results of the strand tensile tests, strand yield and ultimate tensile strengths along with the modulus of elasticity, are shown in Table 3.5.

3.3 Mild Steel Reinforcement

Mild steel reinforcement (MSR) was used in several areas of the test girders. MSR was used in the girder webs as vertical shear stirrups, in the upper flanges for two reasons: to act as a tension tie during prestress transfer and as compression reinforcement at ultimate flexure, and in the composite concrete deck as flexural reinforcement. All MSR, including the reinforcement used in the composite concrete deck was epoxy coated.

Two different vertical shear stirrup configurations were used in the test girders. No. 4 epoxy coated, deformed, Grade 60 reinforcement was used for both types. The first type was a typical Mn/DOT double leg, straight stirrup(designated as a U - stirrup). The second type of stirrup was a modified Mn/DOT standard U - stirrup which had a 90 degree standard hook, 9 in. long at the end of each leg (designated "modified U - stirrup"). The axis of the stirrup hooks was parallel to the longitudinal axis of the girders; the orientation of this type of hook permitted easy installation of the stirrup cages since the stirrups could still be lowered vertically between the prestressing strands. The intent of the 90 degree hook was to provide additional anchorage to the stirrup and to allow for development of its capacity across inclined shear cracks.

The double legs of the stirrups were joined together at the top by a loop that extended 5-1/2 in. out of the top flange of the girders. The purpose of this loop was to provide a horizontal shear tie to ensure composite action between the girder and the deck (stirrup details are shown in Figure 2.10).

Four No. 6 Grade 60 epoxy coated reinforcing bars were used in the upper flange of the test girders. The bars were placed longitudinally with 1-1/2 in. clear concrete cover. The spacings of these bars are shown in the cross-sectional detail.

All deck reinforcement used in the test girders was No. 4 Grade 60 epoxy coated rebar. Because each test girder had its 4 ft. wide concrete deck cast separately, no transverse reinforcement was needed since the deck did not span between girders; only shrinkage and temperature steel was used in this direction. Longitudinal slab reinforcement was provided as directed by the Mn/DOT Bridge Design Manual [3] .

Three samples of each type of stirrup configuration and deck reinforcement were tested for both the yield and ultimate tensile strengths. The stress-strain characteristics were determined using a load cell in the Riehle testing machine and a two in. gage length extensometer attached to the rebar samples. Resistance type strain gages were attached to the three deck reinforcement samples. At the location of the extensometer and strain gages on the specimens, the epoxy coating was removed to prevent slippage and to determine the rebar cross-sectional area. Table 3.6 gives the results of the tensile tests and Figure 3.6 shows a typical stress-strain curve for the MSR tested.

Chapter Four - Instrumentation and Data Acquisition

During the fabrication of the test girders, several different types of instrumentation were installed. The instrumentation was placed to measure prestressing strand transfer length, prestress losses, concrete creep and shrinkage, concrete temperature effects, mild steel reinforcing strains, and strains in the concrete and steel reinforcement due to flexure and shear (Figure 4.1). The instrumentation, where applicable, was monitored throughout the fabrication of the girders and continued to be monitored throughout the duration of the testing program.

Instrumentation installed to measure material strains was of three basic types. The first was bondable, electrical-resistance type strain gages. These gages were of two configurations: one (foil type gage) was glued to steel reinforcement while the other (foil gage encased in a grit coated plastic matrix) was placed in wet concrete and bonded to the paste matrix during curing. Strain was then correlated to the change in resistance of the wires in the gage due to their change in length. The second category of strain measuring devices was a detachable mechanical (DEMEC) strain measuring system consisting of points fixed to the girder surface and measured with a hand-held dial indicator. The dial indicator or Whittemore gage was used along with the DEMEC points, cast into the concrete surface, to measure concrete surface strains. The strain was determined by measuring the change in the initially known distance between DEMEC points. The third type of strain measuring device was a vibrating wire gage. The strain was determined by measuring the change in frequency of a taut wire in the gage vibrating in a static magnetic field. As the distance between the ends of the gage change, the tension in the wire also changes. This change in wire tension affects the natural frequency of vibration which is converted into an electrical impulse since the wire is vibrating in a magnetic field. Also built into the vibrating wire gage is a temperature transducer that gives the actual temperature in the material at the gage location.

The instrumentation was monitored either by use of switch and balance boxes with strain indicator boxes or a MEGADAC data acquisition system for the electronic strain devices. The DEMEC points were measured with a Whittemore gage and recorded by hand.

A list of all electrical type gages installed in the precast girder sections can be found in Appendix D. This appendix includes a description of the gage type used along with the nominal and actual location of these gages.

4.1 Prestressing Strand Strain Gages

After all of the strands were placed on the prestressing bed but before they were stressed, electrical resistance foil type strain gages were bonded to several strands at several different cross section locations of both test girders. Strand strain gages, TML Tokyo Sokki Kenkyujo Co., LTD type FLK-1, were bonded to a single wire of the strand. The gage was oriented such that the longitudinal axis of the strain gage was parallel to the longitudinal axis of the single wire; this meant that the gage was not parallel to axis of the strand. After the gage was applied to the strand, it was tested for conductance and resistance. Then it was coated with a waterproofing compound and covered by a piece of butyl rubber to protect it from impact.

The strand gages were installed to measure the strand strains due to tensioning, transfer length, prestress losses, flexure, and loss of strand anchorage in high shear zones. The locations and number of strain gages applied to strands was chosen to fulfill these needs and was limited by the number of gage reading channels that were available during a particular part of the test.

4.1.1 Transfer Length

It was assumed that the prestressing force in the strand would be transferred to the girder cross section linearly over a distance of 50 times the diameter of the strand from the end of the girder. For the 0.6in. diameter strand used, the predicted transfer length was 30 in. To determine the actual transfer length for the strands in the test girders, gages were applied to the same strand at 15, 22, and 30 in. from the beginning of the transfer length. On ends IA, IB, and IIC of the test girders a combination of draping and debonding was used to control end stresses, while IID used only draping, see Figure 2.3.

The location of strain gages installed on the prestressing strands is shown in the bottom half of Figure 4.2. The gage locations that are numbered in Figure 4.2 must be referenced to Table 4.2; the strand numbering scheme is explained in the bottom half of Figure 2.2. On girder end IID, four different strands were instrumented. Since no strands were debonded on this end, the transfer length for all strands began at the face of the girder end. On girder ends IA, IB, and

IIC four pairs of strands were debonded to various lengths as described in Section 2.2.1. At the end of each length of debonding, the transfer length began. These debonded strands were also instrumented with three strain gages on each strand at the same distances from the end of the debonding: 15, 22, and 30 in.

4.1.2 Flexure and Fatigue

To determine the distribution of stresses in the girder cross section due to flexure, a combination of all gage types was used. Strand strain gages provide information about the distribution of strains in the cross section due to flexure. Both test girders have strand strain gages for flexural strain measurement at the same locations. There were six different strands, strands 12, 15, 17, 19, 25, and 35 (see Figure 2.2 for strand nomenclature), instrumented at each of three different cross section locations in each girder. These locations were 0.45*L* from each end and at midspan and are shown in Figure 4.2.

From the gages installed on the strands, the change in strand strain due to the applied stress ranges could be monitored throughout the flexural fatigue and ultimate flexure testing procedures. The loss of the longitudinal tension tie anchorage could be monitored by the use of the transfer length strand strain gages during the ultimate shear strength tests.

4.2 DEMEC Points/Whittemore Gage

4.2.1 DEMECs in Precast Test Girders

The measurement of concrete surface strain was performed mechanically by the use of DEMEC points and the Whittemore Gage. DEMEC points were installed in both the precast girder and the composite concrete deck. The DEMEC system consists of a set of threaded brass inserts cast into the concrete with a nominal center-to-center spacing of six in. Prior to casting the test girders, the brass DEMEC inserts were attached to an aluminum strip with screws. The screws were counter-sunk so the side of the metal strip opposite the DEMEC inserts could be secured flat against the inside of the girder forms. The metal strips with the brass inserts were located on the formwork so the line of DEMEC points was parallel to the axis of the centroidal height of the prestressing strands as shown in Figure 4.3.

On girder ends IA, IB, and IIC, with draped and debonded strands, the length of the DEMEC point series was 24 ft. on the bottom flange and 4 ft. along the draped strands centroid.

On girder end IID, with draped strands, a 4 ft. long DEMEC point series was used along the centroid of both the straight and draped strands. DEMEC point layout is illustrated in Figure 4.3.

After the concrete was cast in the girder and before the forms were stripped, the screws holding the metal strips to the forms were removed. Formwork was removed from the girders and the remaining metal strip was then pried away from the concrete. Stainless steel DEMEC tips were then screwed into each embedded brass DEMEC insert. The stainless steel tips were precision machined to accept the measuring points of the Whittemore gage.

Before the prestressing strands were cut, an initial reading between each of the 6 in. DEMEC spaces was taken using the Whittemore gage. The Whittemore gage had a dial vernier gage with an accuracy to 0.0001 in. After transfer (release of the prestressing force), another set of DEMEC readings was taken along each line of points. The average strain due to transfer between any set of two DEMEC points was found by dividing the difference between the after transfer and initial DEMEC readings by the initial gage length between inserts. Throughout the duration of the project, DEMEC readings were taken periodically to monitor prestress losses.

4.2.2 Slab DEMECs

The DEMEC points on the slab were installed to measure concrete surface strains. The locations of the DEMEC points cast onto the top of the composite concrete deck above each test girder are shown in Figure 4.4.

4.3 Vibrating Wire Gages

The vibrating wire gages used in the test girders were model VCE-4200 manufactured by Geokon Inc. These gages were designed for direct embedment into concrete. The advantage of using vibrating wire strain gages (VWG) over electrical resistance type strain gages is that VWG have excellent long term stability and do not experience signal degradation over long cable length. The VCE-4200 VWG were 6 in. long and had a maximum strain range of 3000 microstrain with a sensitivity of 1.0 microstrain. Each VWG had a built in thermistor which enabled temperature readings at the gage location to be measured in the range of -20 to 80 degrees C.

The locations of the VWG in the girders were determined by the type of strain response that was to be measured. Each precast girder had eleven VWG installed at various locations.

Each composite concrete deck had five VWG cast into the slab as shown in Figure 4.5. The locations of the VWG in the test girders are shown in Figure 4.2. Nominal and actual slab gage locations are tabulated in Appendix E.

The VWG installed in the girder cross section at the support were used to monitor the strains in the concrete due to shrinkage, concrete creep, and temperature effects. This location was ideal for these measurements because there was little longitudinal strain resulting from flexure. The concrete strains resulting from changes in temperature were readily determined because the VWG was used to measure the temperature at the gage at the same time the strain was recorded.

The concrete shrinkage is primarily a function of the relative humidity of the air surrounding the concrete specimens. The relative humidity of the laboratory was measured using a sling psychrometer (wet bulb and dry bulb thermometers used together to determine relative humidity).

The VWG installed at 0.45*L* from each end and at midspan of each test girder were used to monitor the change in concrete strains due to flexural bending. Like the VWG at the support, these gages were also used to measure temperature and shrinkage strains in the concrete. These gages, along with other gages installed at these cross sections, were used to determine the two-dimensional distribution of flexural strains in the cross section. From this information the location of the neutral axis could be determined.

4.4 Concrete Strain Gages

In addition to the vibrating wire gages, resistance type strain gages were also cast into the precast girders and composite concrete decks. Two types of these gages were cast into the concrete: a single axis strain gage and a three axis strain rosette. Both types of gages were from the Polyester Mold Gauge-Series "PM" manufactured by TML Tokyo Sokki Kenkyujo Co., LTD. The single axis concrete strain gage, designated PML-60, had a gage length of 60 mm. The three axis concrete strain rosette, designated PMR-60 also had a gage length of 60 mm for each axis separated by 45 degrees. Both gages were made of an electrical resistance foil gage sandwiched between two resin plates which came sealed and then coated with a coarse grit to ensure a good bond with the concrete; these gages can measure strains up to 2%.

The PML-60 gages were installed in the precast girder sections as shown in Figure 4.2 and in the composite concrete deck sections as shown in Figure 4.5. These gages were monitored throughout the period of girder testing. The PML-60 gages installed at the girder support cross section were read at the time of transfer of the prestressing force to determine strains in the concrete in the horizontal direction at the plane of the intersection of the web and the bottom flange. The rest of the PML-60 gages in the girders were used to monitor the longitudinal strains in the concrete due to flexural bending. An array of these gages, along with other types of gages in the concrete and bonded to steel, are used to determine the two-dimensional strain distribution of the cross section at midspan due to fatigue and ultimate flexure. Some of the installed PML-60 gages will be monitored during the shear tests.

The PMR-60 concrete strain rosettes were installed in the precast girders as shown in Figure 4.6. These gages monitor strains during the shear tests. Each of the four girder ends had five PMR-60 strain rosettes installed in the plane of the stirrup legs at mid-height of the precast section on one face of the girders. These gages were installed so that the three legs of the rosettes were oriented at 0, 45, and 90 degrees in the vertical plane parallel to the web of the test girders. Since the orientation of the three gage legs are fixed, the principal stresses and their directions can be determined with a Mohr's circle analysis.

4.5 Mild Steel Reinforcement Strain Gages

To determine the strains along the axis of the mild steel reinforcing bars, electrical resistance foil type strain gages were bonded to the surface of the steel. The gages used for this application were type WFLA-3 manufactured by TML Tokyo Sokki Kenkyujo Co., LTD. The WFLA-3 had a gage length of 3mm and could monitor strains up to 3% elongation. These gages were installed in three different locations: on the vertical reinforcement at the girder supports, on various shear stirrups, and on some of the longitudinal reinforcing bars of the composite concrete deck.

The locations of the WFLA-3 gages installed on the vertical bars at the girder supports are shown in Figure 4.6. These gages were intended to monitor the strains in the steel due to the stress in the cross section from the transfer of the prestressing force because horizontal web cracking had been observed by girder fabricators in the past during prestress transfer for girders with a large amount of prestressing steel.

The locations of the steel strain gages installed on the shear stirrups are shown in Figure 4.6. Of the stirrups instrumented, only a single leg of the stirrup had 1, 2, or 3 strain gages bonded to the surface. These gages were used to determine the strains in the stirrups when high shear stresses were applied to the girder cross section. The distribution of the stress in the stirrup leg could be monitored to determine whether the bar strength was developed or if it was losing anchorage if the shear crack intersected the stirrup towards the end of the leg. This information was used to compare stirrups with and without anchorage hooks at the bottom of the legs.

The locations of the WFLA-3 strain gages installed on the slab reinforcement is shown in Figure 4.5. These strain gages were monitored primarily during the ultimate shear tests.

4.6 Data Acquisition System

Aside from the portable P3500 Strain Gage Indicators used to monitor resistance type strain gage channels, a central data acquisition system was used to record instrumentation data throughout the fabrication of the test girders and during future testing phases. The data acquisition system used was an OPTIM MEGADAC 3008AC, with a MEGADAC 0016AC expansion chassis, manufactured by OPTIM Electronics. This system, which operated on AC power, had the following specifications/capabilities:

1) IEEE interface board which communicated with a 486 personal computer via TCS software V. 5.0.0.
2) Two - AC3884-VWIK boards for a total of 16 channels of vibrating wire sensors (8 strain + 8 thermocouple channels)
3) Ten - AD885D boards to monitor a total of 80 channels of low-level (strain gage) signals
4) Seven - SCI-88C/120 signal conditioning boards for a total of 56 channels of 1/4 bridge completion
5) 512 kilobytes of internal memory (data buffer) and the capability to sample data at a rate of 0 - 25,000 samples per second. Instrumentation data was stored on magnetic media (floppy-disks).

Chapter Five - Fabrication of the Test Girders

The process and sequence used for the precast prestressed test girder fabrication is described in this chapter. The two test girders were fabricated at the Elk River Concrete Products prestressing yard in Elk River, Minnesota. It was the goal of the project to fabricate the girders using construction techniques common with current industry. The actual construction of the test girders was done by both Elk River Concrete Products employees and University of Minnesota researchers from August 5 to August 11, 1993. Typically, it takes the fabricator a single day to completely construct a bridge girder: the prestressing strands and mild steel reinforcement are placed into the forms in the morning, the concrete is cast in the afternoon. The next morning the forms are stripped and the strands are released after the concrete is allowed to cure enough to facilitate the release of the prestressing force, usually 18 hours cure time.

The fabrication of the precast test girders can be described by five basic construction steps:

1. Installation and prestressing of the 7-wire strands.
2. Placement of the mild steel stirrups and longitudinal reinforcement.
3. Placing of the girder formwork.
4. Casting and curing of the concrete.
5. Release of the prestressing force.

The two test girders were fabricated simultaneously on an exterior prestressing bed which was 316 ft. long and had a 2 million pound prestressing force capacity (see Figure 5.1 for the prestressing bed layout). Throughout construction of the test girders, instrumentation for the testing program was installed and/or monitored at the various stages. All construction phases were controlled, supervised, and documented by University of Minnesota personnel; a field log of the test girder construction was kept and can be found in Appendix F of this report.

5.1 Installation of the Prestressing Strands

Prior to the placement of the strands in the prestressing bed, the hold-down and lift hardware for the draped strands was located and installed. The hold-down points were located at 0.4*L* or 53.1 ft. from each girder end. The hold-down hardware was of a standard type and supplied by the fabricator. Since both girders were cast simultaneously on the same bed, a lift point was used for the draped strands between girder ends. The girder ends next to each other were separated by about 5 ft. on the bed.

The 0.6 in. diameter strand was placed onto the prestressing bed two at a time from two reels located at the dead-end of the bed. A template was located at each end of the bed through which the strands were threaded and chucked against (Figure 5.2). The live-end of the bed was the end where the strands were jacked to their prestressing force. A total of 46 strands were installed the total length of the prestressing bed and chucked at each end. The draped strands were threaded through the hold-down and lift points.

5.1.1 Instrumentation of Strands

The original intention was to begin installing the strain gages on the strands as soon as all the strands were pulled through the bed, but it was impossible to determine which strand was which because they were all intertwined along the length of the bed. The prestressing bed was only 26 in. wide at the bottom flange, therefore the 46 - 0.6 in. diameter strands made a large heap along the bed length (Figure 5.3). The strands that were to be instrumented were at the bottom of the cross section and hence at the bottom of the strand pile. In order install a strain gage to a given strand it must be pulled to the top of the pile, not an easy task, and then the strand was to be relocated to the bottom of the pile after the gage was applied. It was determined that too many gages would be damaged with this procedure, and if the strand was left at the top of the pile, the gage would probably be damaged during the prestressing process because the strands were stressed from the top down.

In order to ease location of a given strand and to give clearance between them, it was decided to preload all strands to 4,000 lbs. of tension each. Before the preload process began, four different strands were instrumented with strain gages to measure the preload strain. The four gages were read with a P3500 Strain Indicator box at the gage locations. These gages were: F-IA-19, F-IA-C15, F-IID-C12, F-IID-C15.

Two of the four gages were destroyed by friction between strands during the preload process. Thus it was a good decision to apply preload to the strands before all of the gages were installed. Even after the strands were preloaded it was still difficult to get at many of the gage locations away from the ends of the bed due to sag in the strands. To help this problem 2 in. x 2 in. blocks of wood located about every 25 ft. along the length of the bed were used to separate the horizontal layers of strands.

A total of 130 strain gages (Figure 5.4) were installed in both test girders at approximately 50 cross section locations on 11 different strands. To ensure the gages would be positioned correctly in their final locations after prestressing, elongation, sag, and slack of the strand on the prestressing bed had to be considered. Strand elongation was easily predicted since the elastic modulus of the strand is known accurately. The strand sag and slack varies from bed to bed and is a function of the number of girders cast on the bed, the strand draping geometry, the bed length and strand tension.

To help predict the change of relative strand location before and after stressing, measurements of strand movement were taken on girders cast earlier on the same bed. Several strands were marked every 25 ft. along the length of the bed before prestressing. After the strands were stressed, the change in location of the marks on the strands were measured. This was repeated several times and an average elongation along the bed length was found. A linear regression analysis was performed on the averaged data to determine an equation to predict strand elongation on that particular prestressing bed. Using this equation, unstressed gage locations were determined and marked on the strands.

5.1.2 Prestressing of the Strands

Before the strands were prestressed, the bearing sole plates were placed on the bed and the strain gages that were to be read during the stressing operation were hooked up to the data acquisition system and the P3500 Strain Indicator boxes. Initial gage readings were taken on all applicable channels. In order to accommodate the large prestressing force needed to pull the strands to 0.75 f_{pu} or 43.5 kips per strand (as given by the AASHTO code [2]) a larger hydraulic pump was borrowed from another local prestress fabricator.

The process by which the strands were prestressed was given by the girder fabricator as follows (prestressing operation shown in Figure 5.5):

1. Each strand was pretensioned to 5,000 lbs. (including the 4,000 lbs. preload).

2. A total full load was then applied, given by:

Draped strands: 43,500 lbs. - 22.375 in. elongation

(additional tension was applied, after the full pull, to the draped strands by raising the lift point of the draped strands)

Straight strands: 44,500 lbs. - 23.0 in. elongation

The slight over tensioning was done to account for seating losses, to attain the required 43,500 lbs. tension after seating. The strands were pulled to their full prestress in the order shown on Figure 5.6.

Gage readings were taken prior to any strand tensioning, and after the following strands were fully stressed (listed in order of tensioning):

1, 10, 14 (first gaged strand to be stressed), 15, 16, 17, 19, 20, 21, 22, 23, 29, 39 (last gaged strand to be stressed), 46 (last strand stressed)

After all of the strands were stressed, the lift point was tightened (raised) to put the draped strands into their final position and apply the final prestressing load to these strands.

5.2 Placing the Mild Steel Reinforcement

There were three basic types of mild steel reinforcement installed into the precast test girders:

1. Vertical shear stirrups
2. Longitudinal flexural rebar in the top flange
3. Concrete confinement rebar at the end regions and around the prestressing strands in the bottom flange

Of these bar types, only the stirrups were instrumented for strain measurements. Prior to the placement of the vertical stirrups onto the prestressing bed, all of the steel strain gages on the stirrups were installed at the University of Minnesota structural laboratory.

The vertical shear stirrups, with G301 bars (reference Section 2.3.3.2.2, Figure 2.9), were tied together with four No. 8 longitudinal top flange bars to form rebar cages. Each stirrup cage was about 30 ft. long. After the prestressing operation, the stirrup cages were carefully lowered into place on the bed with a fork truck so that no instrumentation was disturbed. The cages with the modified "U" stirrups required a little more attention during installation since these stirrups had leg extensions, that were parallel to the length of the girder, which had to be placed between the stressed strands. Each girder required four stirrup cages; additional stirrups between the cages were tied in by hand. A minimum 4 ft. - 11 in. lap splice was used for the No. 8 longitudinal bars where cages met. The additional stirrups and confinement bars were then tied in at the support regions (Figure 5.7). Minimum bar cover requirements were strictly followed.

5.3 Placing the Formwork

Before the steel side forms were placed onto the prestressing bed, the rest of the instrumentation was installed along with the lift hooks, which were tied to the strands. This instrumentation included concrete strain gages, concrete rosettes, and vibrating wire gages. A description of these gages and their locations is described in the previous chapter. In addition to the gages interior to the test girders, the DEMEC strips (Section 4.2) were cast into the surface of the girders by attaching the strips to the inside surface of the steel forms.

After all of the gages were installed and the DEMEC strips were attached to the forms, the forms were sprayed with a release agent (the DEMEC inserts were wiped clean to remove any release agent) and then lifted into place by the overhead cranes (Figure 5.8). The end forms were the first part to be installed; they were held in place by means of a clamp on the outside of the girder which was secured to the draped strands. The steel side forms were attached to the prestressing bed by a through-bolt at the bottom and a pin-ended bar across the top of the forms. All connections between pieces of formwork were sealed with a double-sided adhesive foam weather-strip to prevent concrete paste leakage. During the placement of the formwork by the fabricator, instrumentation cables were routed and connected to the data acquisition systems by the University research team.

5.4 Casting and Curing of the Concrete

Several days before the construction of the test girders, the fabricator cast two short prestressed girder sections (one each of the two different concrete mixes). These test sections were used to evaluate the ability of the mixes to provide a guaranteed concrete strength while maintaining acceptable workability. From these test sections, the fabricator was able to "fine tune" the test mixes to achieve both of the previously mentioned goals.

With the forms in place, all of the instrumentation installed and pertinent gages routed to the data acquisition systems, the test girders were ready to be cast. Prior to casting, a concrete sampling station was set up next to the prestressing bed so that fresh samples of the mixed concrete could be taken.

The concrete was mixed at a batch plant within 200 yards of the prestressing bed. Each test girder required approximately 21 cubic yards of concrete. With a 4 cubic yard capacity for each mixing drum at the batch plant, it took seven 3 cubic yard batches to fill each girder. Before the batch was accepted for use in the test girders, a slump test was performed by the fabricator's Quality Control Team. The target slump was between 5 to 7 in. with the superplasticizer added. The concrete was transported to the girder by a special funnel dump truck. The concrete was then dumped into a bucket which was lifted over the forms with an overhead crane and then dumped into the forms at a standard rate with consideration being taken so as not to damage instrumentation (Figure 5.9).

The concrete was consolidated with both hand held and form mounted mechanical vibrators. The hand held vibrators were of types typically used in concrete construction. The form mounted vibrator was a large unit attached to the forms so it could be pulled along the bed with a "bobcat" type loader at a rate that was equal to the rate of the pour (Figure 5.10). Areas along the length of the girders where instrumentation was susceptible to damage from vibration were marked with construction tape. This tape indicated to the vibrating crew that no vibration was to be done between the marked areas. Care was taken not to overvibrate the concrete.

After the forms were filled with concrete and consolidated, the concrete was screeded (struck) off and the surface was intentionally roughened by raking perpendicular to the axis of the girder (Figure 5.11). The surface was roughened to ensure a good bond between the precast section and the composite concrete deck.

Throughout casting of the test girders, concrete samples were taken by both the project personnel and the fabricator. The fabricator used the concrete cylinders to determine when the concrete had reached sufficient compressive strength for prestress release. After casting the test cylinders and beams, they were placed next to the forms for curing. Construction blankets were placed over the girders and cylinder/beam samples as soon as the casting was completed (Figure 5.12). No other curing aid was used for the test girders or specimens (except for the cylinders cast by the fabricator to be used for the SURE CURE Cylinder Mould System [7], reference Section 3.1.3).

5.5 Release of the Prestressing Strands

Fourteen hours after finishing the casting of Girder II, test cylinders from each girder, cast by the fabricator and cured using the SURE CURE Cylinder Mould System [7], were tested to determine whether the girders had reached the design release compressive strength. A minimum concrete compressive strength at release was specified as 8925 psi. At that time the cylinders had compressive strengths of:

Girder I (limestone mix) = 8634 psi @ 14 hours the end of casting of Girder II

Girder II (glacial gravel w/microsilica) = 10424 psi

Since Girder II met the required concrete strength, the tarps were removed from that girder only to prepare the DEMEC inserts for measuring surface strains. It was thought that Girder I would reach the required release strength shortly thereafter.

About two hours after the first cylinder breaks, instrumentation for determining release stresses had been connected to the data acquisition system and the forms were removed from Girder II. A half hour later another "SURE CURE" Girder I test cylinder was tested by the fabricator; its strength:

Girder I = 8873 psi @ 16.5 hours after casting

Because 45M girders of this length and amount of prestressing had never been built before, the stability characteristics of the test girders during handling were unknown. For this reason tiltmeters were attached to Girder II. Six tiltmeters were placed in 3 locations (at quarter points along the span of the girders and centered longitudinally on top of the section) ; 1 parallel to the longitudinal axis and 1 parallel to the transverse direction at each location .

At 3.5 hours after the first test cylinders were broken, additional samples were tested for strength by the fabricator; these strengths were:

Girder I =	8515 psi, "SURE CURE" @ 17.5 hours after casting
	8440 psi, 135 degree F heat cure
	8630 psi, 135 degree F heat cure
Girder II =	8950 psi, 135 degree F heat cure

9310 psi, 135 degree F heat cure

Concrete test cylinders cast by U of M personnel, and cured next to the girders under the tarps, were brought back to the laboratory to be tested. These cylinders tested at:

Girder I = 7750 psi @ 18 hours after casting

7697 psi

Girder II = 8140 psi

Although Girder I had not fully achieved the required release strength, it was decided to remove the tarps from Girder I so the instrumentation to be monitored at release could be hooked up to the data acquisition systems. At about two hours after the forms were stripped off Girder II, noticeable shrinkage cracking began to appear through the top flange and into the web of Girder II along its span (Figure 5.13). It was possible to see cracks form in the top flange, extend into the web and then migrate into the bottom flange of Girder II. These cracks were most likely due to restrained shrinkage of the microsilica concrete mix from the time the forms were removed until the release of the prestress.

About 1.5 hours after the cracking of Girder II was first noticed the DEMEC strip screws were removed from Girder I as had been done for Girder II, and the formwork removal began. Shortly after, other test cylinders were broken by the fabricator; their strengths were:

Girder I = 9291 psi @ 21 hours after casting

Girder II = 9749, 10,067, 10705 psi all "SURE CURE" @ 21 hours after casting

This was sufficient compressive strength for the release of the prestress.

At the same time, cylinders were also broken at the University of Minnesota; the strengths from these test were

Girder I = 8140, 8692, 8327, 8537 psi @ 21 hours after casting

Girder II = 8142, 8858, 8692, 8864, 9135

Before the release of the prestressing strands, the DEMEC strips on both of the test girders had to be removed from the face of the concrete, leaving the brass inserts embedded into the concrete. The strips were difficult to remove from the surface of the girders, probably due to the embedded aluminum strip expanding from the high temperatures of the curing girders, so they had to be pried off. After the strips were removed, the stainless steel points were screwed into the brass inserts in the concrete. An initial reading between points was taken with the Whittemore gage.

The prestress force was introduced into the girders by cutting the individual strands with an oxy-acetelene cutting torch. Each strand was released by cutting it simultaneously at three different locations: at both ends of the girders and between the two test girders. Figure 5.14 illustrates the strands in the process of being released at the live end; note the draped strand lifting hardware also shown in the photo. The sequence by which the strands were released is shown in Figure 5.15.

The harp point hardware was disconnected from the bottom of the prestressing bed after the 12th strand was released. After release of the prestress was completed, all instrumentation that was monitored earlier was reread. All DEMEC points were re-measured and compared to the initial measurements to determine the surface strains in the concrete due to the release of the prestress.

The camber of the test girders was measured immediately after the prestress was released. The camber was determined by measuring the distance between the bottom of the girders and the top of the prestressing bed with a ruler at 1/4 points along the length of the girders. The employees at the fabricating yard were surprised by the large amount of camber (Figure 5.16). The midspan camber was recorded as follows:

Girder I = 4.8 in.

Girder II = 3.9 in.

Cracks in the test girders were documented. Most cracking at release occurred in the bottom flange and web of the end regions. Figure 5.17 shows the worst of the cracking found in the girder end with fully draped strands. The other three ends used a draped/debonded strand combination and no web cracking was observed, only minor (1-2) cracks in the bottom flange were noted. All of the cracks due to shrinkage that appeared in Girder II before release had completely closed and could no longer be detected after transfer.

The next day, any new cracks that formed since the last inspection were marked and documented; only minor additional cracking was observed. Since the fabricator needed to put the prestressing bed back into production, the test girders were moved to a storage area in the yard. Before the girders were moved all instrumentation was monitored and camber was measured. Figure 5.18 shows a timeline history for the two girders from the time of casting until release. Major milestones, such as cylinder strengths and form removal are noted on these timelines.

After the camber was measured, the north end of each test girder was lifted off the bed and set back down; the camber was re-measured. This was done to determine whether the friction between the bottom of the girders and the prestressing bed had an effect on the camber. The midspan camber after the lift was recorded as follows:

Girder I = 5.5 in.

Girder II = 4.1 in.

To evaluate the stability of the test girders during their handling, Girder II was lifted and held above the bed for 5 minutes. During this time tiltmeters installed earlier were monitored. No adverse rotation or rolling of the girder was detected. The girder was then set back down on the bed. Camber measurements were then taken and the girder was inspected for new cracks; none were found. The test girders were then moved to the storage area that was about 100 yards away. No stability or handling problems were noticed during the move. Instrumentation was monitored periodically and camber was measured throughout the period when the girders were stored at the prestressing yard.

With the test girders instrumented and fabricated, the girders were ready to be shipped. The next chapter will describe the method by which the test girders were transported and installed at the testing site.

Chapter Six - Transportation of the Test Girders

6.1 Transportation of the Test Girders to the Testing Site

As soon as the testing facility was made ready, the test girders were transported to the off-campus testing site by LeFebvre & Sons Trucking Co. This company is used regularly to transport girders for the fabricator. The truck that transported the test girders was designed to carry long bridge girders by supporting the front end of the girder at the truck tractor and the rear end of the girder by a truck bogey. The truck bogey, a hydraulically self-contained unit with steerable axles, was connected to the tractor trailer by the bridge girder. The steering of the rear truck bogey was remotely controlled by the truck driver via a control cable that was chained along the length of the girder during transportation.

Each test girder was lifted onto a truck by an overhead crane lifting at each end lift hook location as shown in Figure 6.1. The girder was then set onto the truck, which had a cradle made of wood timbers that would support the girder near the end bearing plate locations. It was then tied down with chains around the top of the girder section. Each test girder was then transported separately to the testing site by the same truck (Figure 6.2). After the trips, discussions with the truck driver indicated the test girders exhibited no adverse transportation characteristics while traveling at interstate highway speeds.

6.2 Installation of the Test Girders into the Laboratory

Prior to arrival of the test girders at the testing location, preparations were made by Truck Crane Service Co. to install the girders into the laboratory. The layout of the building, shown in Figure 6.3, made it impossible to bring the girder into the building through the large overhead door. Truck Crane Service Co. proposed that the girders be installed into the building through a small door at end of the bay in which the girders were to be tested.

Once the first truck and girder arrived at the building, the truck was backed up to the door (Figure 6.4). The back end of the girder was then lifted off the rear truck bogey with a truck crane and the bogey was moved out of the way. The truck backed the end of the girder into the building while the crane swung its boom. With about 5 ft. of the girder end inside of the building, the crane lowered the end of the girder onto a large steel skate which rode on a path of steel plates. The truck backed the girder into the building as far as it could. The front end of the girder was lifted off the truck by the crane and swung the front end of the girder into the building and then lowered it onto another large steel skate (Figure 6.5). The girder was then rolled into the building, by a fork-lift truck, until it was adjacent to the test girder abutments. Each end of the girder was then lifted by a large loader type truck, as shown in Figure 6.6, with forks; the steel skate was then rotated 90 degrees and the end was set back down. The girder was rolled sideways next to the abutments with two fork-lift trucks. Each end of the girder was then raised to an elevation of about 1 ft. above the top of the abutments, about 6 ft. above the floor, by a lift and block method. The girder was then set back down onto the skates and skidded sideways until it was centered above the abutments. Each end of the girder was then lifted, so the skate and blocking could be removed, and then set down onto the abutments. A similar method was used to install the second girder onto abutments next to the first girder. It took a total of 20 hours, with a crew of 8 people, for Truck Crane Service Co. to install both test girders in the testing laboratory.

The overall operation went smoothly and as expected. No additional cracking was observed due to transportation or girder placement. The test girders were ready to have the composite concrete deck cast on each of them.

Chapter Seven - Fabrication of the Composite Concrete Bridge Decks

7.1 Description and Design of Composite Decks

Once the test girders were delivered and placed in the testing facility, preparations began for casting the composite concrete deck on top of each test girder. A description of the composite decks as well as the design methodology is discussed in Section 2.2.3. Each deck was 48 in. wide (the centerline spacing of girders in the design bridge), 10 in. thick (this includes a 9 in. deck with a 1 in. haunch) above the precast girder sections and 9 in. thick in the span between girders as shown in Figure 2.6. This deck thickness is the standard used by Mn/DOT [3] in composite bridge construction.

7.2 Formwork

It was the intent of this project to cast each concrete deck using unshored construction techniques similar to those typically used in Mn/DOT bridge construction. Because each composite test girder was designed to be tested individually, the deck above each girder was cast separately but simultaneously. The layout of the girders at the laboratory made it possible to place the formwork around and between the girders and to shore the forming to the girder sections. The limited clearance between the two test girders was such that it was difficult to form and shore the deck of each girder separately. For this reason, the shoring of the formwork between the girders was shared by both specimens. This seemed reasonable since both concrete decks were cast at the same time.

The concrete forming was primarily 1/2 in. plywood. The falsework used was hardware typically used by bridge contractors. Between the girders, adjustable joist hangers were used to support the plywood forming (Figure 7.1). The joist hangers were spaced at 24 in. on centers with wood 2x4 or 2x6 boards being used as joists between the hangers. A steel plate former was used with these joist hangers to provide the vertical forming surface for the 1 in. haunch typical along the length of the test specimens.

At the exterior forming of the decks, adjustable fascia overhang brackets were used in conjunction with adjustable half hangers. The half hangers were 1/2 in. contour threaded rods with a clip that was clamped around the vertical shear stirrups that extended above the surface of the precast section. The half hangers provided support for another 1/2 in. contour threaded rod which served as a hanger for the overhang brackets. The overhang brackets, shown in Figure 7.2, were supported vertically by the threaded rod and horizontally by a diagonal kicker which bear against the web of the precast girder sections. The overhang brackets were spaced at 16 in. on centers, or one bracket per vertical shear stirrup spacing. Each of the falsework hardware types were borrowed either from a local construction contractor or their suppliers.

After the formwork and shoring was placed, the inside surface of the plywood forms was coated with a release agent. The release agent used was a brand commonly available and used by concrete contractors.

7.3 Placement of Mild Steel Deck Reinforcement

Once the deck forming was completed, the mild steel reinforcement was placed as shown in Figure 2.7 and described in Section 2.2.3 of this report. All mild steel reinforcement used in the decks was No. 4, Grade 60 ksi, epoxy coated deformed bars. Following the directions of the Mn/DOT Bridge Design Manual [3], the reinforcement was placed in two layers in both directions. Because the deck width was 48 in. (the deck cantilevered 9 in. over the girder flanges) and did not actually span between girders, no flexural reinforcement was required transverse to the girder spans. The only transverse reinforcement placed was for shrinkage and temperature effects and was designed to satisfy ACI [5] minimum requirements.

The reinforcement was ordered from the rebar manufacturer cut to length and epoxy coated so no bars had to be field cut. The longitudinal bars were delivered in 45 ft. lengths while the transverse bars came 44 in. long. A 15 in. lap splice was used for the longitudinal reinforcement as prescribed by equation 8.32.4 of the AASHTO Specifications [2] for lap splices in compression.

A 3 in. minimum cover was used for the top bars while the bottom bars had a 1 in. cover at the deck span; a 2 in. cover was used at the ends of bars. Non-epoxy coated chairs were used to support the bars during the concrete casting. The top bars used 6-1/2 in. high rebar chairs while the bottom bar chairs were 2 in. high. All deck reinforcement was tied together with non-epoxy coated standard rebar ties. The deck reinforcement placed in the forms can be seen in Figure 7.3.

7.4 Placement of Deck Instrumentation

The instrumentation installed into the concrete deck of the test girders was similar to that used in the precast girder sections. This instrumentation included strain gages bonded to the mild steel reinforcement, vibrating wire gages and concrete strain gages cast into the concrete and DEMEC points cast onto the top surface of the concrete decks. Chapter Four gives a description and reasons for using these instrumentation types.

Once the mild steel reinforcement was placed, the rebar strain gages were installed. This process included the removal of the epoxy coating, sanding away the rebar deformations and bonding a strain gage to the top of the rebar at each location. A total of 18 rebar strain gages were installed in the deck of Test Girder I, while the deck of Test Girder II had a total of 19 strain gages. The vibrating wire gages and concrete strain gages were installed next. They were suspended in the deck in a similar fashion as the gages that were used in the precast girder section. These gages were installed so their longitudinal axis was parallel to the longitudinal axis of the girder sections. A total of 5 vibrating wire gages and 17 concrete strain gages were installed into each composite concrete deck (Figure 7.4 shows a typical location of concrete, rebar and vibrating wire gages installed in the concrete deck). Figure 4.5 and Appendix E show the nominal and actual locations of these electrical type gages installed into the decks. The cabling for these gages was routed through the side of the formwork so they wouldn't be damaged during the casting of the deck concrete.

After the concrete was cast, DEMEC points were installed into the top surface of the concrete deck above each test girder. The locations of these DEMEC points is shown in Figure 4.4. Steel strips, similar to the ones used for the DEMEC points installed onto the girder sections, were used to embed the brass inserts into the top of the deck after the fresh concrete was screeded off. After the concrete was cured for approximately twelve hours, the steel strips were removed from the brass inserts and the stainless steel tips were installed into the brass inserts. Because the DEMEC points were arranged in a two-dimensional array, readings were taken in both the longitudinal and perpendicular directions of the girder.

7.5 Casting and Curing of the Deck Concrete

The concrete for the girder decks was supplied by a local ready-mix company: Cemstone Products Co. The concrete mix specified for the decks, given in Table 7.1, was Mn/DOT 3Y33 [9]; this is a concrete mix typically specified by Mn/DOT for the construction of their bridge decks. This concrete mix had a design 28 day compressive strength of 4300 psi with a nominal water-to-cement ratio of 0.42 and a 3 in. slump. An estimated total of 16 cubic yards of concrete was needed for each girder deck.

Because the test girders were not delivered to the testing facility until the fall season, the forming and casting of the decks was completed during the winter months. At the time the decks were cast the temperature of the laboratory was below freezing and heat had to be applied to the girders and the area around them so the deck concrete would not freeze during curing. To keep the fresh concrete from freezing during its casting and curing, a temporary tent made of polyethylene sheets was set up below the deck forms around the perimeter test girders. Several days before the deck concrete was cast, a 500,000 BTU natural gas heater was installed under the tent to warm the girder sections to about 75 degrees and construction tarps were laid on top of the deck forms over the polyethylene sheets to prevent heat loss and warm the rebar and formwork.

Before each load of concrete was accepted from the ready-mix concrete supplier, slump-cone (ASTM C 143 [8]) and air entrainment (ASTM C 173 [8]) tests were preformed on the batch. A nominal 3 in. slump and 5.5% (+/-1.5%) added air was specified for the deck concrete. During the pour, concrete cylinder and beam samples were taken to determine the structural properties of the deck concrete; the concrete batch from which each sample was taken was documented. An inventory of the concrete samples taken for the concrete deck along with the schedule of their testing is shown in Table 3.3. Preliminary test data are summarized in Table 3.4.

The concrete was placed in the deck forms from the ready-mix trucks via a machine that was equipped with a telescoping conveyor belt on a rotating turret; this unit was supplied by the ready-mix company. The concrete was carefully placed so as not to disturb any of the instrumentation. Vibration from hand-held units was used throughout the casting operation to consolidate and ensure there were no voids in the concrete. The concrete was then screeded (struck) off to form a flat surface.

Because of the layout of the laboratory bays and the effective boom length of the conveyor, the concrete could not be poured into each deck continuously. After casting about a 50 ft. length of deck on each test girder, the conveyor machine had to relocate its turret base along the length of the girder; this caused the formation of at least 3 wet construction joints along the length of the decks. It was also necessary to pour both decks simultaneously (rather than each deck continuously). Since the formwork between the girders was shored by both girders, it was desired to keep the deflection of each girder section (due to the weight of the wet concrete) about the same so the forms would stay level. The sequence of the deck casting and locations of wet construction joints is shown in Figure 7.5.

After screeding the concrete, the DEMEC points were installed as previously described in Section 4.2.2 (Figure 7.6). A layer of polyethylene sheeting was then placed over the decks, and the concrete cylinder and beam samples were placed next to the forms at the approximate location where the samples were taken from the pour. Insulated construction blankets were then placed over the decks and cylinders. The heat was maintained under the tent for a curing period of 4 days and slowly lowered to the ambient temperature of the laboratory over a 3 day period following curing. The insulating blankets and polyethylene sheets were removed after 7 days of curing. Formwork removal began shortly after the blankets were removed.

A vertical crack about 26 ft. long was found in the concrete deck of Girder I as shown in Figure 7.5. This crack was thought to be created when the concrete was in its plastic state from the rotation of the formwork between the test girders when Test Girder II deflected from the weight of the fresh concrete. The crack was eventually filled with an epoxy adhesive once the weather warmed up.

Chapter Eight - Description of the Test Setup

Because of the length of the test girders, 132.75 ft., it was not possible to load test them at the University of Minnesota Department of Civil and Mineral Engineering structures laboratory which has a 40 ft. by 80 ft. clear area for testing. It was determined that an area of about 40 ft. by 145 ft. with a minimum 20 ft. ceiling height would be necessary to test these girders. A Mn/DOT maintenance facility was found that had enough unoccupied space to fulfill the project needs; approval was given to use this space as the laboratory for testing the girder specimens.

This building (see Figure 6.3 for the building layout) consisted of a pre-fabricated steel frame structure that was supported from a slab-on-grade with precast concrete wall panels. The building had adequate electrical, gas and water services, but the space was unheated. There were two significant disadvantages in using this building as a structural testing laboratory: there were no overhead lifting capabilities and there was no structural strong floor to provide an uplift resistance for load testing frames. To help with the first problem, Mn/DOT provided a small tractor type loader kept at the building site which could be used for lifting smaller items.

The proposed method of load testing the girder specimens in flexure was to use two point loads, at 40% of the span length from each girder end, to provide a constant bending moment at the region between the load points including midspan. The point loads would be supported by a steel frame. To provide an uplift reaction for the load testing frames, two schemes were considered. The first option considered using a gravity load to offset the uplift forces of the load testing frame. The second option considered using a soil anchor type foundation system to resist uplift forces. The second option was chosen because of construction feasibility and economic considerations.

8.1 Soil Reaction Anchors

To provide an uplift resistance for the load frames, soil anchors were installed through the slab on grade into the ground at locations were the load testing frames were to be mounted. The A. B. Chance Co. soil anchors used were earth anchors constructed of helical-shaped circular steel plates of varying diameter welded to a square steel shaft at a given spacing. These anchors were turned into the ground by a tractor type auger. The soil anchors were supplied and installed by a local foundations contractor, Atlas Foundation Co.

The required uplift capacity for the soil anchors was determined by the load testing capabilities of the hydraulic load actuators to be used during testing. The ultimate flexure tests required two - 77 kip actuators at each of the two load frames. The ultimate shear tests required two - 200 kip load jacks at one load frame only. To minimize the required number of soil anchors, the load frames were located such that common column base connections would be used between the test girders. Since each test girder would be tested separately, only two load testing frames would be needed for the testing program. A total of six soil anchor locations were required, four of these locations required a 77 kip minimum uplift capacity while the other two locations required 200 kip uplift capacity. See Figure 6.3 for the testing laboratory layout and soil anchor locations.

The typical soil anchor used consisted of a 1-3/4 in. square steel shaft that had a lead section with four helices of different diameters: 8, 10, 12 and 14 in. with the smallest helix at the bottom (Figure 8.1). The helices were pitched so each plate penetrated the soil in the same path as the previous plate to reduce soil disturbance. The top of each soil anchor was terminated with an adapter tapped for a 1 in. diameter threadbar. Each soil anchor had a guaranteed ultimate tensile capacity of 100 kips.

The uplift capacity of these soil anchors was also a function of the soil they bore against. Because no formal tests were performed on the soil below the laboratory site, except for soil borings to determine general soil types, the uplift capacities of the soil anchors could not be estimated using soil bearing/friction theory. The soil anchor manufacturer suggested that the uplift capacity could be estimated by a rule-of-thumb 10:1 ratio of the anchor holding capacity to the installation torque. Therefore the ultimate tensile capacity of these soil anchors could be utilized if they were installed with the maximum 10 ft-kip torque (the higher the soil bearing capacity the higher the required soil anchor installation torque). The soil anchors installed at the laboratory were augured in until this maximum applied torque was reached. The depth of the installed anchors was between 18 - 23 ft. below the bottom of the concrete slab in the laboratory. At the column locations where an uplift capacity of 77 kips was required, a single soil anchor was installed. Where the uplift capacity of 200 kips was required, two soil anchors were installed at a 24 in. spacing and at a slight angle (batter) away from each other so each one would be founded in its own soil column.

The soil anchors were load tested after their installation. The load testing of each anchor consisted of an applied uplift force of 74 kips held for 30 minutes and then 5 load cycles of 0 - 20 kips. The relative displacement of each anchor was measured throughout the load testing; no noticeable slippage of the anchors was measured. Because these anchor systems were primarily designed for guy wire applications, a failure due to anchor fatigue would not be a concern in these cases.

Once the soil anchors were installed and load tested, a detail for the column base plate-to-soil anchor connection had to be devised. It was desired to have two anchor bolts at each column base plate and have access to the top nut of the soil anchor in case it would need to be tightened should the soil loosen-up during the flexural fatigue testing of the girders. The column base connections, see Figure 8.2, consisted of a 2 x 4 in. steel bar - 14 in. long with two 1-1/2 in. diameter threaded rods at 10 in. on centers welded to the bar. At the center of the bar was a 1-5/8 in. diameter hole to accept the 1 in. threadbar attached to the soil anchor below. This connection was embedded in a 36 x 24 in. by 24 in. deep block of concrete and doweled to the existing concrete slab with No. 5 rebar. The connection of the soil anchor-to-steel bar was isolated from the concrete block by a length of 4 in. diameter plastic pipe. After the nominal 4000 psi concrete cast around the connection had cured for about two weeks, the soil anchors were preloaded (the preload force compressed the soil column between soil anchor and the base plate connection, to a force approximately equal to the flexural fatigue load that would be used in the girder tests, to help eliminate the possibility of the degradation of soil strength due to fatigue loading) with a minimum force of 20 kips each by tightening the threadbar nut using a DYWIDAG Systems International threadbar post-tensioning system.

8.2 Load Frames

Each load testing frame, shown in Figure 8.3, consisted of two W12 x 65 Grade 50 steel columns, 15 ft. tall with a 1-1/2 x 16 x 16 in. Grade 36 steel base plate fully welded to the bottom of each column. Each column was fabricated with a 15/16 in. diameter hole pattern with a horizontal gage of 6 in. and a vertical gage of 4 in. along the height. The hole pattern was punched on the web and one flange only. Because a column height of 20 ft. was required for the clearance of the actuators and the composite girder section ultimate flexural deflection, a 5 ft. tall stub column was installed beneath each of the 15 ft. column sections. These were W14 x 99

Grade 36 steel stub columns with cap/base plates similar to those used on the W12 x 65 columns and welded to each end. Each column section was connected together with two 1-1/2 diameter high strength bolts along with 1-1/2 x 4-1/2 x 8-1/2 in. steel pillow blocks to reduce the possibility of local plate bending.

A TS 12 x 16 x 1/2 Grade 46 steel cross member 66-3/4 in. long was bolted between the column flanges at the top of each load frame to provide a support for the hydraulic actuators. A 1 x 14 in. Grade 36 steel plate, 40 in. long, with a bolt hole pattern similar to that in the column flange was attached to each end of the tube section with a fillet weld around perimeter of the tube section. Each end of the cross member was bolted to the column flange with twelve 7/8 in. diameter A325 bolts. Each cross-member was drilled to accept the bolt pattern of two 77 kip hydraulic actuators. An adapter plate was manufactured so a single 35 kip hydraulic actuator could be mounted to the midspan of each cross member. The actuators were attached to the cross members using 3/4 in. diameter threaded rods.

To spread the load from the actuators to the girder sections, a spreader bar the width of the concrete deck was used below each actuator. Below the 10 in. stroke actuator, a TS 10 x 4 x 3/8 steel tube was used, while a TS 10 x 6 x 3/8 steel tube was bolted to the bottom of the 6 in. stroke actuator; each spreader tube was 4 ft. long. Grout was added under each spreader bar to ensure a level, uniform contact surface. Each load actuator was braced from movement in the plane of the load frame with threaded rods.

8.3 Hydraulic System and Load Actuators

The hydraulic loading equipment used for this project was manufactured by a local firm: MTS Systems Corporation. The hydraulic power for the loading system was supplied by a 21 gallon per minute MTS pump. The pump operated at a pressure of 3,000 psi and used a water-cooled heat exchanger to control the operating temperature of the hydraulic fluid. The hydraulic fluid was driven through 1-1/2 in. diameter high pressure hoses to the load actuators via a fluid accumulator. Multiple load actuators were connected into the fluid circuit in parallel.

The flexural fatigue testing used two actuators, one on each load frame. These were MTS fatigue rated actuators with a loading capacity of 35 kips each; one of these actuators had a 10 in. stroke range while other had a 6 in. stroke range. The stroke of the actuator was controlled by one or more hydraulic servovalves which were controlled either by a load feedback signal or a displacement feedback signal. The signal input/output to each load channel was regulated by a MTS 406 Controller, while the hydraulic circuit for each was controlled by a MTS 436 Control Unit. A MTS 410 Function Generator provided a harmonic load/displacement signal for the flexural fatigue tests and a ramp signal for the static tests. The load and displacement of the actuators was monitored with a MTS 464 Data Display. A fail-safe interlock system was built into the control circuit to shut down the hydraulic power should any loading parameters go beyond preset limits.

The ultimate flexure tests will use two 200 kip hydraulic load jacks. These load jacks do not have any type of load/displacement feedback system; only a monotonic load will be needed for these tests. The applied load was monitored with load cells at the location of the applied loads. Displacements and rotations were monitored with LVDTs.

The load for the ultimate shear test was provided by an MTS 600 kip universal testing machine. The girder ends were transported back to the University of Minnesota Structural Engineering Laboratory for these tests.

8.4 Reaction Blocks

To provide an abutment at each end of the test girder specimens, four concrete reaction blocks were fabricated. The reaction blocks resist the support load of the test girders by bearing against the slab on grade concrete floor of the test laboratory. The height of the reaction blocks was dictated by the clearance required between the bottom of the girder section and the concrete floor during the ultimate flexure tests. A midspan ultimate flexural deflection of about 4 ft. was predicted, so the reaction blocks were fabricated at a height to give a 5 ft. clearance at the bottom of the girder sections. To provide overturning stability, the reaction blocks had relatively large base dimensions: 4 x 4 ft (Figure 8.4). Each reaction block was cast-in-place with nominal 4,000 psi concrete and reinforced with column rebar along with shrinkage and temperature steel; minimum steel requirements of ACI 318-89 [5] were adhered to in their design.

In actual prestressed bridge girder construction, the sole plate at the girder support bears against an elastomeric pad that provides adequate girder rotation and expansion flexibility under service load conditions. It was determined that the use of these elastomeric bearing pads would not allow enough end rotation and travel at the ultimate flexure load. To provide enough rotation and horizontal travel, a 3-1/8 ft. diameter steel roller, 22 in. long was centered under the sole plate at each girder end. The rollers were set on a 2 x 12 ft. steel plate, 24 in. long, that was embedded into the top of each reaction block. At one end of the girder the roller was welded to the embedded plate, while the roller at the opposite end was allowed to roll freely.

Chapter Nine - Summary and Conclusions

9.1 Summary

Current AASHTO [2] prestressed concrete bridge girder design specifications were based on empirical data from test specimens with concrete compressive strengths less than 6,000 psi and 0.5 in. diameter prestressing strand. Using these design specifications, two Mn/DOT 45M composite girder specimens utilizing 10,500 psi concrete and 0.6 in. diameter strand were designed to achieve a maximum length (132 ft.- 9 in.) using a minimal girder spacing (4 ft.). To evaluate the use of high strength concrete in bridge girders using current design specifications, the resulting design girders were fabricated and tested as previously described.

The two high strength concrete prestressed bridge girders were fabricated and instrumented to investigate the feasibility of constructing long span high strength concrete prestressed bridge girders. The test girders were cast in August of 1993, a composite concrete deck was cast on each of the girders in February of 1994. Testing of the girders began in September of 1994.

The test girders were instrumented with over 660 steel and concrete strain gages, vibrating wire gages, DEMEC points, displacement/rotation transducers including LVDTs. This instrumentation was used to obtain information on the effect of high strength concrete on strand transfer length, prestress losses, camber, flexural fatigue and ultimate flexural and shear strengths.

9.2 Conclusions

From the fabrication of the two test girder specimens, the following conclusions were made:

1) The possibility of damaging strain gages bonded to prestressing strands in congested sections can be reduced if the strands are separated during gage installation and then stressed in order from the highest strand to the lowest strand.

2) No installation problems were encountered when the modified Mn/DOT "U" stirrup was used at the girder end regions.

3) High strength concrete with compressive strengths of 12,000 psi can be produced using locally available materials. Rigorous quality control practices (water to cement ratio, aggregate type and gradation, amount of water reducing and superplasticizer agents added and slump cone test) must be used to ensure consistent concrete strength, workability and placement in narrow congested girder sections.

4) Shrinkage cracking was observed to penetrate the cross section from the top flange to the bottom flange in the test girder with glacial gravel aggregate/microsilica concrete. This cracking was due to the restrained shrinkage of the concrete prior to the release of the prestress in the strands and occurred approximately 2 hours after the forms for this girder were removed. The shrinkage cracks were observed to fully close after release of the prestressing. After release, cracking at the support regions was observed in all four test girder ends. Some of these cracks were observed to penetrate the cross section either inclined or vertically from the bottom flange and into the web, while others were in the low in the bottom flange parallel to the length of the girders. The cracks in the end regions never closed.

5) Both girders demonstrated substantial initial cambers due to the large amount of prestressing. Initial cambers were in the range of 3.9 in. to 5.5 in., similar to the predicted cambers of 4.2 in. and 4.6 in.

6) No adverse handling characteristics were observed when the test girders were moved at the prestressing yard and when they were transported to the testing site. Lifting hook locations were given by PCI [10].

From the design of the test girders the following conclusions were made concerning the performance of high strength concrete:

7) The design to maximize girder length was controlled by midspan service stresses. The stress limit is reached due to the maximum amount of prestressing strands that can fit in the girder section at an effective eccentricity. Because of this effect, it was found that no additional benefit was realized in the girder design for concrete with strength greater than 12,000 psi.

8) The use of 0.6 in. diameter prestressing strand increased the effective prestressing force on the girder section by 49 percent when compared to the use of 0.5 in. diameter strand. The final girder design yielded a 48 percent increase in girder span when compared to a design using 6,000 psi concrete and 0.5 in. diameter strand.

9) Because HSC has a relatively small effect in increasing flexural stiffness, the extreme girder length that can be attained using HSC may have adverse service characteristics with respect to camber, deflection and the member's natural period of vibration.

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Tables

Table 2.1 Test Girder Design Loads

Load Description	Design [plf]	Actual [plf]
Girder Self Weight Girder I Girder II	 671.7 671.7	 710* 716*
SDL on precast section due to bridge diaphragms	20	0
Composite deck self weight (including wearing course)	481.3	479*
SDL on composite section due to guard rails	203	#
Truck live load	HS25 Truck **	#

SDL = Super-imposed Dead Load

* Weight of concrete determined by average weight of 6 cured test cylinders. Volume of concrete replaced by reinforcement taken into account. As-built dimensions of deck was used in calculations

** Reference Figure 2.8 for HS25 design truck loading

Actual applied SDL and live load to be determined at time of flexural fatigue tests

Table 2.2 Estimated Prestress Losses

AASHTO Losses [1]		At Release [psi]	At Final Service Conditions [psi]
Steel Relaxation: (PCI)	RET *	1760	
Steel Relaxation: (AASHTO)	CRs**		868
Elastic Shortening:	ES	19,653	19653
Concrete Shrinkage:	SH		5750
Concrete Creep:	CRc		37584
Total Loss: as percent of initial pull #		21,412 10.6 %	63855 31.5 %

* Steel relaxation before release of prestressing force [8]

** Steel relaxation at final service conditions

Initial pull assumed to be 75% of ultimate strength (270 ksi) after seating = 202.5 ksi

Table 3.1 High Strength Concrete Mix Designs

Materials	Quantity / Cubic Yard	
	Mix #1	Mix#2
Type III Cement	750 lb	695 lb
Fine Aggregate (Sand)	1330 lb	1290 lb
Coarse Aggregate (5/8" Limestone)	1970 lb	----
Coarse Aggregate (3/4" Glacial Gravel)	---	1880 lb
Water Reducer (WRDA 19)	150 oz.	123.3 oz.
Microsilica	---	56 lb
Water / Cement Ratio	0.323	0.359

Mix # 1 : Crushed Limestone Aggregate Concrete

Mix # 2 : Round Glacial Gravel Concrete with Microsilica

* 28 Day Design Compressive Strength = 10,500 psi

Measured moisture absorption capacity of aggregates, by weight :

limestone - 2.9%, glacial gravel - 2.6%, sand - 3.7%

3.2 High Strength Concrete Aggregate Sieve Analysis

Coarse Aggregates			
Percent Retained By Sieve			
Sieve Size	Mn/DOT Specifications *	5/8" Crushed Limestone	3/4" Round Glacial Gravel
3/4"	0 %	0	3.7 %
5/8"	0 (0 - 15) %	0.1 %	16.2 %
1/2"	0 - 15 (N.S.)%	7.5 %	28.0 %
3/8"	0 - 50 (30 - 60)%	52.9 %	47.9 %
#4	75 - 100 (90 -100) %	94.2 %	90.4 %
#8	N.S.	97.0 %	98.5 %

Fine Aggregates		
Percent Retained By Sieve		
Sieve Size	Mn/DOT Specifications **	Sand
3/8"	0	0
#4	0 - 5 %	0.1 %
#8	0 - 20 %	5.6 %
#16	15 - 45 %	22.2 %
#30	40 - 70 %	47.0 %
#50	70 - 95 %	82.8 %
#100	90 - 100 %	97.6 %
#200	N.S.	99.4 %

Date of Test : 8/10/94
N.S. = Not specified

Aggregate specifications and gradation by Mn/DOT - Standards Specifications for Construction [9]

* Coarse aggregate gradation from item 3137.2E [9]; gradation limits: +/- 5%
Gradation limits in parenthesis is for 3/4" gravel

** Fine aggregate gradation from item 3126.2F [9]; gradation limits: +/- 3%

Gradation limits given by item 2461.2 [9]

Table 3.3 Inventory of Concrete Test Samples and Testing Schedule

GIRDER I - Limestone (black labels)

Concrete Batch #	Total cast	f'c @rel	Ec @rel	f'c 7day	Ec 7day	f'c 28day	Ec 28day	ft 28day	fr 28day	f'c 7-deck	f'c 28deck	f'c fatig	f'c ult	Ec ult	fr ult	f'c shear	ft shear
Cylinders:	8/9/93	8/10	8/10	8/16	8/16	9/15	9/15	9/15	9/15	3/4/94	3/25						
1	15	3		3		2	2	2								3	
2	7					2		2									3
3	8				2	2						1	3				
4	12	2	1			2						3	3	2			
5	8			3		2		1				2					
6	8					2		1		2						3	
7	8	1	1			2				1							3
Beams: 3	3								3								
5	3														3		

GIRDER II - Glacial Gravel w/ Microsilica (red labels)

Concrete Batch #	Total cast	f'c @rel	Ec @rel	f'c 7day	Ec 7day	f'c 28day	Ec 28day	ft 28day	fr 28day	f'c 7-deck	f'c 28deck	f'c fatig	f'c ult	Ec ult	fr ult	f'c shear	ft shear
Cylinders:	8/9/93	8/10	8/10	8/16	8/16	9/15	9/15	9/15	9/15	3/4/94	3/25						
1	15	3		3		2	2					2				3	
2	7					2		1						1			3
3	8				2	2		1					3				
4	6	2	1			2		2									
5	9			3		2							3	1			
6	9					2				2		3					3
7	11	1	1			2		2		1						3	
Beams: 3	3								3								
5	3														3		

Table 3.3 (continued)

DECK - both girders are the same

Concrete Batch #	Total cast	f'c 7deck	Ec 7deck	f'c 28deck	Ec 28deck	ft 28deck	fr 28deck	f'c fatig	f'c ult	Ec ult	f'c shear	f'c shear	f'c shear	f'c shear
Cylinders:	2/25/94	3/4	3/4	3/25	3/25	3/25	3/25				IA	IB	IIC	IID
Truck 1	15	3		3								3		3
Truck 2	18	3	0	3	2			3	3	2				
Truck 3	7	1		3		3								
Truck 4	8	2									3		3	
Beams:														
Truck 1	1						1							
Truck 2	1						1							
Truck 3	1						1							

Table 3.4 Concrete Material Properties for Test Girders

Girder	Concrete Age	Compressive Strength, psi	Modulus of Elasticity, ksi	Modulus of Rupture, psi	Splitting Strength, psi
Girder Concrete Strength					
design	release	8925			
design	28 days	10500			
As Tested by the Girder Fabricator *					
Girder I	release	9291			
	7 days	11380			
	28 days	12100			
Girder II	release	10424			
	7 days	10980			
	28 days	11100			
As Tested by the University of Minnesota **					
Girder I	release	8424	4380		
	7 days	9853	4610		
	28 days	10867	4812	951	1072
Girder II	release	8738	4748		
	7 days	9489	5343		
	28 days	9648	4799	747	1113
Slab Concrete Strength **					
design	28 days	4000			
actual	7 day	5120			
	28 days	5862	4139	632	808

Girder I : Limestone aggregate concrete
Girder II : Glacial gravel aggregate concrete with microsilica

Average measured slump at site : Girder I - 6.8", Girder II - 6.3" (no added air)
Slab - 3.6" with 4 % nominal air entrainment

* Samples used: 4" x 8" cylinders for compressive strength;
Curing: SURE CURE System [5]

** Samples used: 6" x 12" cylinders for compressive strength and splitting strength, modulus of rupture used 6" x 6" x 24" beams;
Curing: samples under test girder curing tarps for 20 hours, then ambient laboratory conditions until testing

All testing procedures followed ASTM Standards [6]

Table 3.5 Prestressing Strand Tensile Test Results

Strand Sample	Test Load		Stress		Measured * Elastic Modulus [ksi]	Measured ** Elastic Modulus [ksi]	Measured *** Elastic Modulus [ksi]
	Py [kips]	Pu [kips]	Fy [ksi]	Fu [ksi]			
A-1	54.1	61.3	237.1	268.7	-	28790	28210
A-2	54.0	61.2	236.8	268.6	-	28800	28510
B-1	54.3	61.2	238.2	268.5	-	29700	29340
B-2	53.5	61.3	234.6	268.9	-	25320	25780
L-1	54.6	61.3	239.6	268.8	28730	28440	26320
L-2	55.0	61.0	241.1	267.6	29030	34830	31080
L-3	54.3	60.9	238.2	267.1	29200	30380	28310
M-1	54.4	61.0	238.7	267.7	29210	32570	29200
M-2	54.6	61.1	239.5	267.8	29210	-	27660
M-3	55.6	61.7	243.7	270.8	29230	30180	30800
Average	54.4	61.2	238.7	268.5	29100	29890	28520

0.6" diameter, 7 wire, Grade 270 ksi low - relaxation prestressing strand

Measured strand area, all 4 samples (used in all calculations): 0.228 sq. in.

Strand yield load was taken at 1% elongation

* Elastic modulus measured from strain gage data

** Elastic modulus measured by Mn/DOT from extensometer data

*** Elastic modulus measured from slope of elastic line on load vs. strain plots from Mn/DOT tensile tests

^ Letter designation indicates a different sample of strand, 4 total: A,B,L & M

Table 3.6 Mild Steel Reinforcement Tensile Test Results.

Rebar Specimen	Measured Area [sq in]	Test Load		Stress	
		Py [kips]	Pu [kips]	Fy [ksi]	Fu [ksi]
Slab Rebar					
1	0.185	13.65	22.23	73.78	120.16
2	0.184	13.80	22.22	75.00	120.76
3	0.186	14.19	22.66	76.29	121.83
Average		13.88	22.37	75.02	120.92
Modified-U Stirrup					
1	0.183	13.48	21.11	73.66	115.36
2	0.176	12.73	20.77	72.33	118.01
3	0.182	13.13	20.95	72.14	115.11
Average		13.11	20.94	72.71	116.16
Mn/DOT U Stirrup					
1	0.186	14.48	22.01	77.85	118.33
2	0.184	13.70	21.94	74.46	119.24
3	0.184	13.40	21.68	72.83	117.83
Average		13.86	21.88	75.04	118.47

Bar size : No. 4

Date of tests : 3/3/94

Table 4.1 Designation of Strand and Concrete Instrumentation.

Reference Figure 4.2 for gage locations
Nominal and actual gage location documented in Appendix D

Gage Location Number	Gage Designation			
	Test Girder End			
	IA	IB	IIC	IID
	Displacement Transducers			
1	R-IA-D1	R-IB-D1	R-IIC-D1	R-IID-D1
	Vibrating Wire Gages			
2	V-IA-2	V-IB-2	V-IIC-2	V-IID-2
3	V-IA-1	V-IB-1	V-IIC-1	V-IID-1
4	V-IA-4	V-IB-4	V-IIC-4	V-IID-4
5	V-IA-3	V-IB-3	V-IIC-3	V-IID-3
6	V-IA-7			V-IID-7
7	V-IA-6			V-IID-6
8	V-IA-5			V-IID-5
	Concrete Gages			
9	R-IA-P2	R-IB-P2	R-IIC-P2	R-IID-P2
10	R-IA-P1	R-IB-P1	R-IIC-P1	R-IID-P1
11	F-IA-P22	F-IB-P22	F-IIC-P22	F-IID-P22
11	F-IA-P32	F-IB-P32	F-IIC-P32	F-IID-P32
12	F-IA-P23	F-IB-P23	F-IIC-P23	F-IID-P23
12	F-IA-P33	F-IB-P33	F-IIC-P33	F-IID-P33
13	F-IA-P21	F-IB-P21	F-IIC-P21	F-IID-P21
13	F-IA-P31	F-IB-P31	F-IIC-P31	F-IID-P31
14	F-IA-P55			F-IID-P55
15	F-IA-P56			F-IID-P56
16	F-IA-P54			F-IID-P54
17	F-IA-P53			F-IID-P53
18	F-IA-P52			F-IID-P52
19	F-IA-P51			F-IID-P51
	Strand Transfer Gages			
20				T-IID-02A
20				T-IID-02B
20				T-IID-02C
21				T-IID-05A
21				T-IID-05B

(continued)

Table 4.1 (Continued)

Gage Location Number	Gage Designation			
	Test Girder End			
	IA	IB	IIC	IID
	Strand Transfer Gages			
21				T-IID-05C
22		T-IB-07A	T-IIC-07A	
22		T-IB-07B	T-IIC-07B	
22		T-IB-07C	T-IIC-07C	
23				T-IID-09A
23				T-IID-09B
23				T-IID-09C
24		T-IB-035A	T-IIC-035A	T-IID-035A
24		T-IB-035B	T-IIC-035B	T-IID-035B
24		T-IB-035C	T-IIC-035C	T-IID-035C
25	T-IA-25A	T-IB-25A	T-IIC-25A	
25	T-IA-25B	T-IB-25B	T-IIC-25B	
25	T-IA-25C	T-IB-25C	T-IIC-25C	
26	T-IA-28A	T-IB-28A	T-IIC-28A	
26	T-IA-28B	T-IB-28B	T-IIC-28B	
26	T-IA-28C	T-IB-28C	T-IIC-28C	
27	T-IA-44A	T-IB-44A	T-IIC-44A	
27		T-IB-44B	T-IIC-44B	
27	T-IA-44C	T-IB-44C	T-IIC-44C	
28	T-IA-49A	T-IB-49A	T-IIC-49A	
28		T-IB-49B	T-IIC-49B	
28	T-IA-49C	T-IB-49C	T-IIC-49C	
29	T-IA-123A	T-IB-123A	T-IIC-123A	
29	T-IA-123B	T-IB-123B	T-IIC-123B	
29	T-IA-123C	T-IB-123C	T-IIC-123C	
30	T-IA-1210A	T-IB-1210A	T-IIC-1210A	
30	T-IA-1210B	T-IB-1210B	T-IIC-1210B	
30	T-IA-1210C	T-IB-1210C	T-IIC-1210C	
31	T-IA-202A	T-IB-202A	T-IIC-202A	
31	T-IA-202B	T-IB-202B	T-IIC-202B	
31	T-IA-202C	T-IB-202C	T-IIC-202C	
32	T-IA-2011A	T-IB-2011A	T-IIC-2011A	
32	T-IA-2011B	T-IB-2011B	T-IIC-2011B	
32	T-IA-2011C	T-IB-2011C	T-IIC-2011C	

(continued)

Table 4.1 (Continued)

Gage Location Number	Gage Designation			
	Test Girder End			
	IA	IB	IIC	IID
	Strand Flexure Gages			
33	F-IA-12	F-IB-12	F-IIC-12	F-IID-12
33	F-IA-C12			F-IID-C12
34	F-IA-15	F-IB-15	F-IIC-15	F-IID-15
34	F-IA-C15			F-IID-C15
35	F-IA-17	F-IB-17	F-IIC-17	F-IID-17
35	F-IA-C17			F-IID-C17
36	F-IA-19	F-IB-19	F-IIC-19	F-IID-19
36	F-IA-C19			F-IID-C19
37	F-IA-25	F-IB-25	F-IIC-25	F-IID-25
37	F-IA-C25			F-IID-C25
38	F-IA-35	F-IB-35	F-IIC-35	F-IID-35
38	F-IA-C35			F-IID-C35

Table 4.2 Designation of Shear Instrumentation

Reference Figure 4.6 for gage locations
Nominal and actual gage locations documented in Appendix D

Gage Location Number	Gage Designation
	WFLA Stirrups gages
1	S - X - S1B
2	S - X - S1C
3	S - X - S3B
4	S - X - S3C
5	S - X - S4A
6	S - X - S4B
7	S - X - S4C
8	S - X - S5A
9	S - X - S5B
10	S - X - S5C
11	S - X - S6A
12	S - X - S6B
13	S - X - S6C
14	S - X - S7A
15	S - X - S7B
16	S - X - S8A
17	S - X - S8B
18	S - X - S12A
19	S - X - S12B
20	S - X - S12C
21	S - X - S13A
22	S - X - S13B
23	S - X - S13C
24	S - X - S15A
25	S - X - S15B
26	S - X - S18A
27	S - X - S18B
28	S - X - S18C
29	S - X - S22B
30	S - X - S22C
31	R - X - B1
32	R - X - B2
33	R - X - B3

Gage Location Number	PMR - 60 Concrete Rosettes		
34	S - X - R1A	S - X - R1B	S - X - R1C
35	S - X - R2A	S - X - R2B	S - X - R2C
36	S - X - R3A	S - X - R3B	S - X - R3C
37	S - X - R4A	S - X - R4B	S - X - R4C
38	S - X - R5A	S - X - R5B	S - X - R5C

X = Represents the test girder end for the instrumentation: IA, IB, IIC, or IID
The shear instrumentation in each girder end is similar

4.3 Designation of Concrete Deck Instrumentation

Reference Figure 4.5 for gage locations
Nominal and actual gage locations documented in Appendix E

Gage Location Number	Gage Designation			
	Test Girder End			
	IA	IB	IIC	IID
	Vibrating Wire Gages			
1	V-IAS-2	V-IBS-5	V-IICS-2	V-IIDS-5
2	V-IAS-1	V-IBS-4	V-IICS-1	V-IIDS-4
2	V-IAS-3			V-IIDS-3
	Concrete Gages			
3	F-IAS-P54			F-IIDS-P54
3	F-IAS-P55			F-IIDS-P55
4	F-IAS-P23	F-IBS-P23	F-IICS-P23	F-IIDS-P23
4	F-IAS-P33	F-IBS-P33	F-IICS-P33	F-IIDS-P33
4	F-IAS-P53			F-IIDS-P53
5	F-IAS-P22	F-IBS-P22	F-IICS-P22	F-IIDS-P22
5	F-IAS-P32	F-IBS-P32	F-IICS-P32	F-IIDS-P32
5	F-IAS-P52			F-IIDS-P52
6	F-IAS-P21	F-IBS-P21	F-IICS-P21	F-IIDS-P21
6	F-IAS-P31	F-IBS-P31	F-IICS-P31	F-IIDS-P31
6	F-IAS-P51			F-IIDS-P51
	Rebar Gages			
7	F-IAS-45C		F-IICS-45C	F-IIDS-45C
7	F-IAS-5C			F-IIDS-5C
8	F-IAS-2E			
8	F-IAS-3E	F-IBS-3E	F-IICS-3E	F-IIDS-3E
8	F-IAS-45E	F-IBS-45E	F-IICS-45E	F-IIDS-45E
8	F-IAS-5E			F-IIDS-5E
9	F-IAS-2B			
9	F-IAS-3B	F-IBS-3B	F-IICS-3B	F-IIDS-3B
9	F-IAS-45B	F-IBS-45B	F-IICS-45B	F-IIDS-45B
9	F-IAS-5B			F-IIDS-5B
10	F-IAS-45D		F-IICS-45D	F-IIDS-45D
10	F-IAS-5D			F-IIDS-5D
11	F-IAS-45A		F-IICS-45A	F-IIDS-45A
11	F-IAS-5A			F-IIDS-5A

7.1 Deck Concrete Mix Design

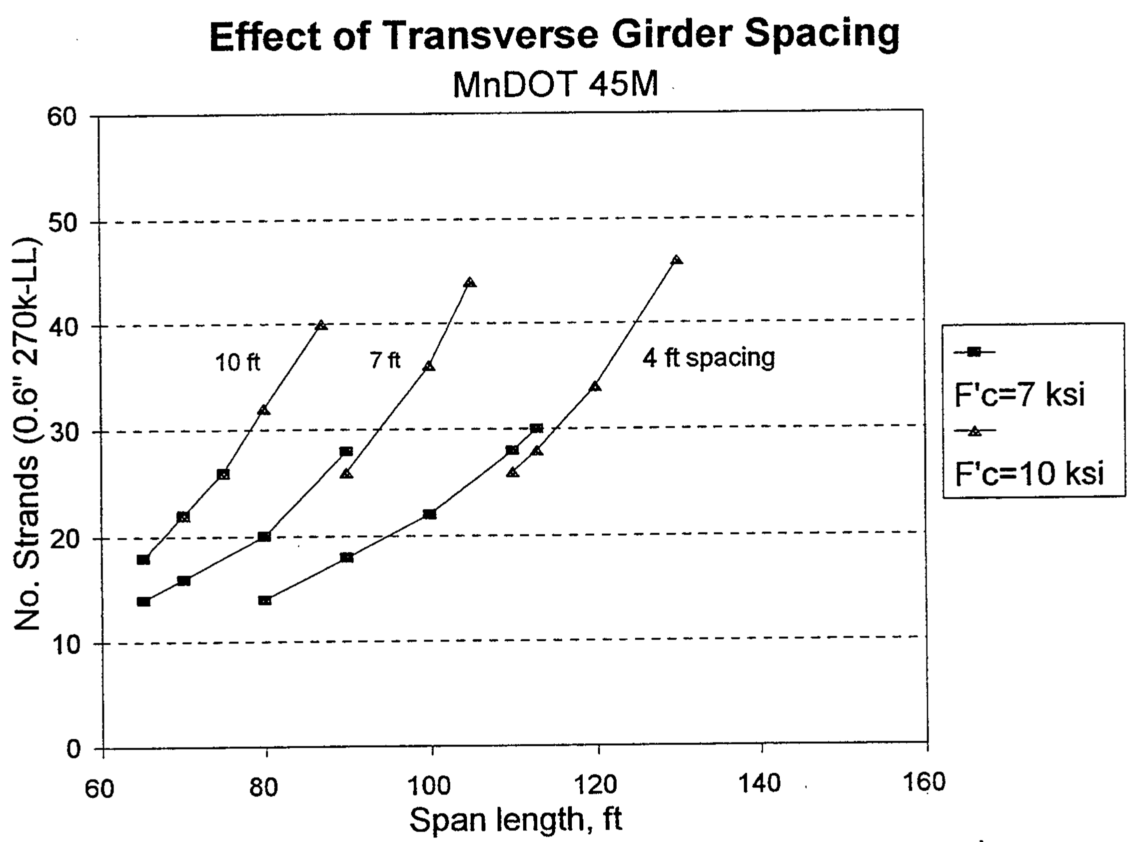
Materials	Quantity / Cubic Yard
Type I Cement	640 lb
Fine Aggregate (Sand)	1195 lb
Coarse Aggregate (3/4" Gravel)	1810 lb
Water	270 lb
Air Entrainment	5.5 % +/- 1.5%
Water / Cement Ratio	0.42

* 28 Day Design Compressive Strength = 4300 psi

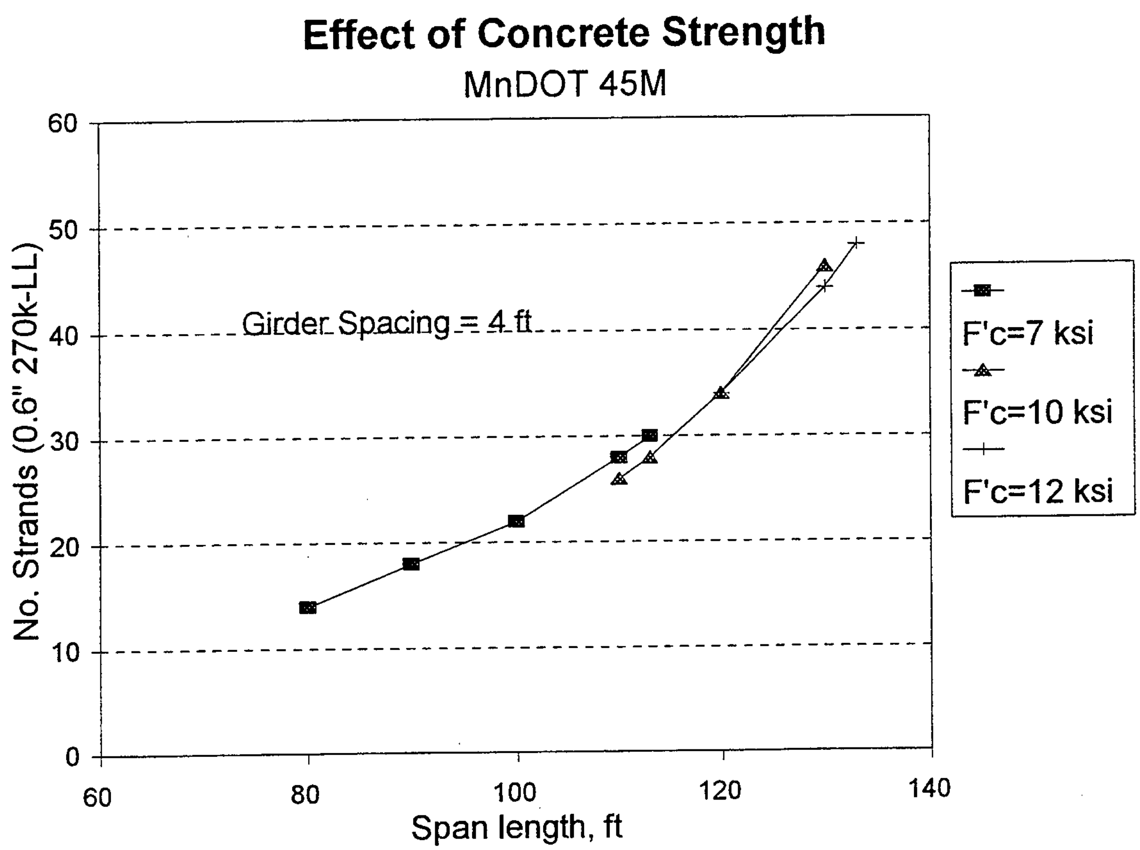
Design slump = 3"

Concrete unit weight = 144.8 lb/cubic ft.

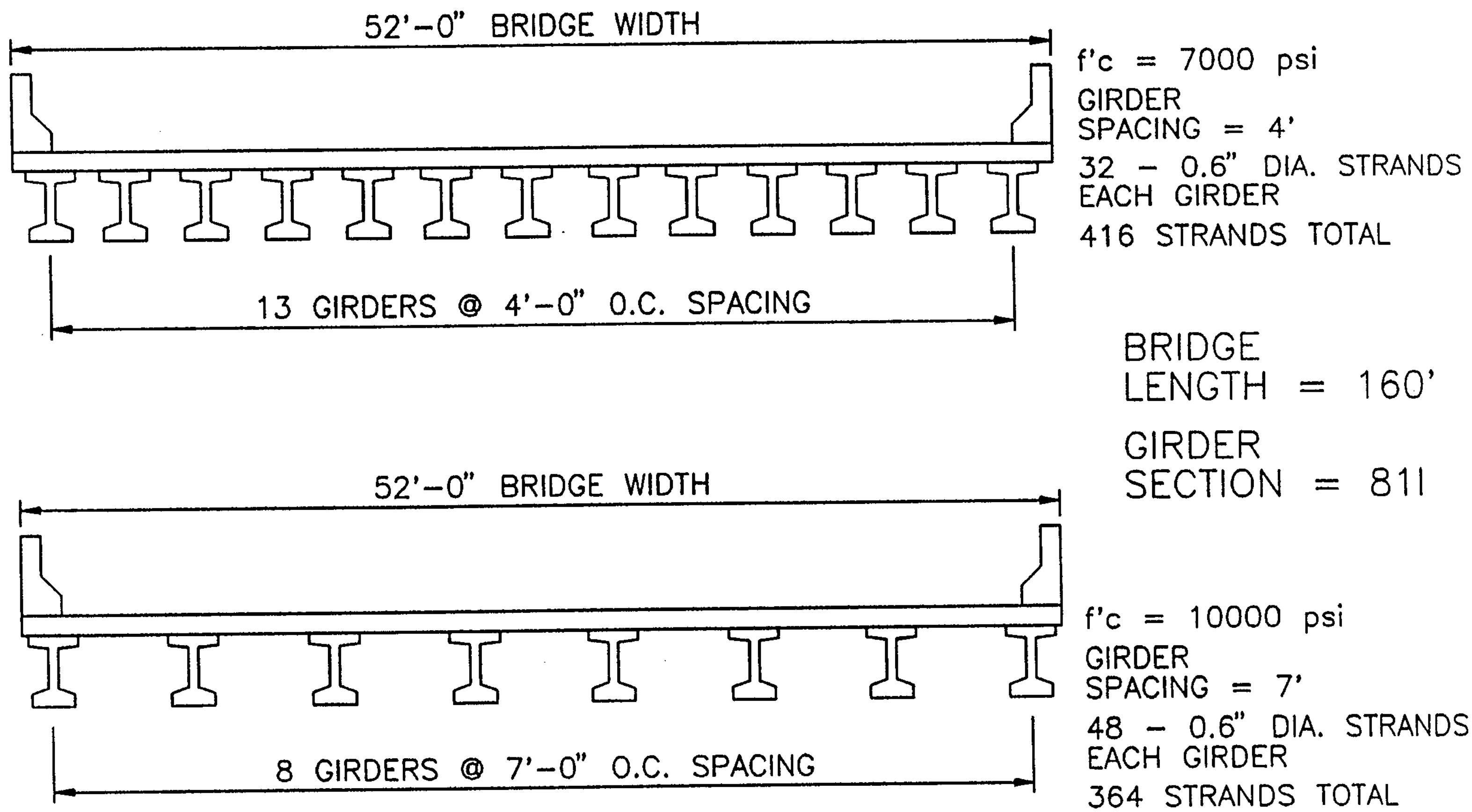
Figures



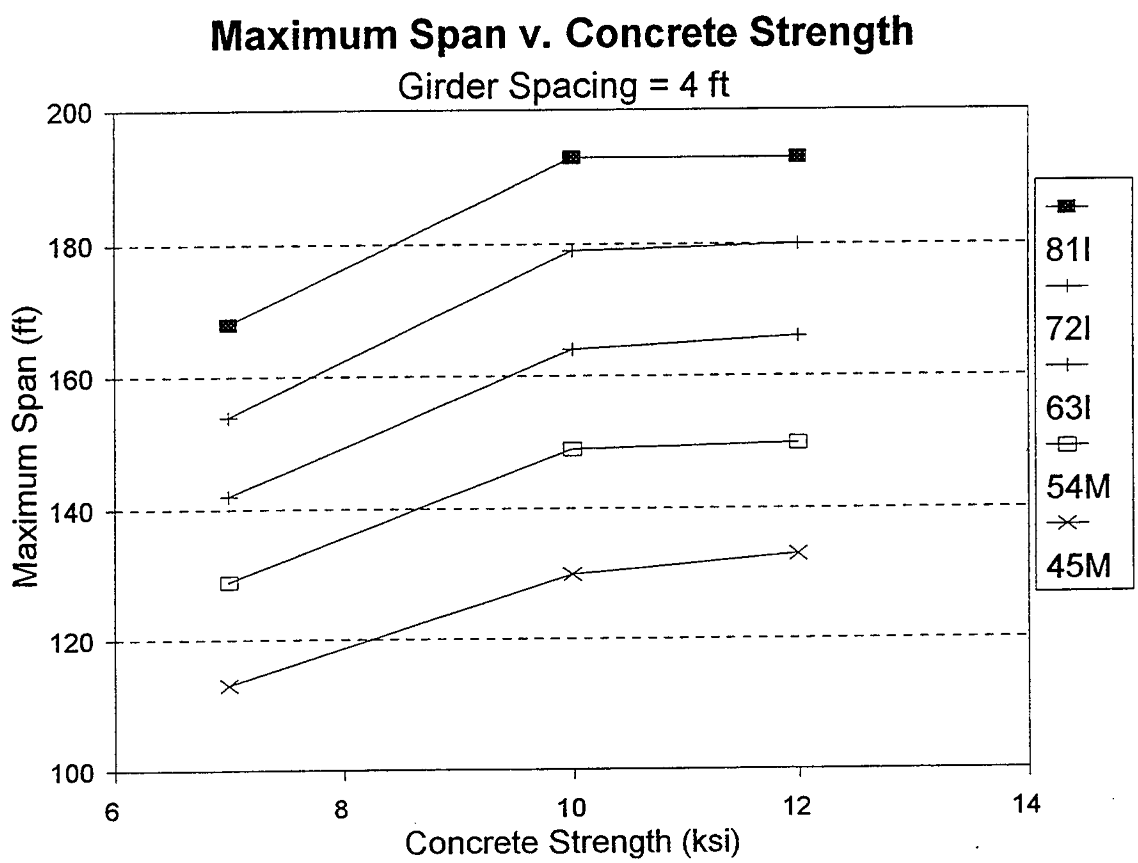
Effect of Transverse Girder Spacing
Figure 1.1



Effect of Concrete Strength
Figure 1.2

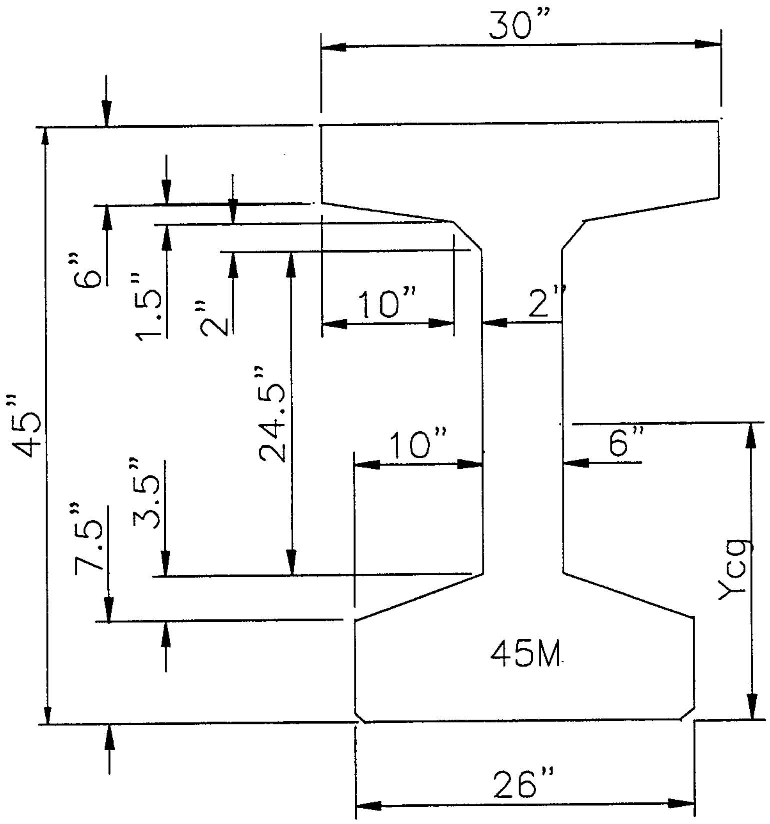


Effect of Concrete Strength on Bridge Design
Figure 1.3



Girder Span versus Concrete Strength
Figure 1.4

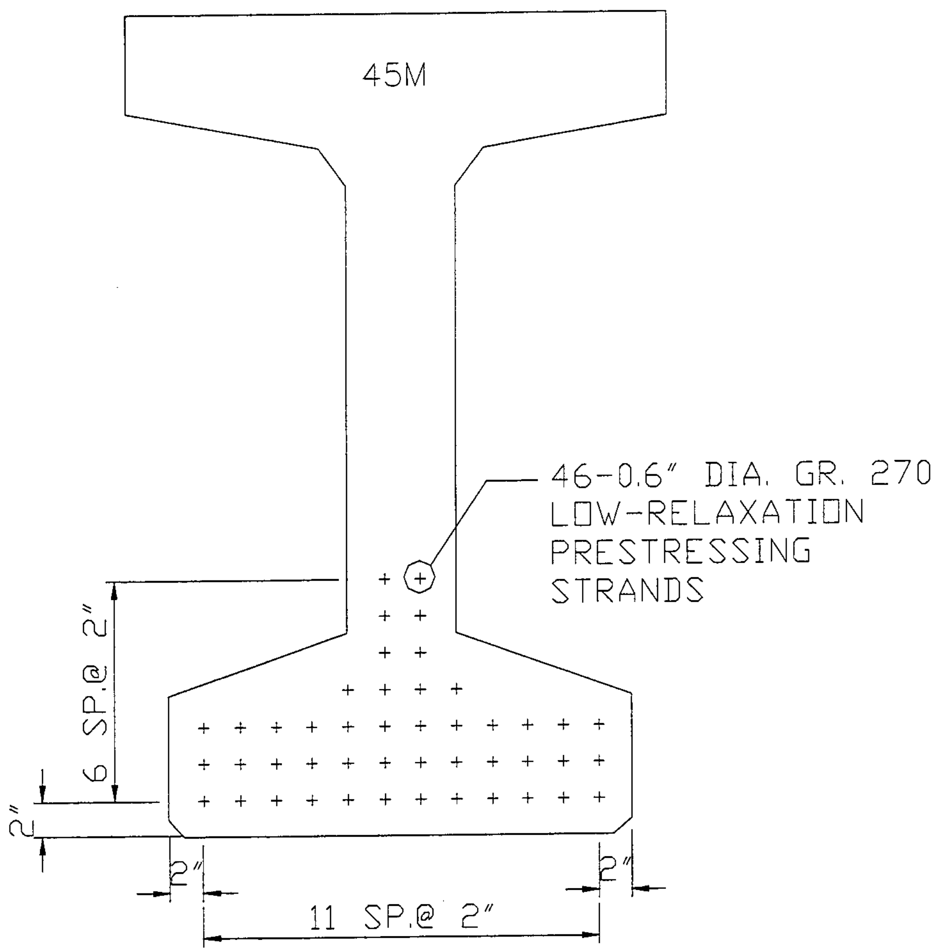
45M I-GIRDER CROSS SECTION DIMENSIONS

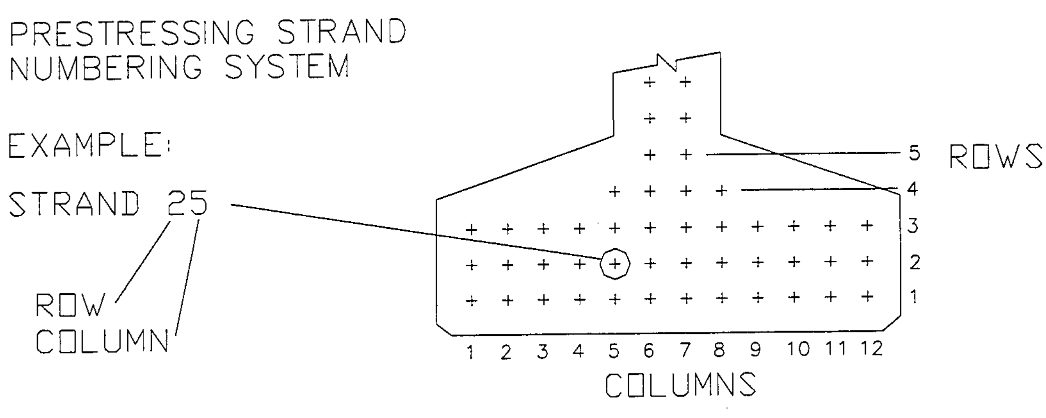


GEOMETRIC PROPERTIES

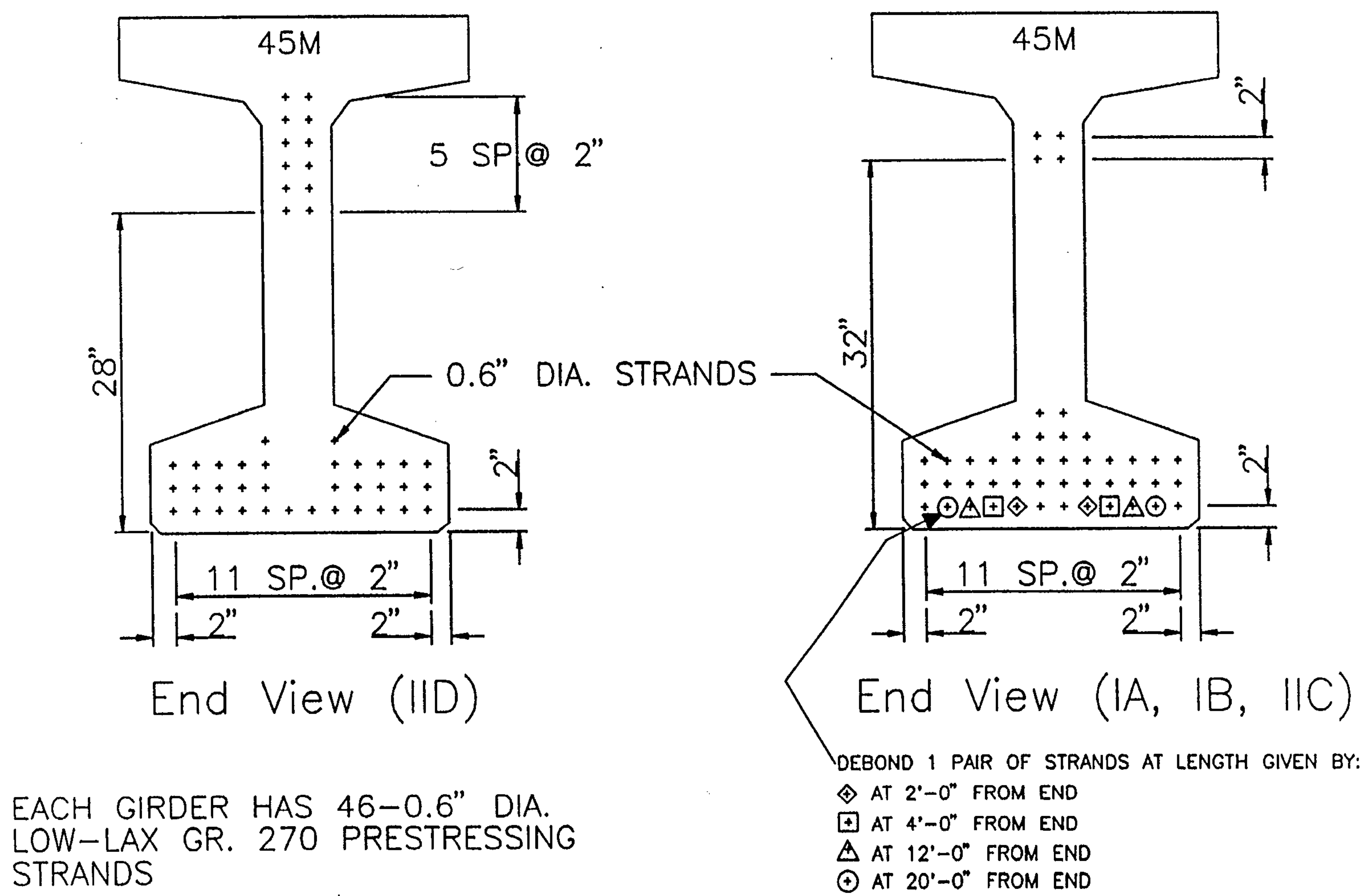
	MnDOT [7]	GROSS CONCRETE SECTION ONLY
A =	624 in^2	623 in^2
I =	167,048 in^4	166,600 in^4
Ycg =	22.34 in	22.38 in

Test Girder Cross Section
Figure 2.1

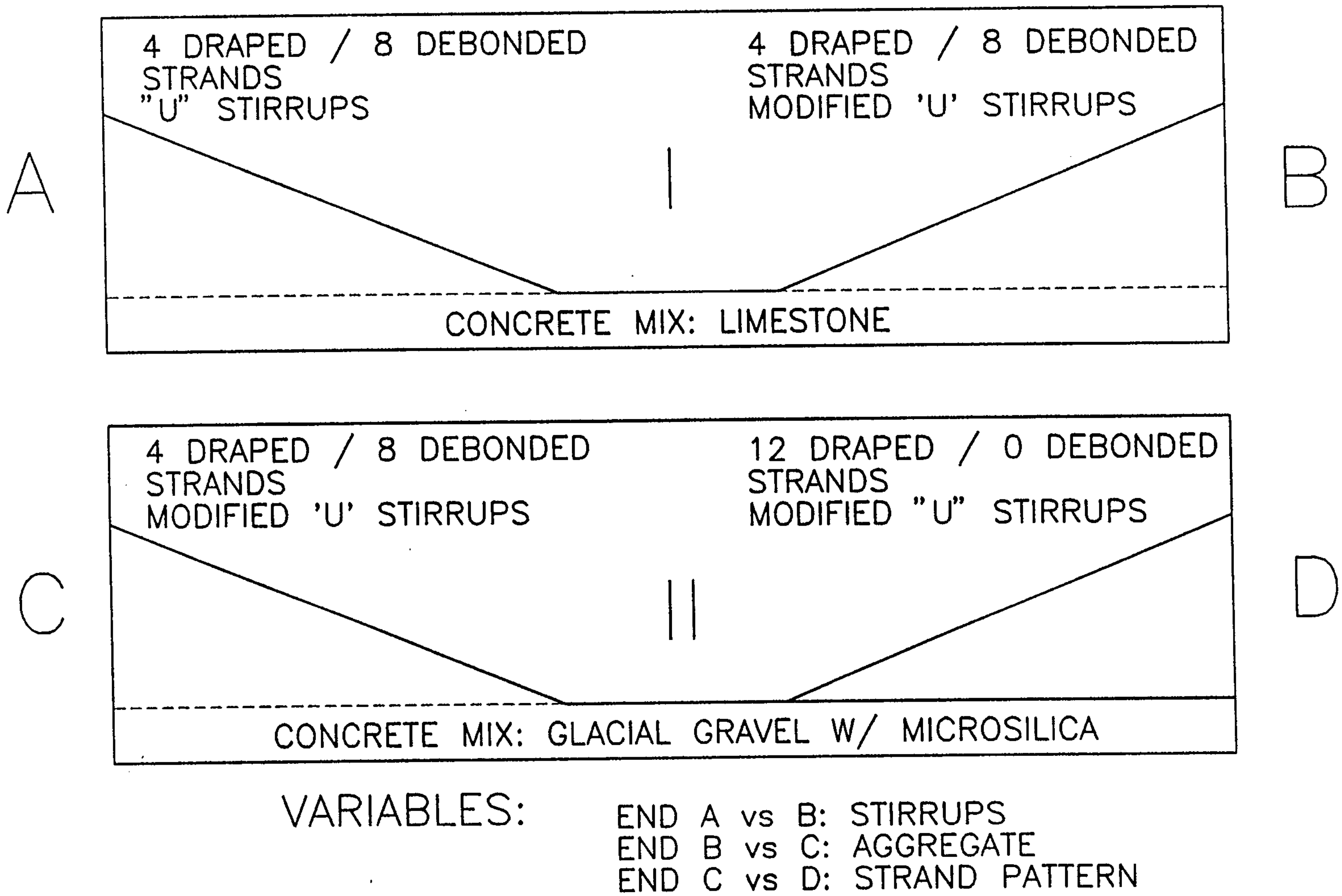




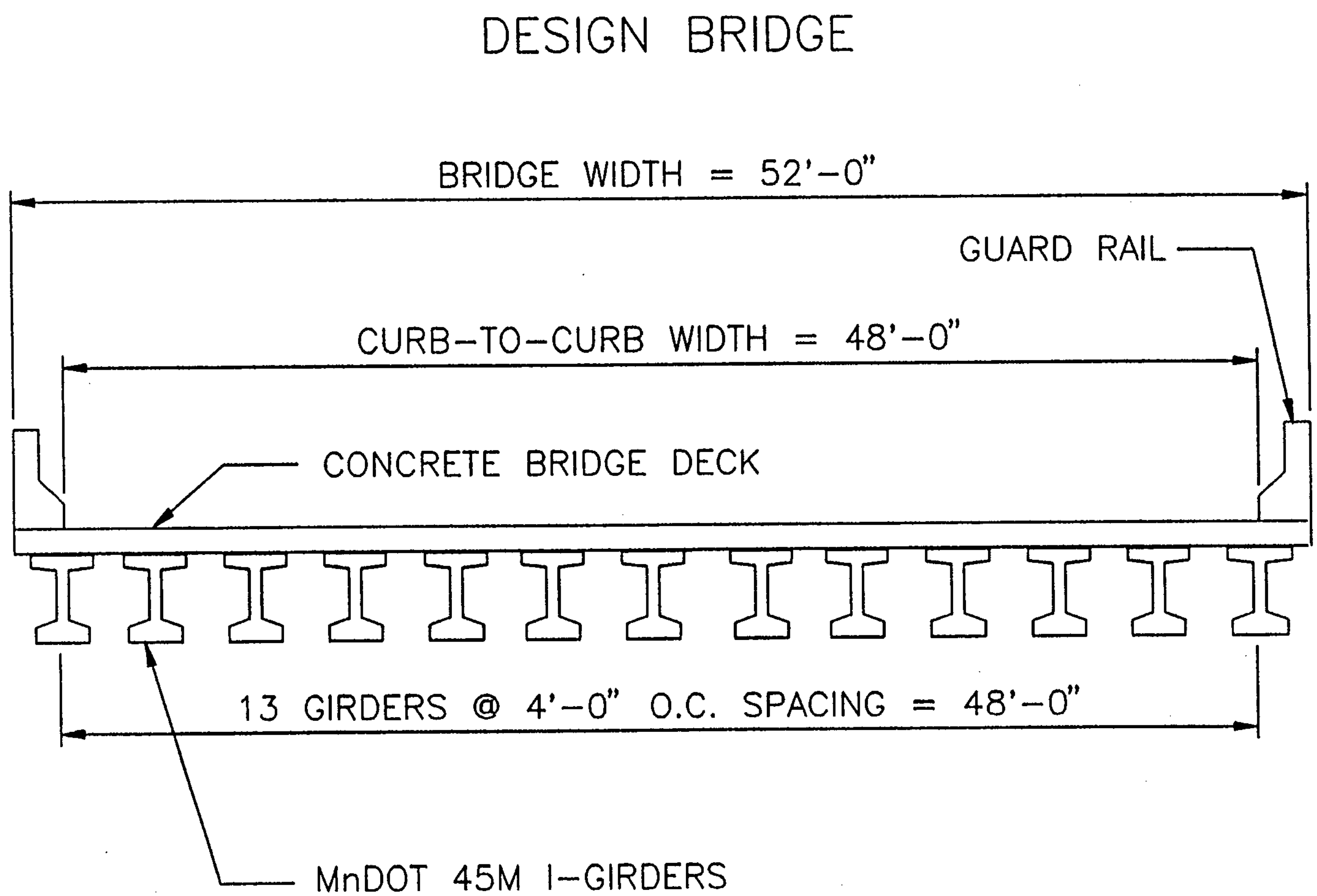
Test Girder Strand Pattern at Midspan
Figure 2.2



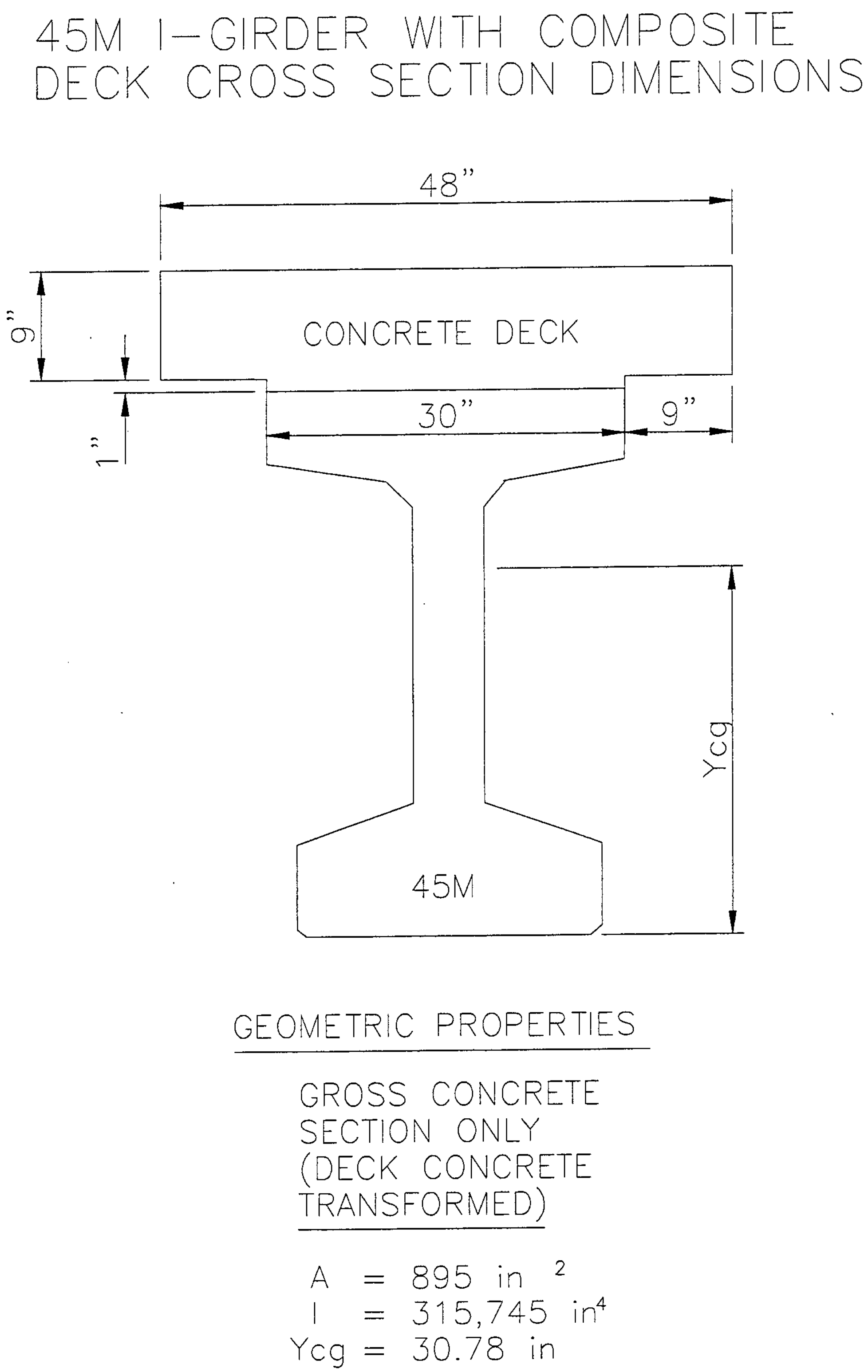
Test Girder Strand Pattern at Ends
Figure 2.3



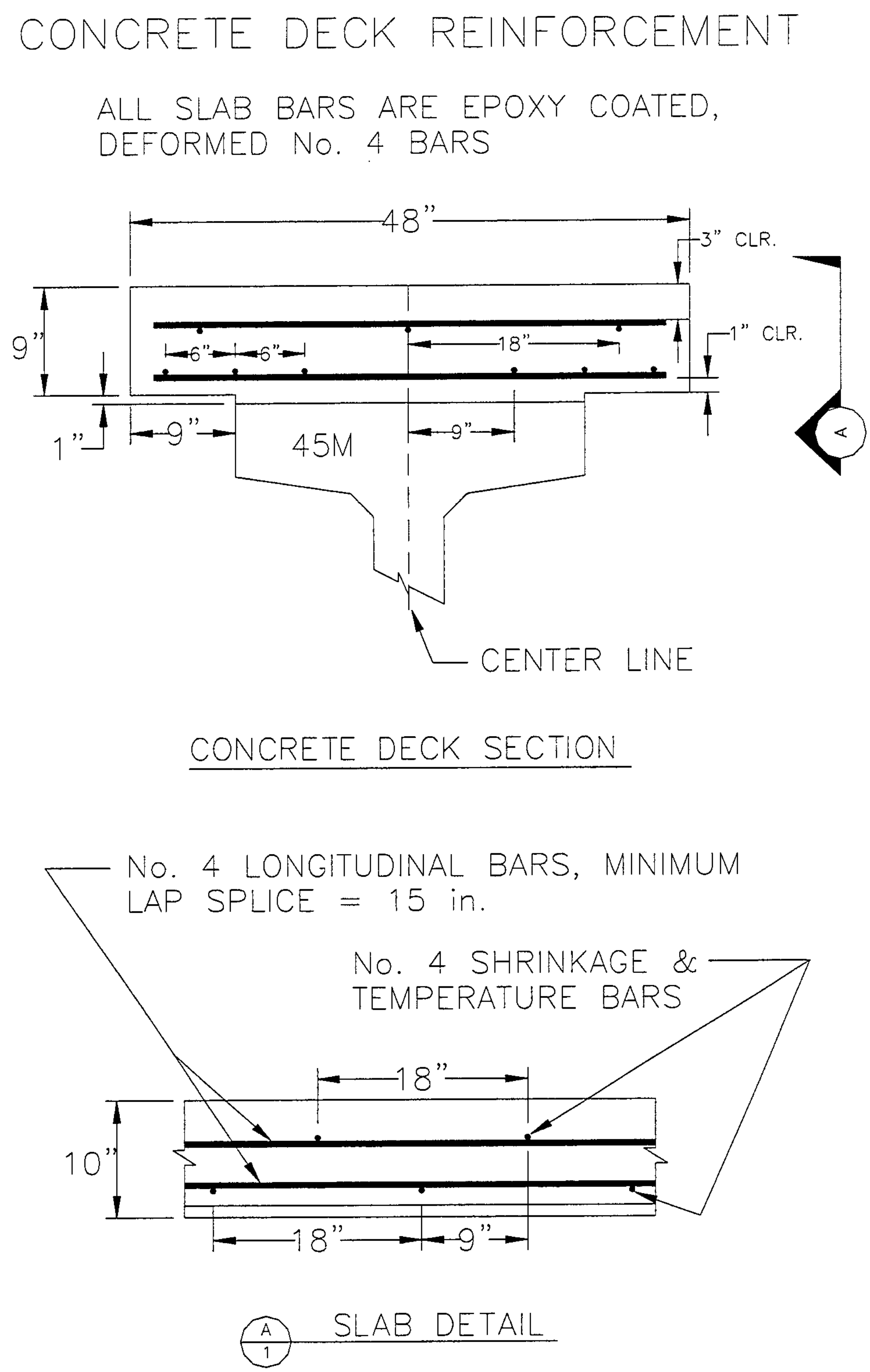
Test Girder Variables
Figure 2.4



Design Bridge
Figure 2.5

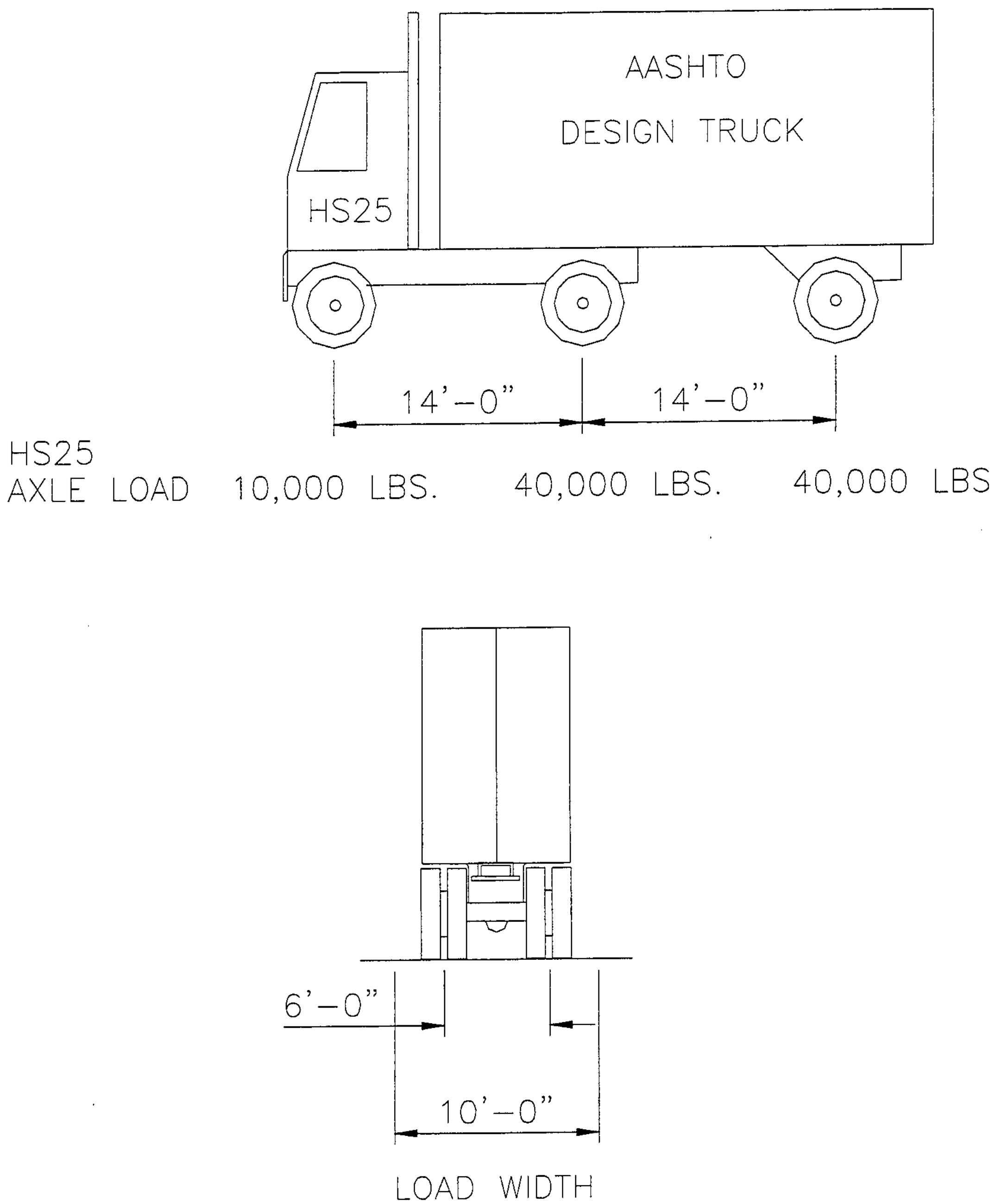


Composite Test Girder Cross Section
Figure 2.6



Concrete Deck Reinforcement
Figure 2.7

DESIGN LIVE LOAD



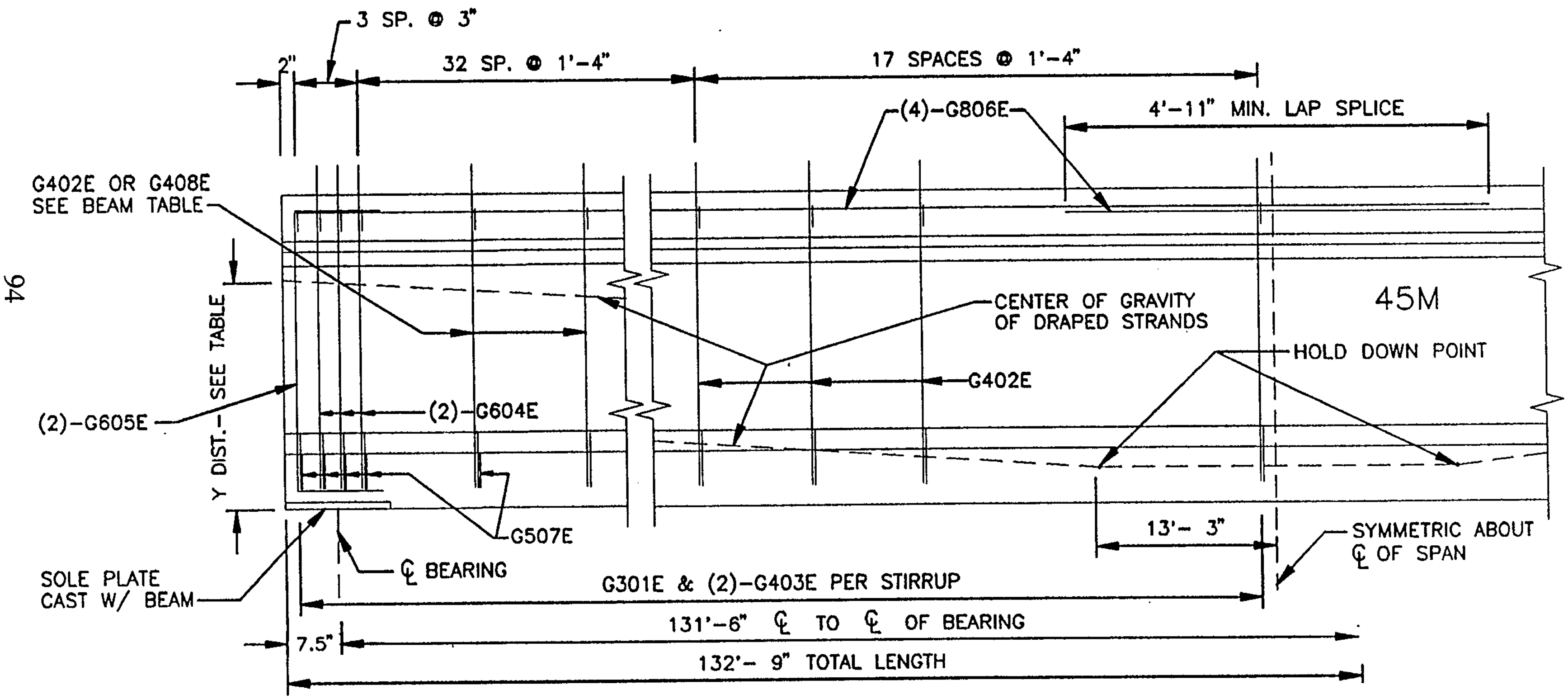
Design Live Load
Figure 2.8

BEAM CONFIGURATION TABLE

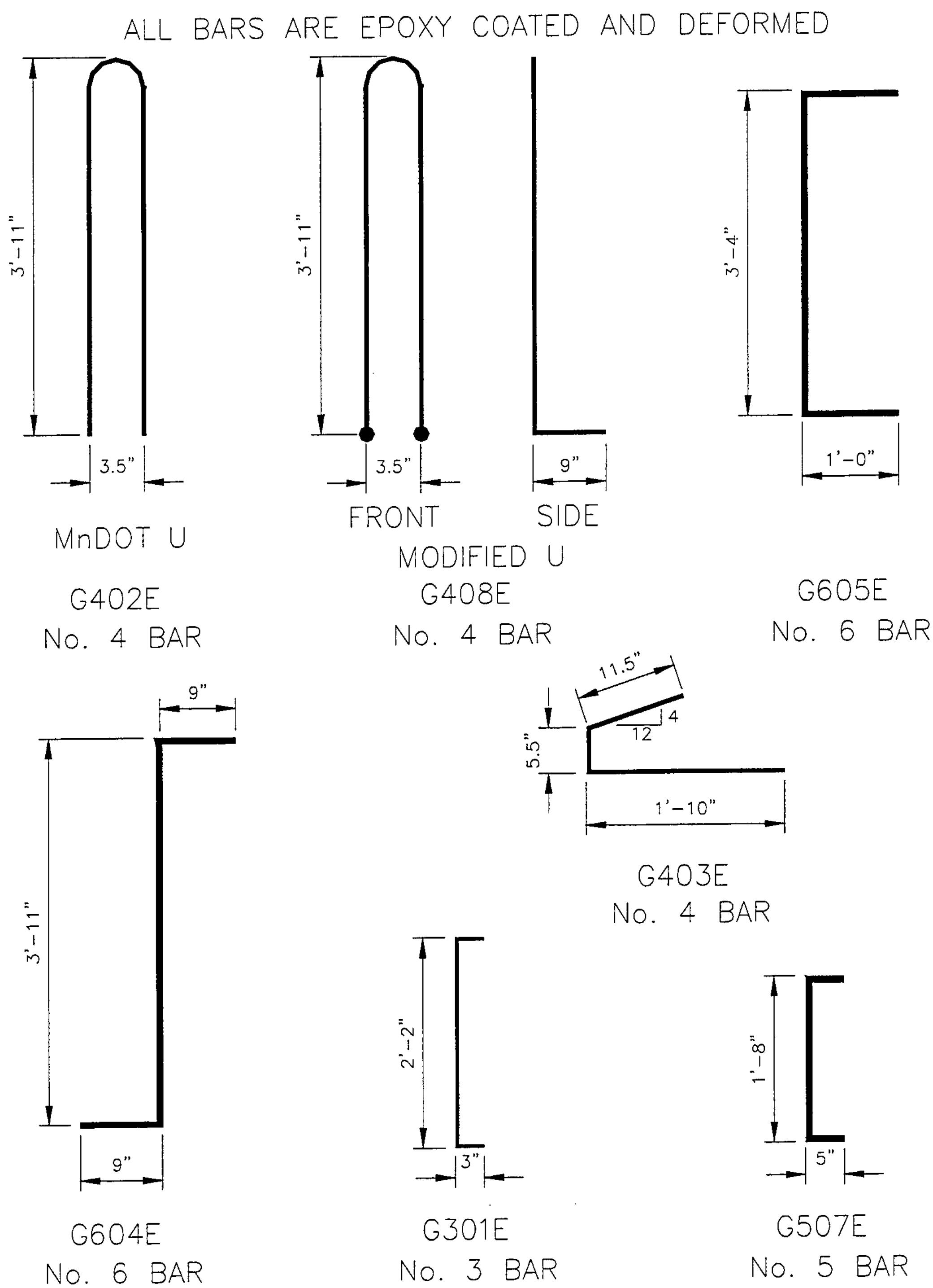
BEAM	END	AGGREGATE FOR MIX	NO. STRANDS DRAPED	NO. STRANDS DEBONDED	END REGION STIRRUP CONFIGURATION
I	SOUTH	LIMESTONE	4	8	G402E
I	SOUTH	LIMESTONE	4	8	G408E
II	NORTH	GG w/ MS	4	8	G408E
II	NORTH	GG w/ MS	12	0	G408E

Y DISTANCES (in inches)

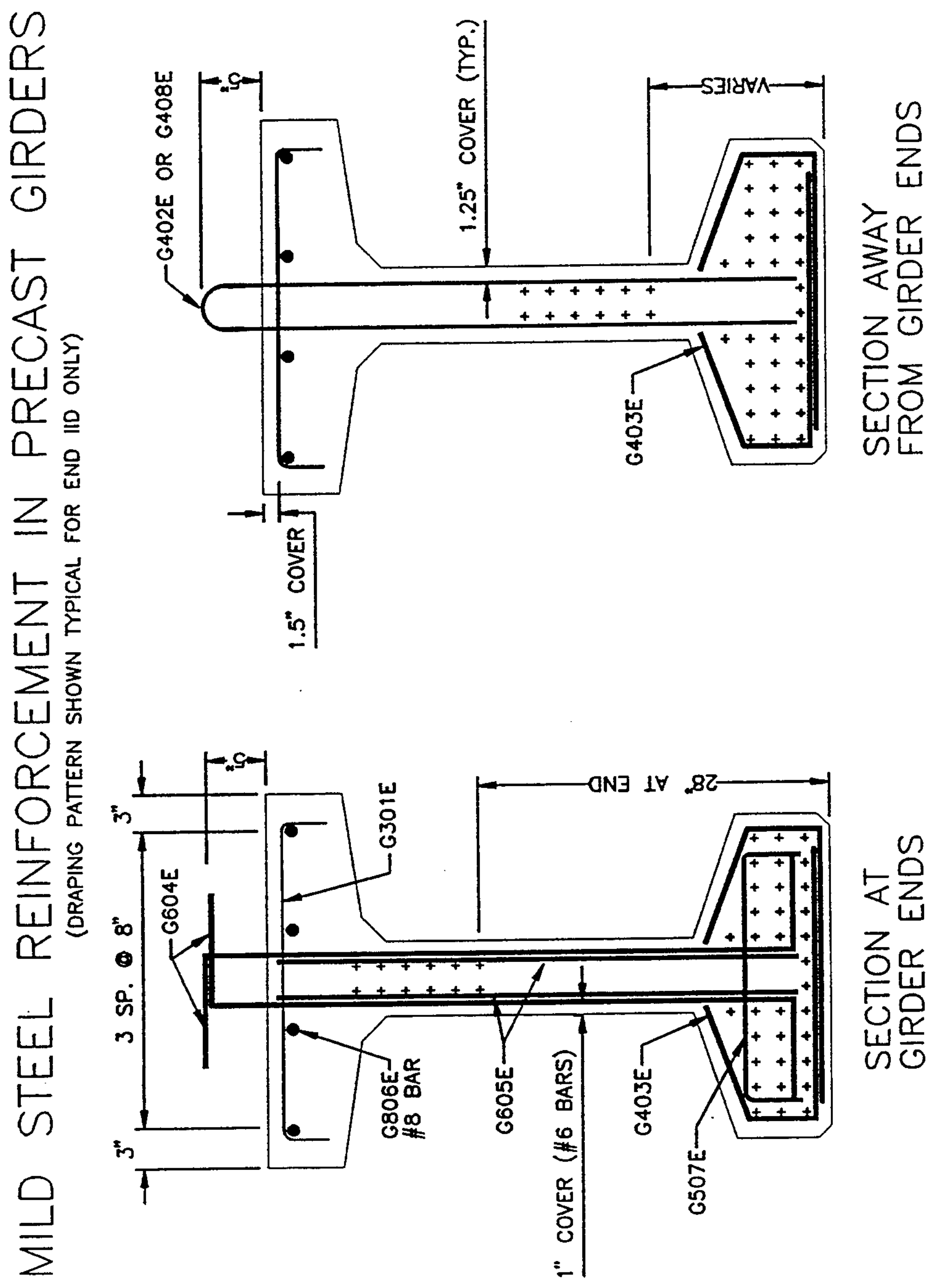
	NO.	SPAN	END
DRAPED SECTION:			
STRAIGHT STRANDS	34	4.12"	
DRAPED STRANDS	12	9.00"	33.00"
TOTAL	46	5.39"	11.65"
DEBONDED SECTION:			
STRAIGHT STRANDS	42	4.67"	5.29"
DRAPED STRANDS	4	13.00"	33.00"
TOTAL	46	5.39"	



Test Girder Reinforcement, Elevation
Figure 2.9



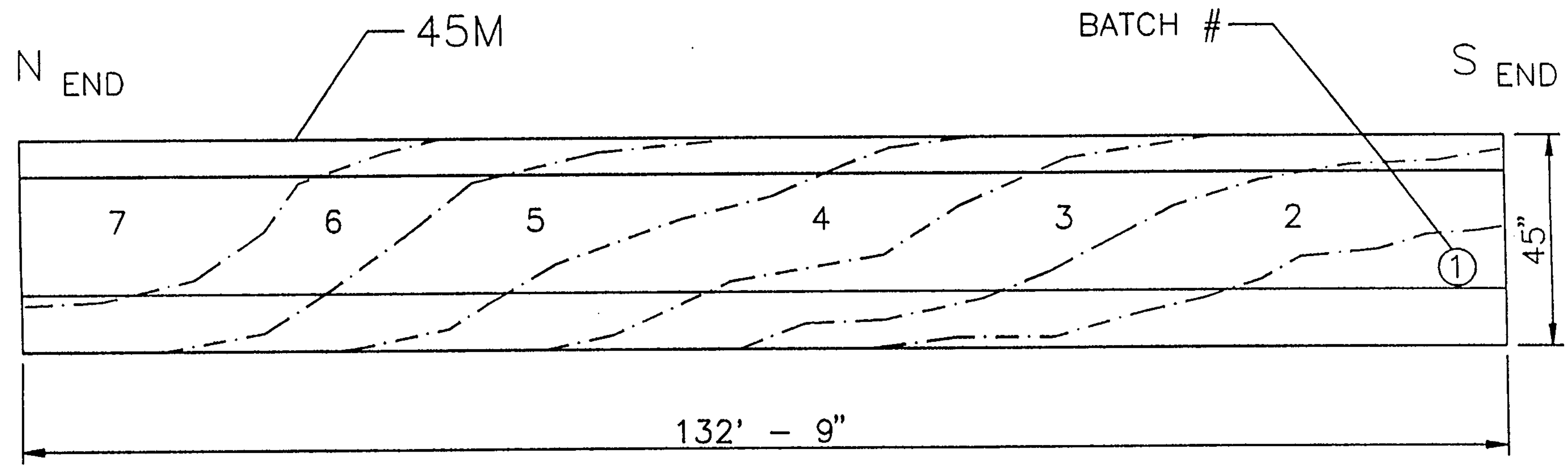
Precast Test Girder Reinforcement
Figure 2.10



Test Girder Reinforcement, Sections
Figure 2.11

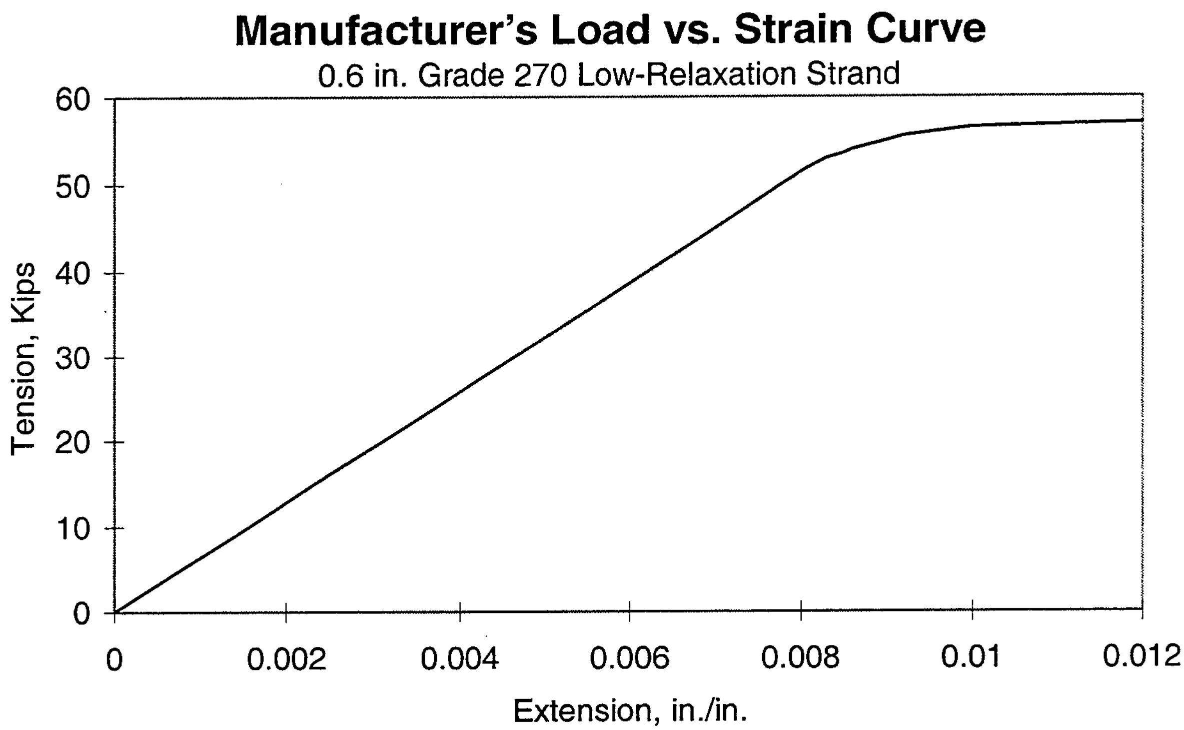
CASTING SEQUENCE AND APPROXIMATE CONCRETE BATCH LOCATIONS FOR THE TEST GIRDERS

APPROXIMATELY 3 CUBIC YARDS PER CONCRETE BATCH



ELEVATION OF TEST GIRDER. TYP.

Concrete Casting Sequence and Batch Locations
Figure 3.1

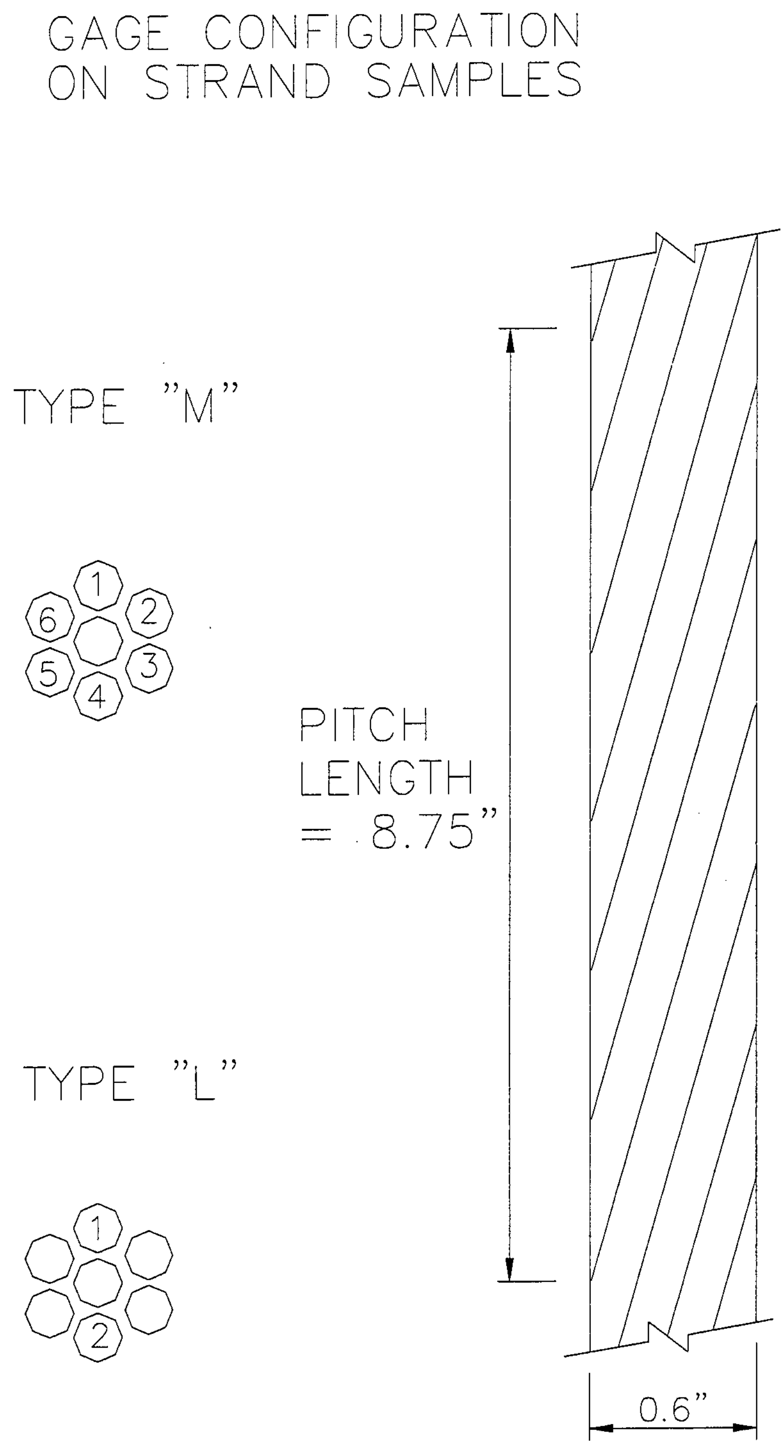


in. Diameter, Grade 270
ASTM A416 Strand Low-Relaxation

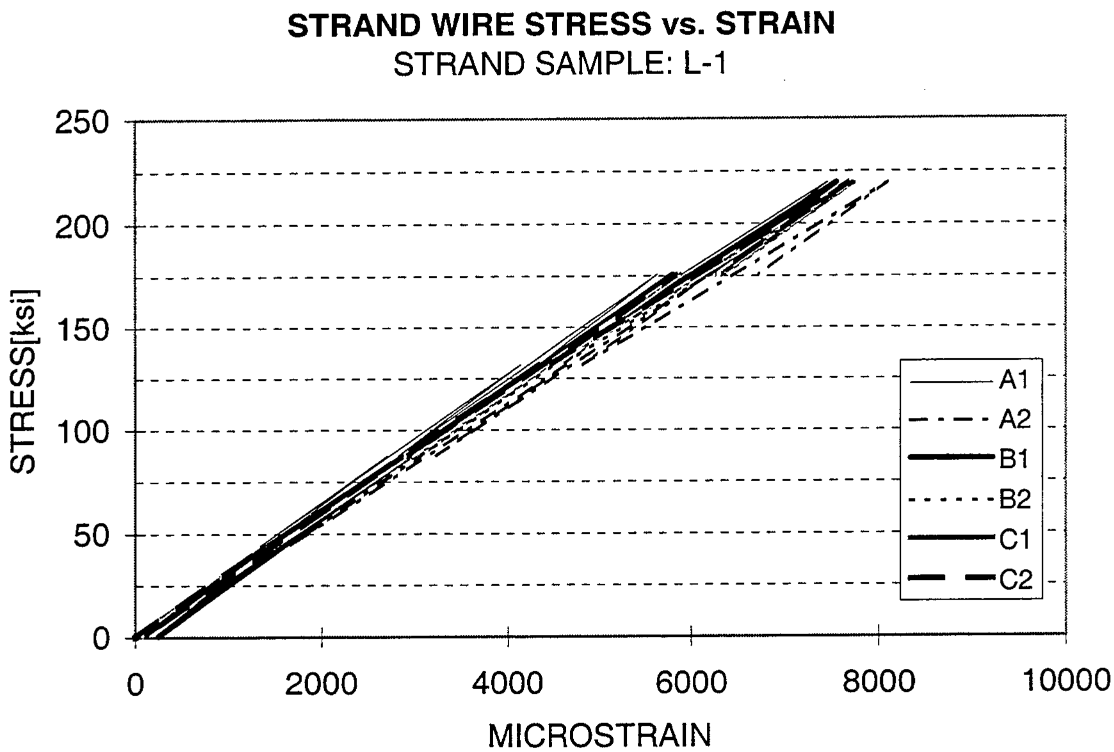
Yield Strength at 1% Extension:	56,400 lb.
Ultimate Tensile Strength:	60,880 lb.
Ultimate Elongation in 24 in.:	6.67%
Average Modulus of Elasticity:	28,800 psi
Metallic Area:	0.228 sq. In.
Length of Lay:	8.50 in. = 14.15 D

Union Wire Rope
Kansas City, Missouri

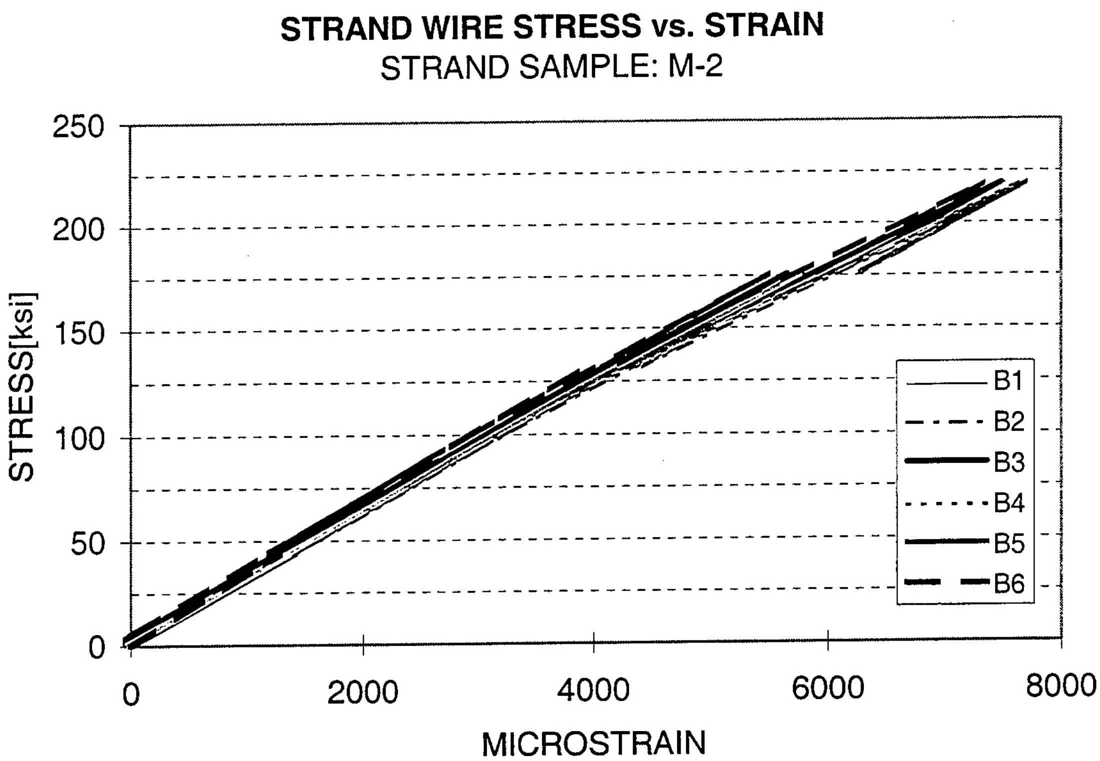
Manufacturer's Load versus Strain Curve for Strand
Figure 3.2



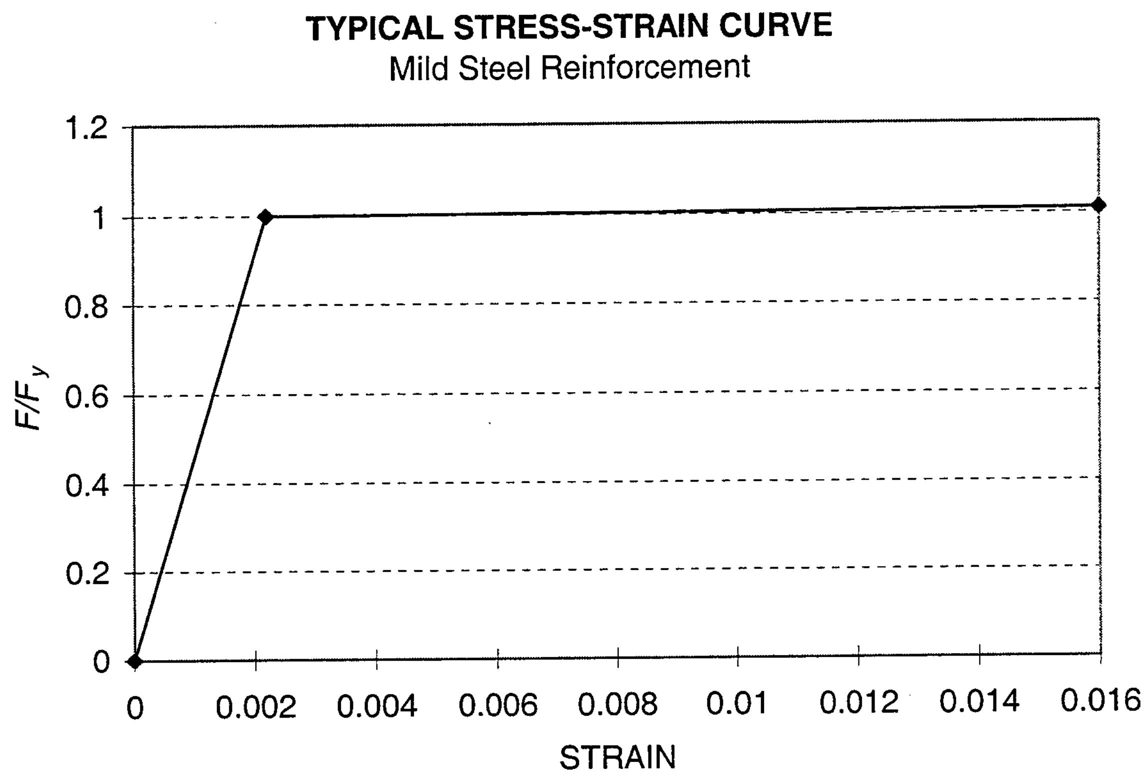
Strain Gage Configuration on Strand Samples
Figure 3.3



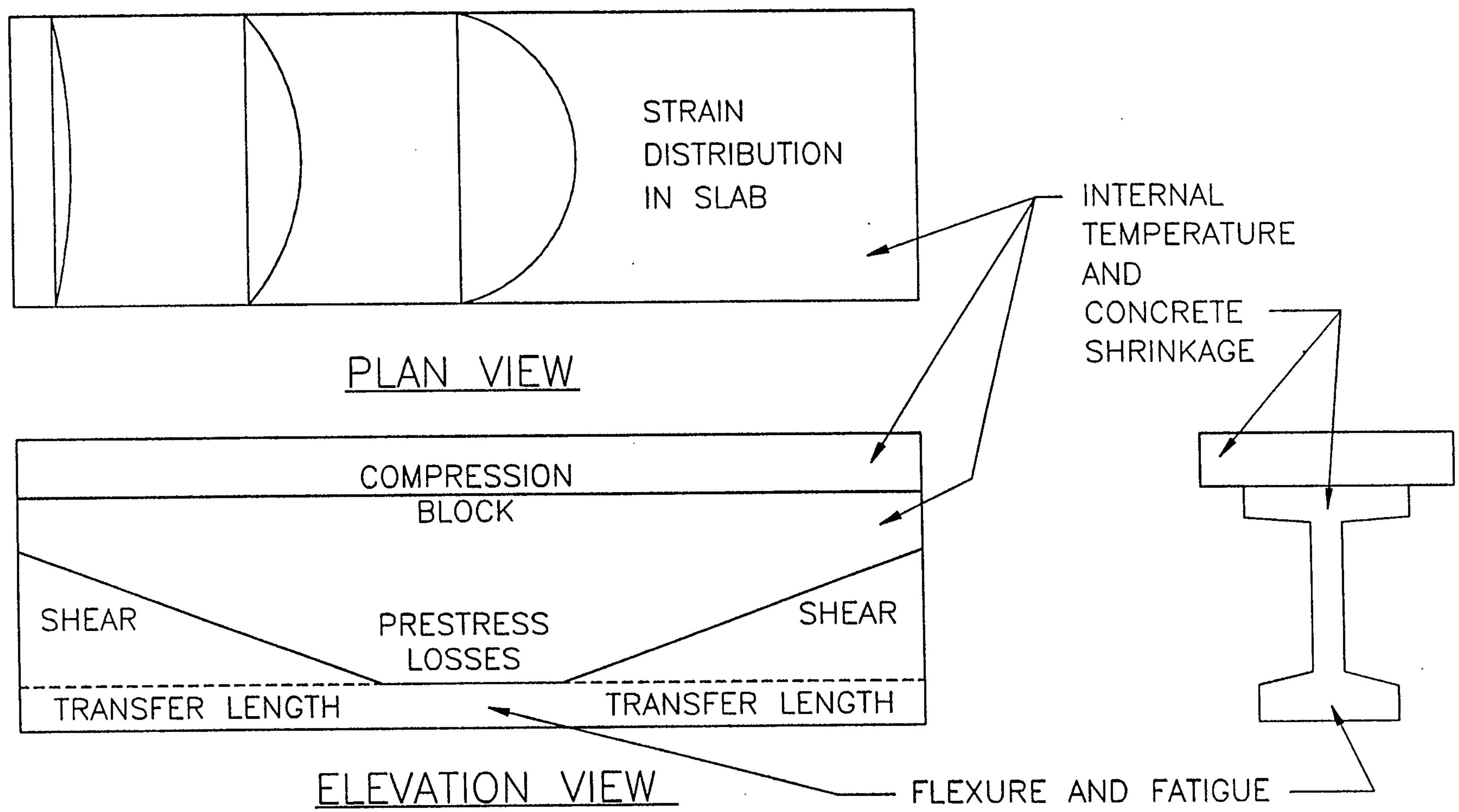
Experimental Stress versus Strain Curve for Strand (Type L)
Figure 3.4



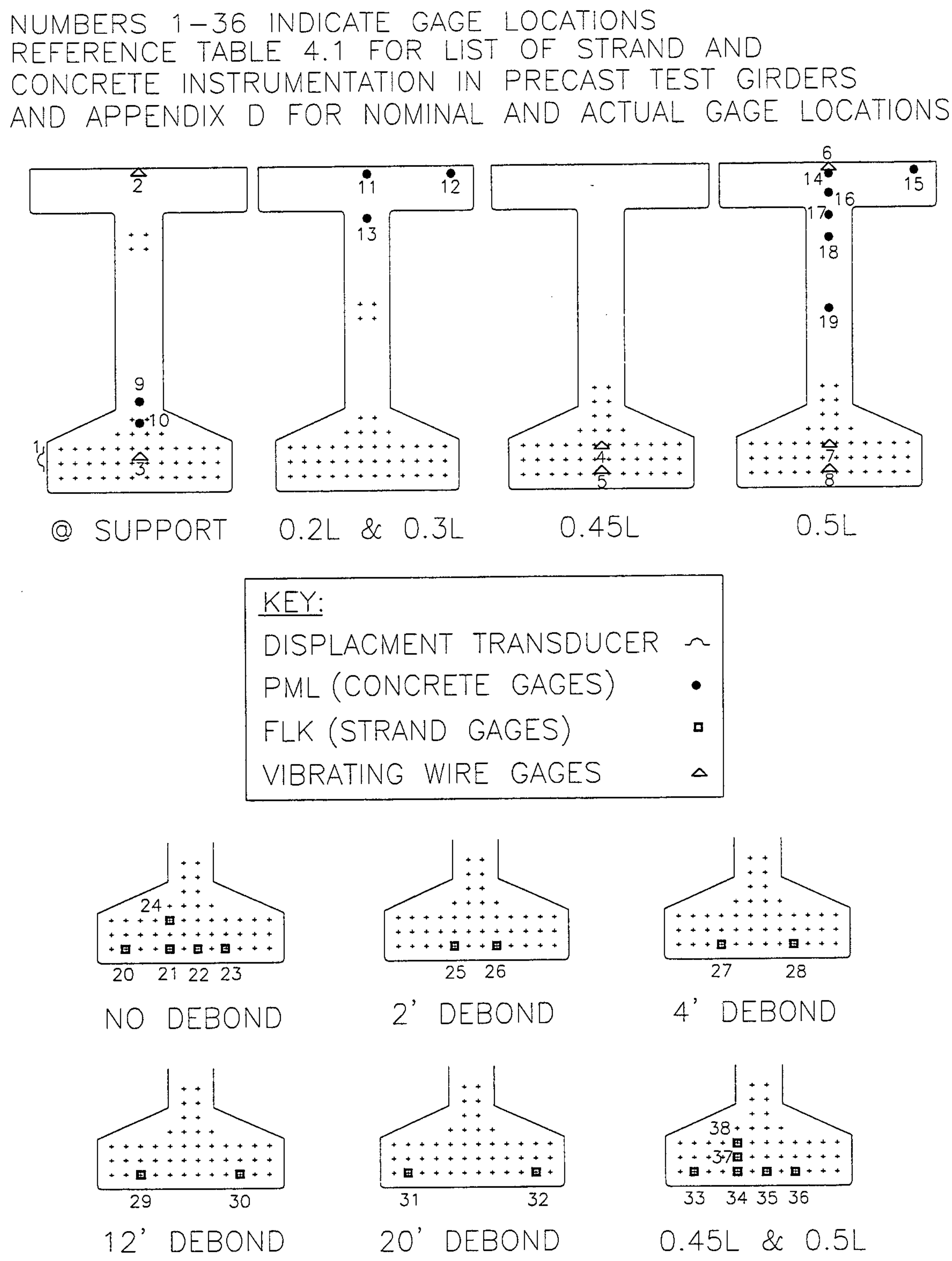
Experimental Stress versus Strain Curve for Strand (Type M)
Figure 3.5



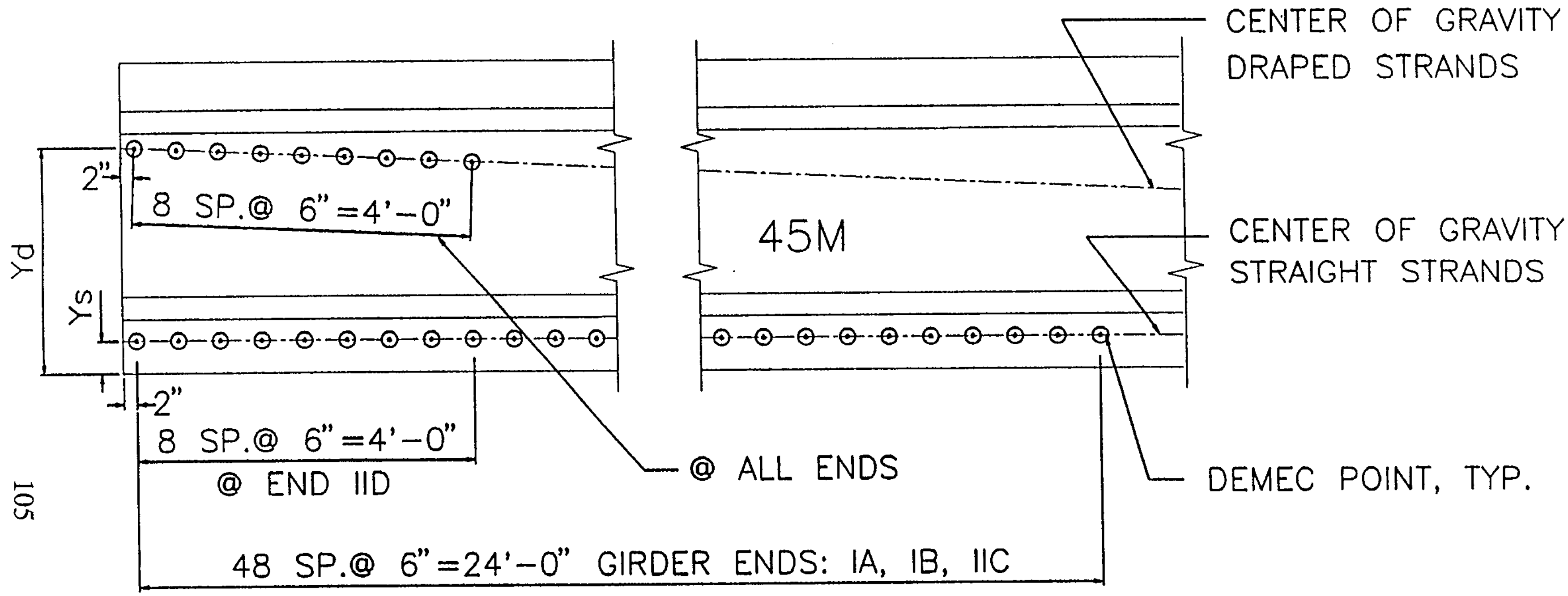
Typical Stress versus Strain Curve for Mild Steel Reinforcement
Figure 3.6



Instrumentation Scheme
for Testing Program
Figure 4.1



Strand and Concrete Instrumentation in Precast Girders
Figure 4.2



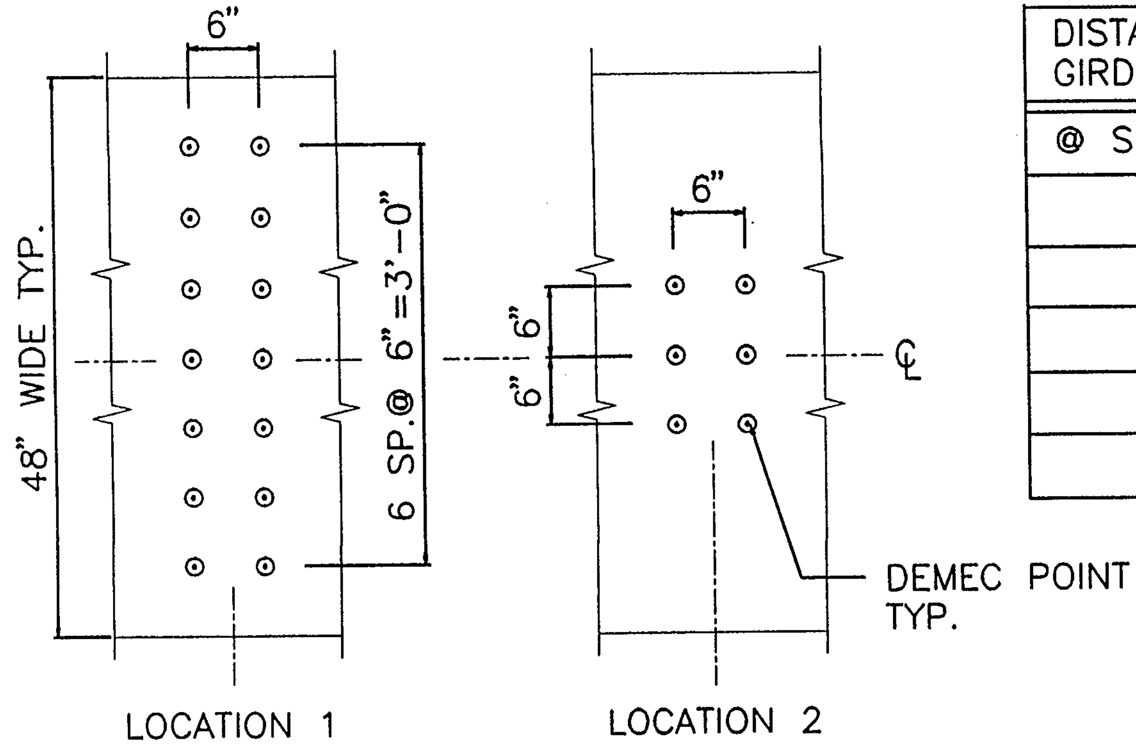
ELEVATION OF TEST GIRDER END

Yd = HEIGHT OF DEMECS AT THE DRAPED STRANDS, ALL GIRDER ENDS = 33"

Ys = HEIGHT OF DEMECS AT THE STRAIGHT STRANDS: GIRDER END IA, IB AND IIC = 5.2"; AT GIRDER END IID = 5.3"

DEMEC Instrumentation in Precast Girders
Figure 4.3

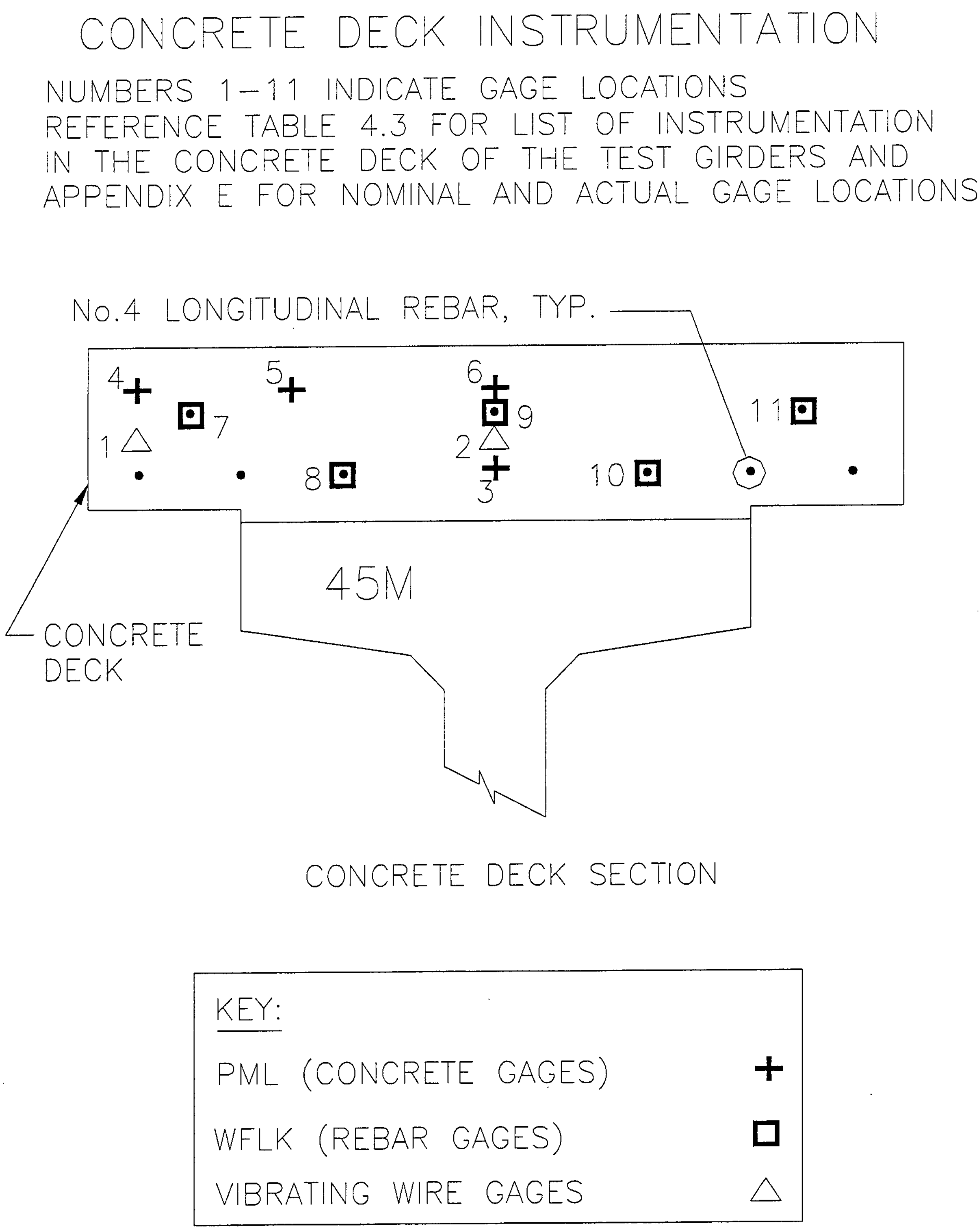
LOCATIONS OF DEMEC POINTS ON CONCRETE DECKS



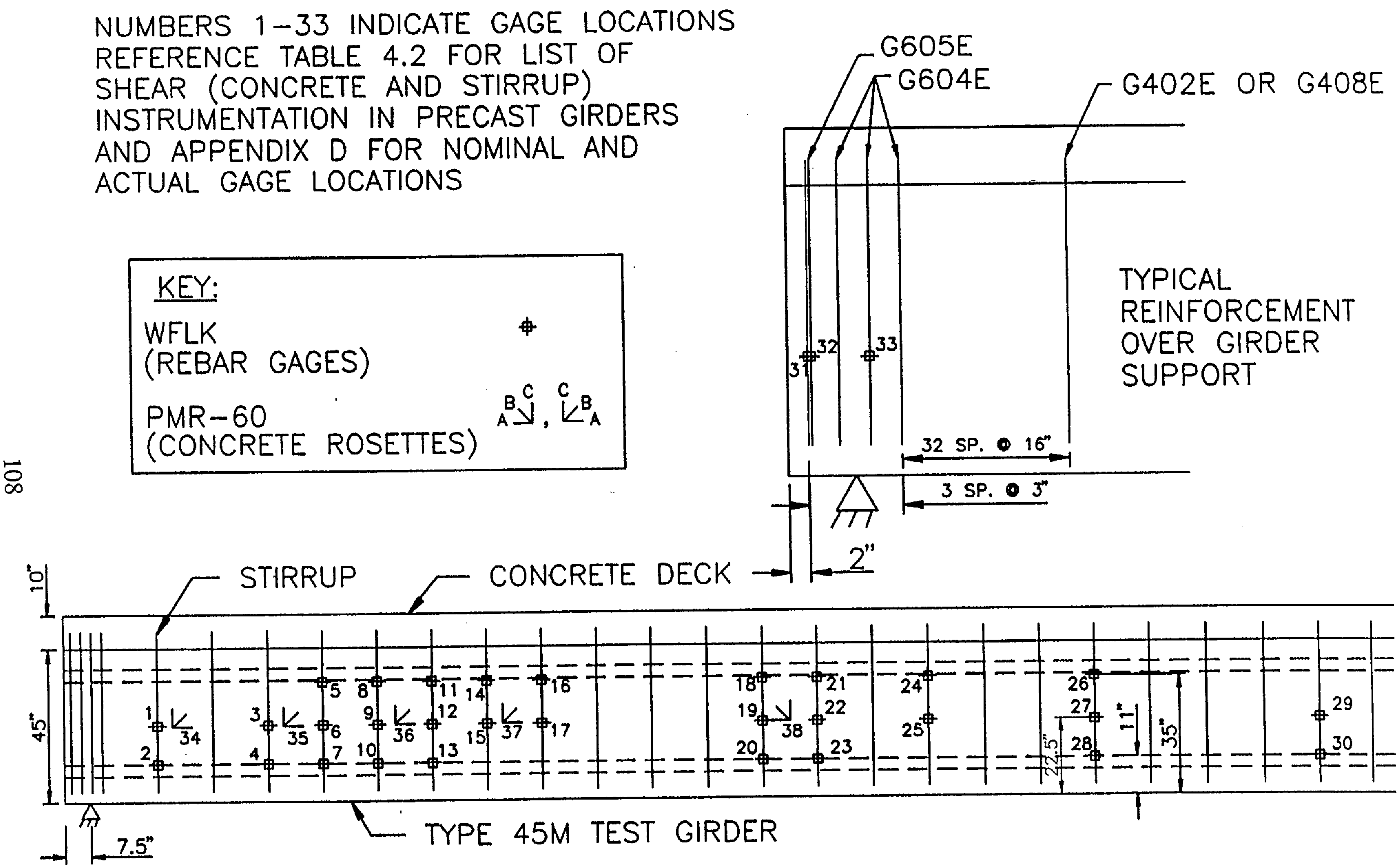
DISTANCE FROM GIRDER END	LOCATION #
@ SUPPORT	1
0.1L	2
0.2L	1
0.3L	1
0.4L	2
0.5L	1

PARTIAL PLAN OF CONCRETE DECK

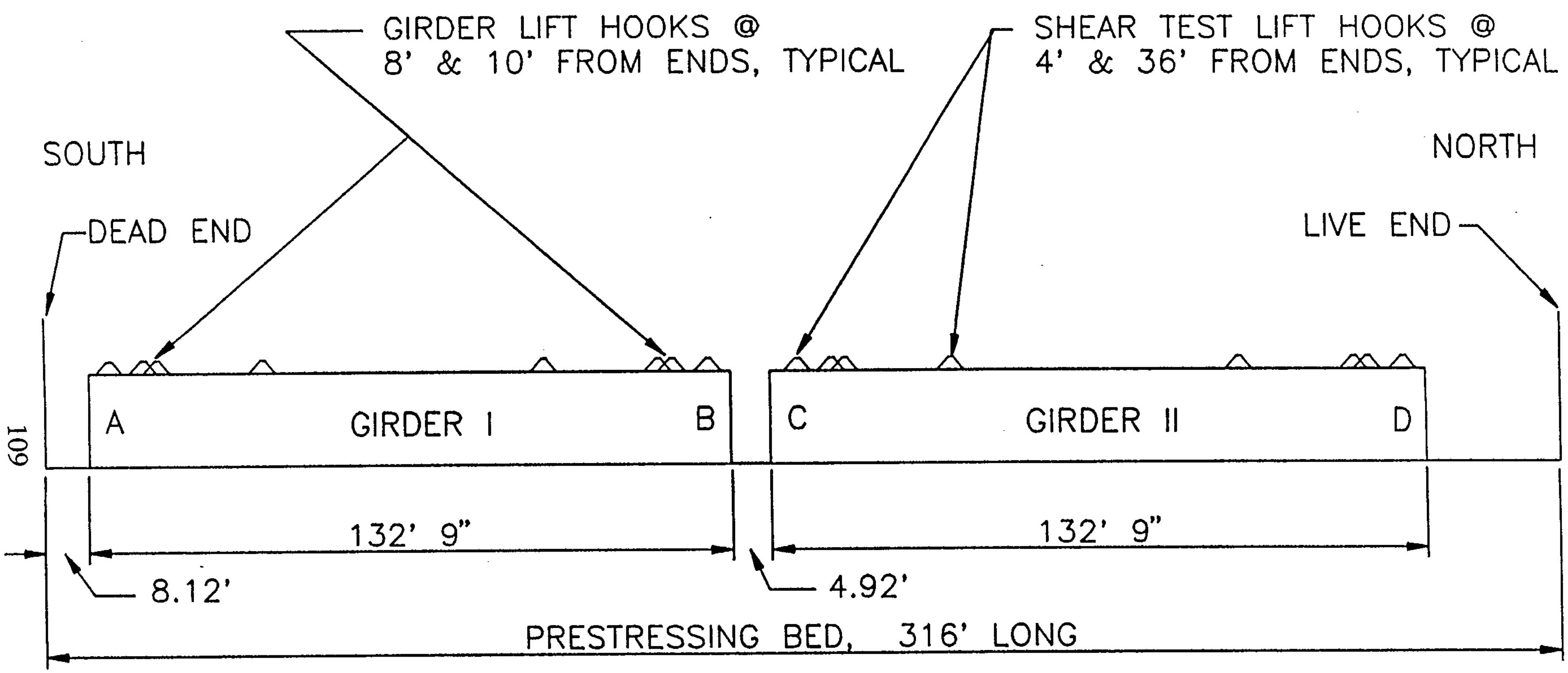
DEMEC Instrumentation on Concrete Decks
Figure 4.4



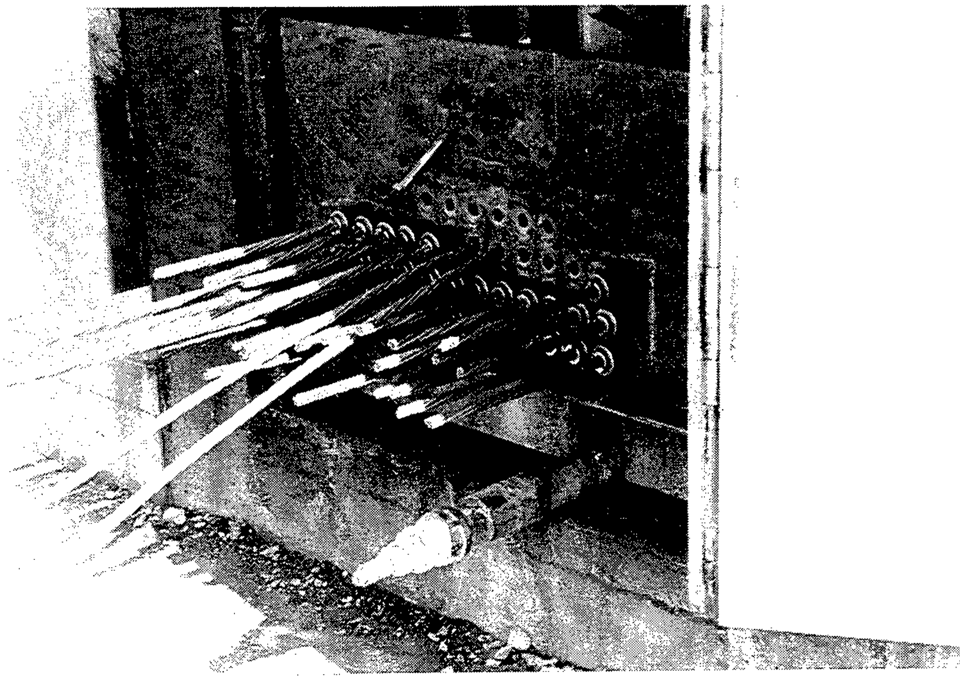
Concrete Deck Instrumentation
Figure 4.5



Shear Instrumentation in Test Girders, Elevation
Figure 4.6

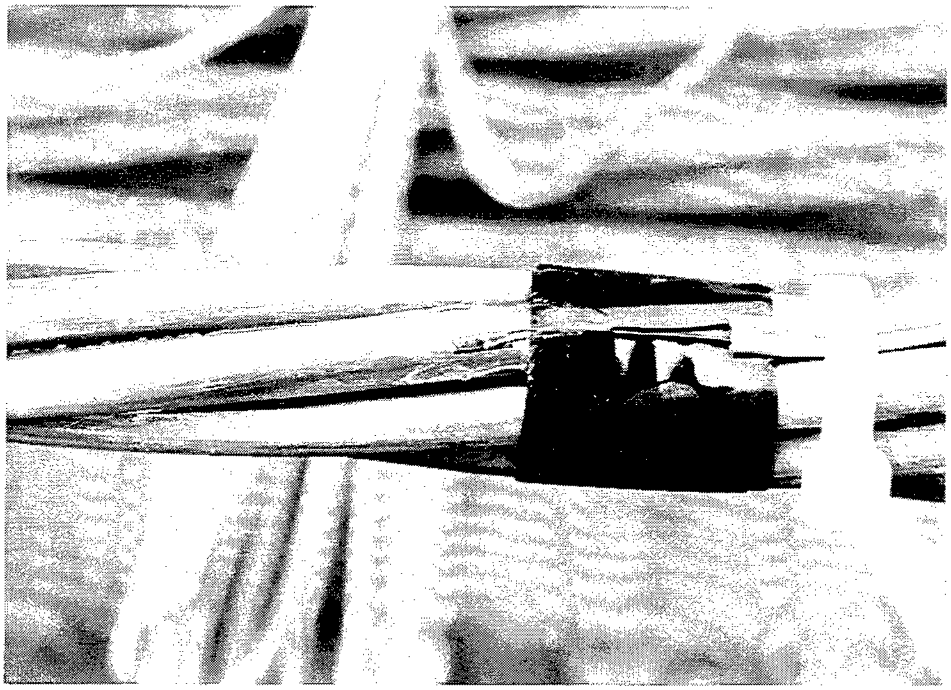


Prestressing Bed Layout for Test Girders
Figure 5.1

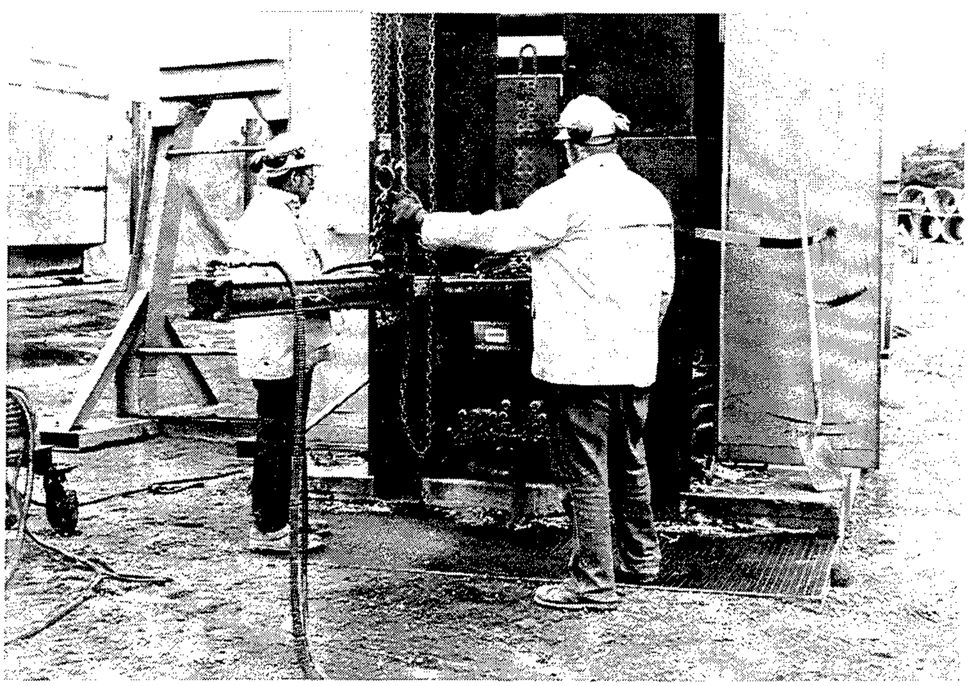


Prestressing Strand Template and End Chucks
Figure 5.2

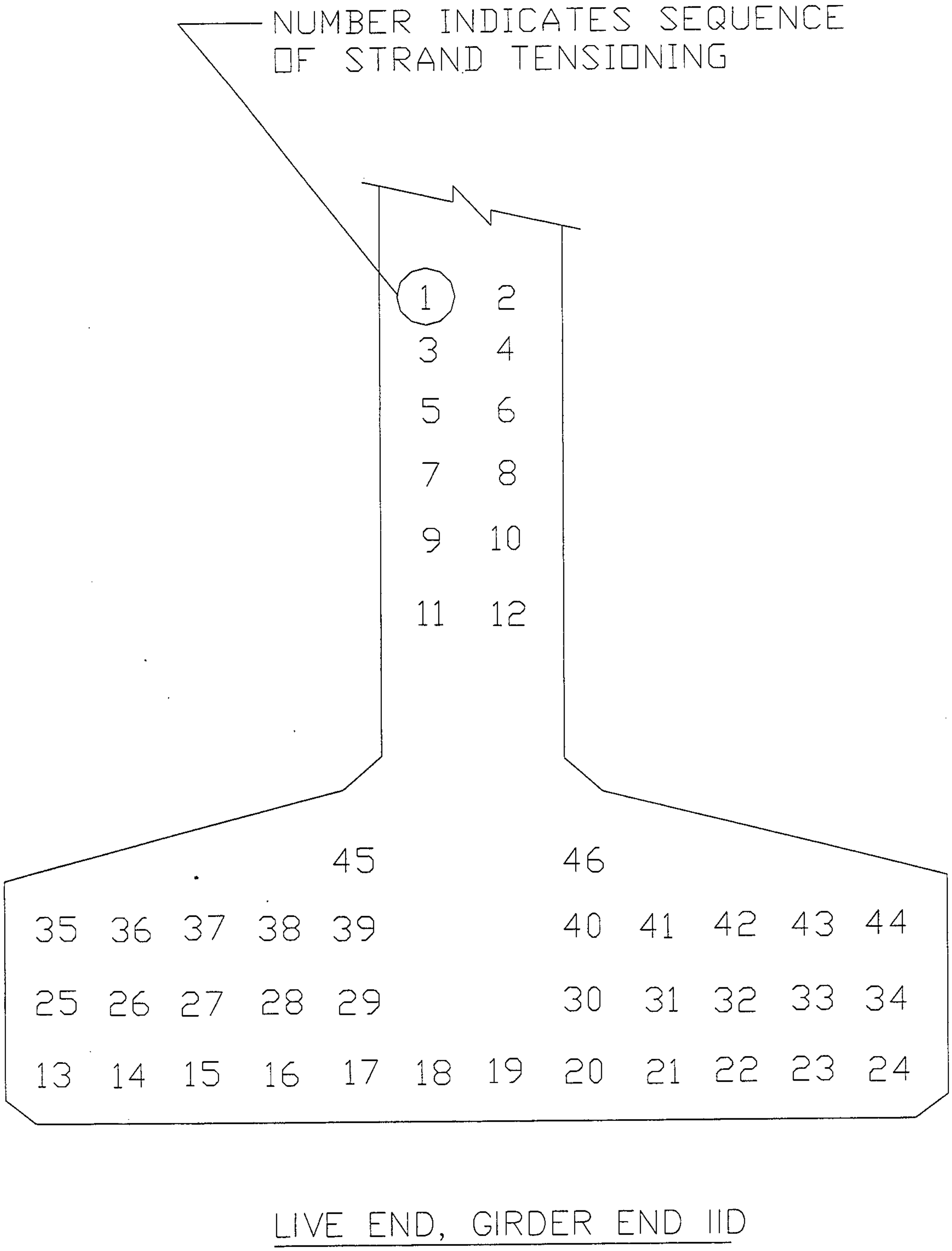
Unstressed Strands on Prestressing Bed
Figure 5.3



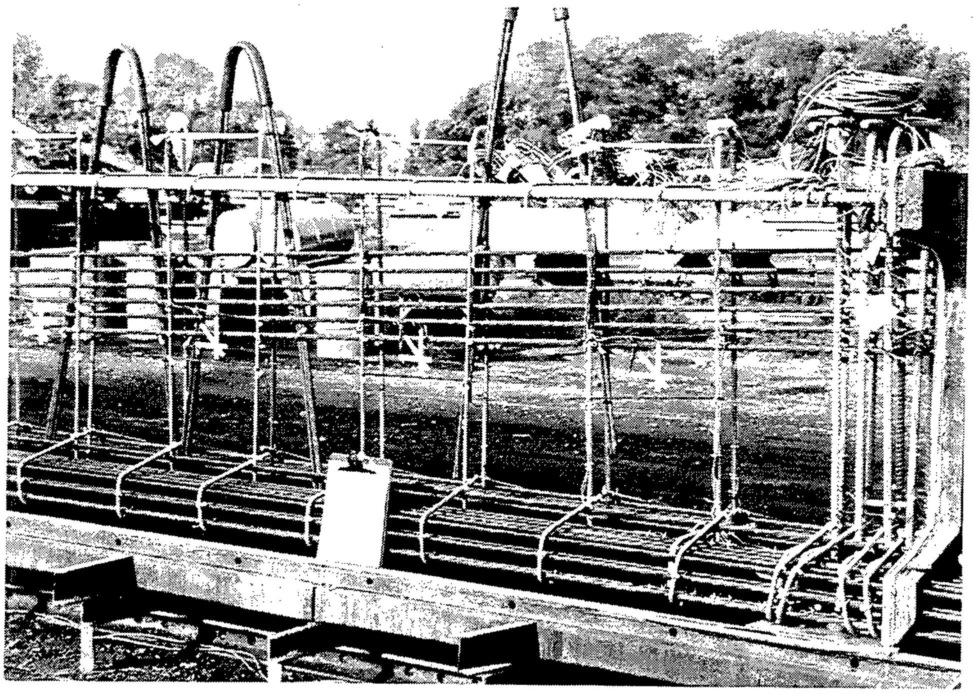
Strain Gage on Prestressing Strand
Figure 5.4



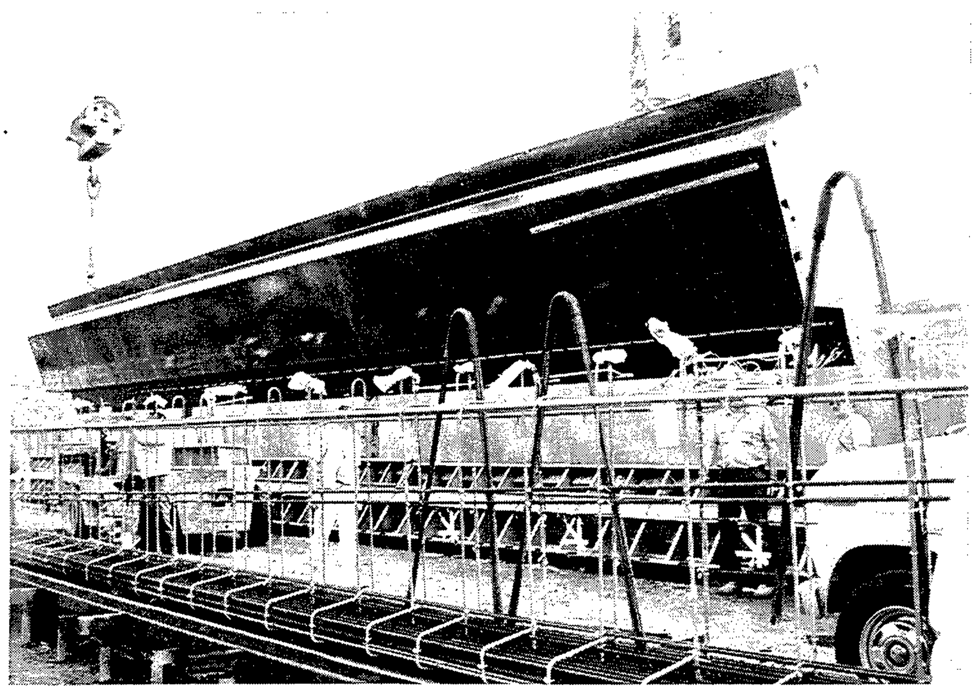
Prestressing The Strands
Figure 5.5



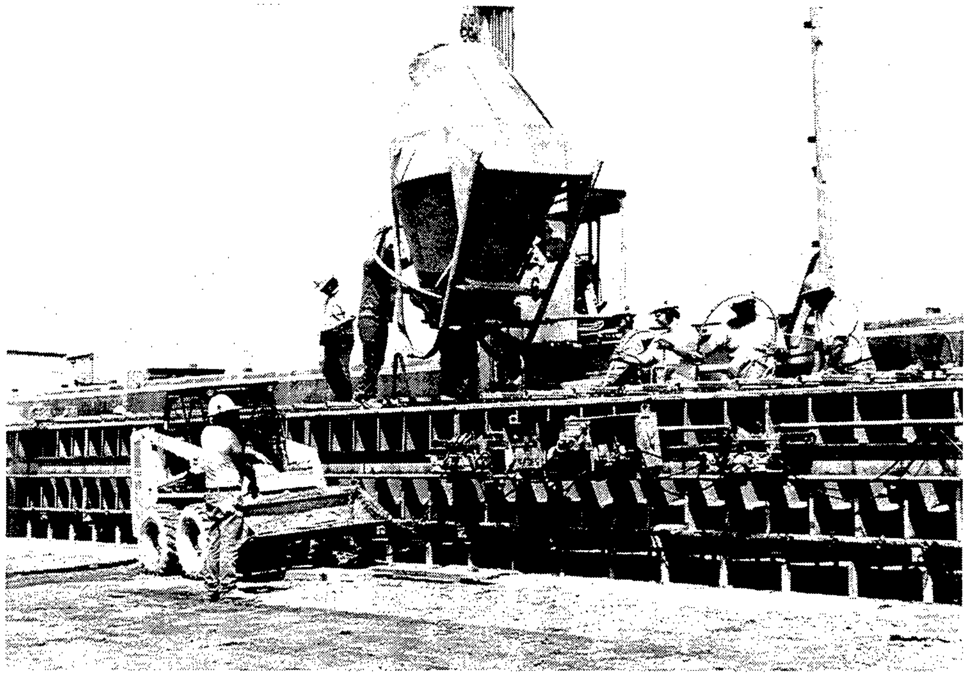
Strand Tensioning Sequence
Figure 5.6



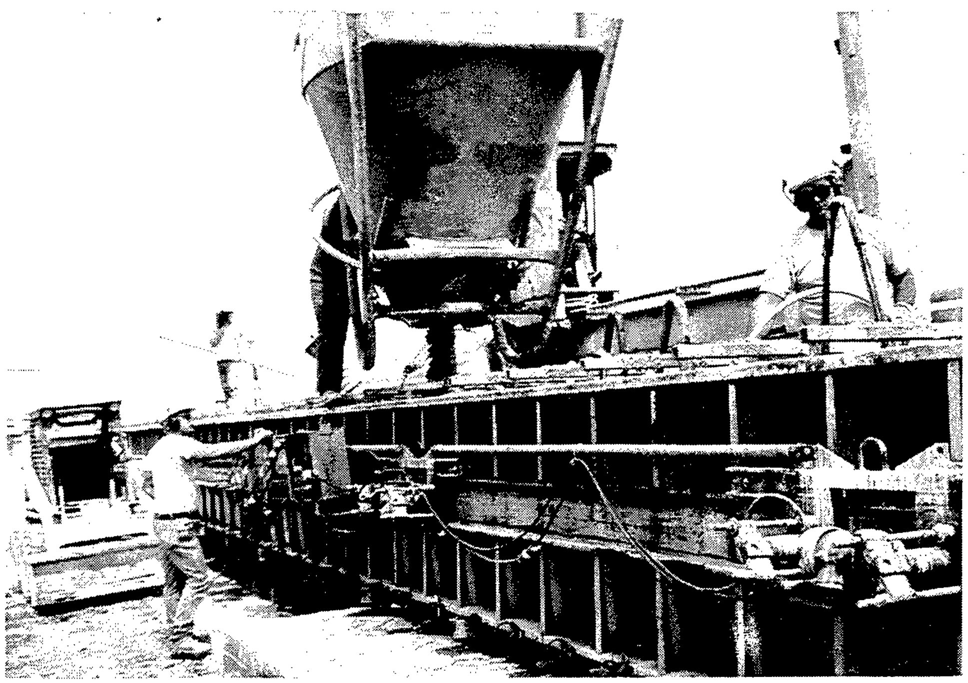
Test Girder End Instrumentation and Reinforcement
Figure 5.7



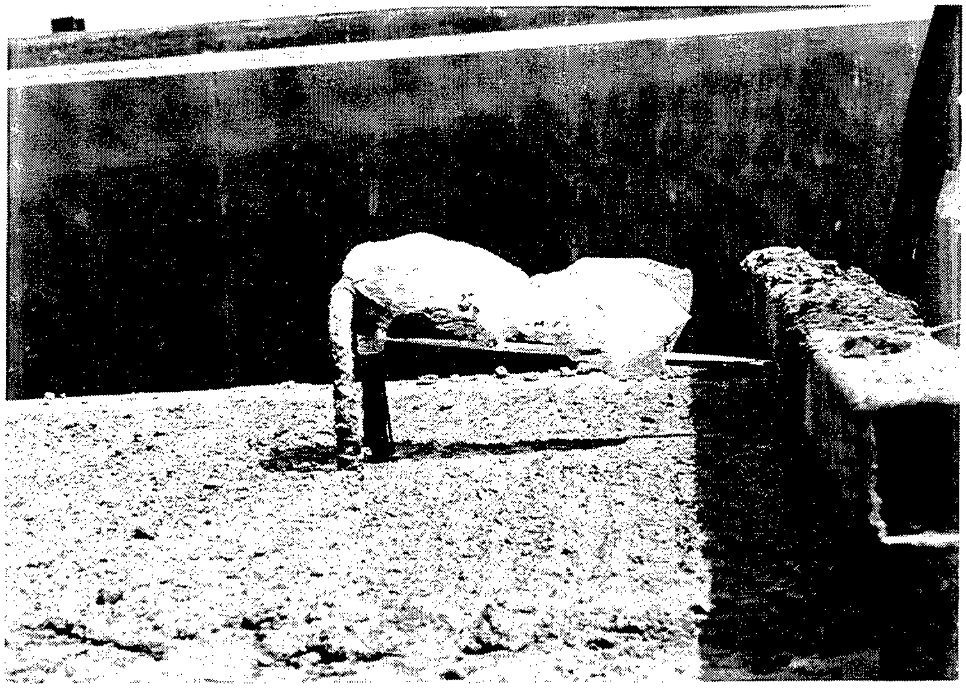
Installing Girder Forms
Figure 5.8



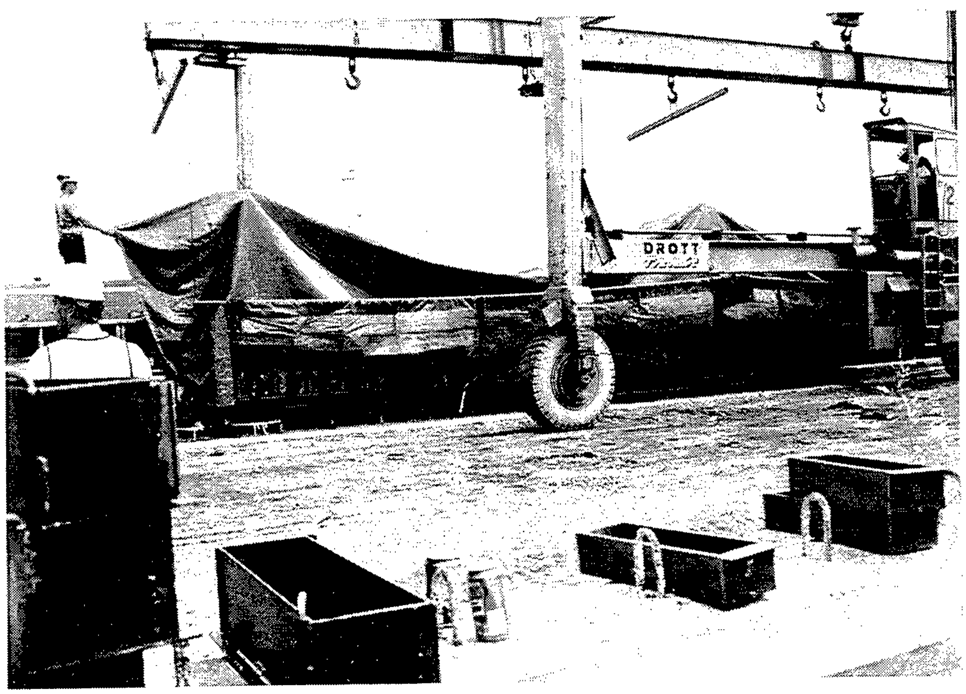
Casting the Test Girders
Figure 5.9



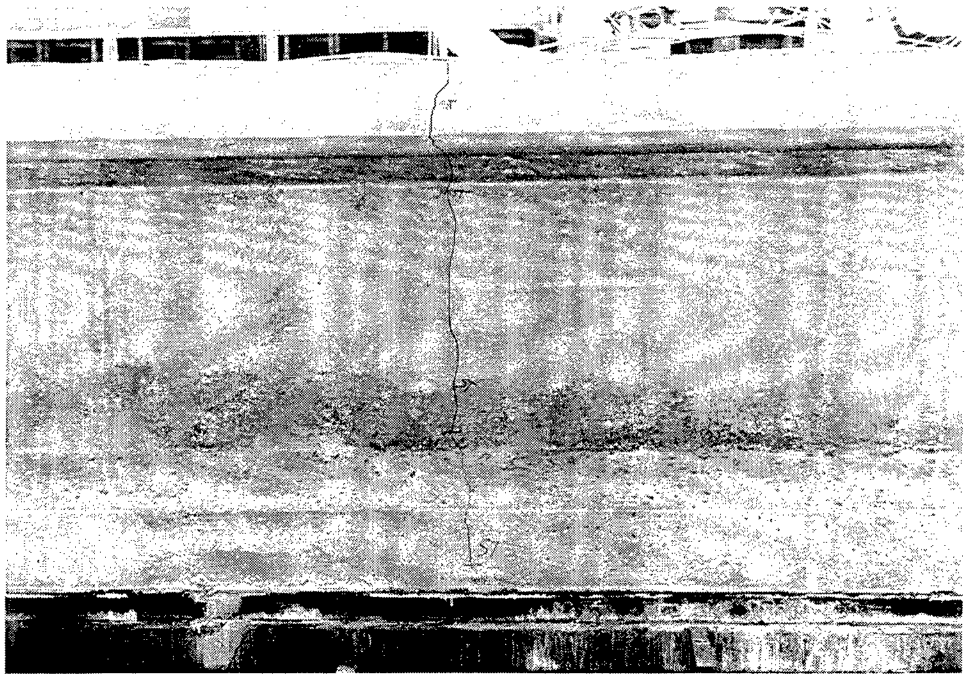
Concrete Consolidation
Figure 5.10



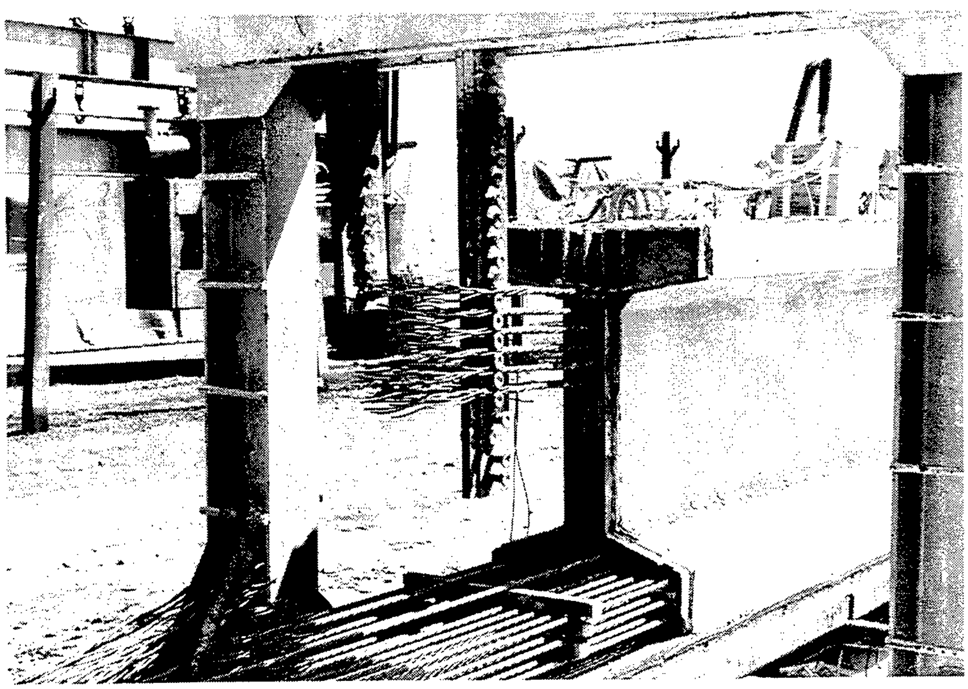
Finished Surface at Top of Girder Section
Figure 5.11



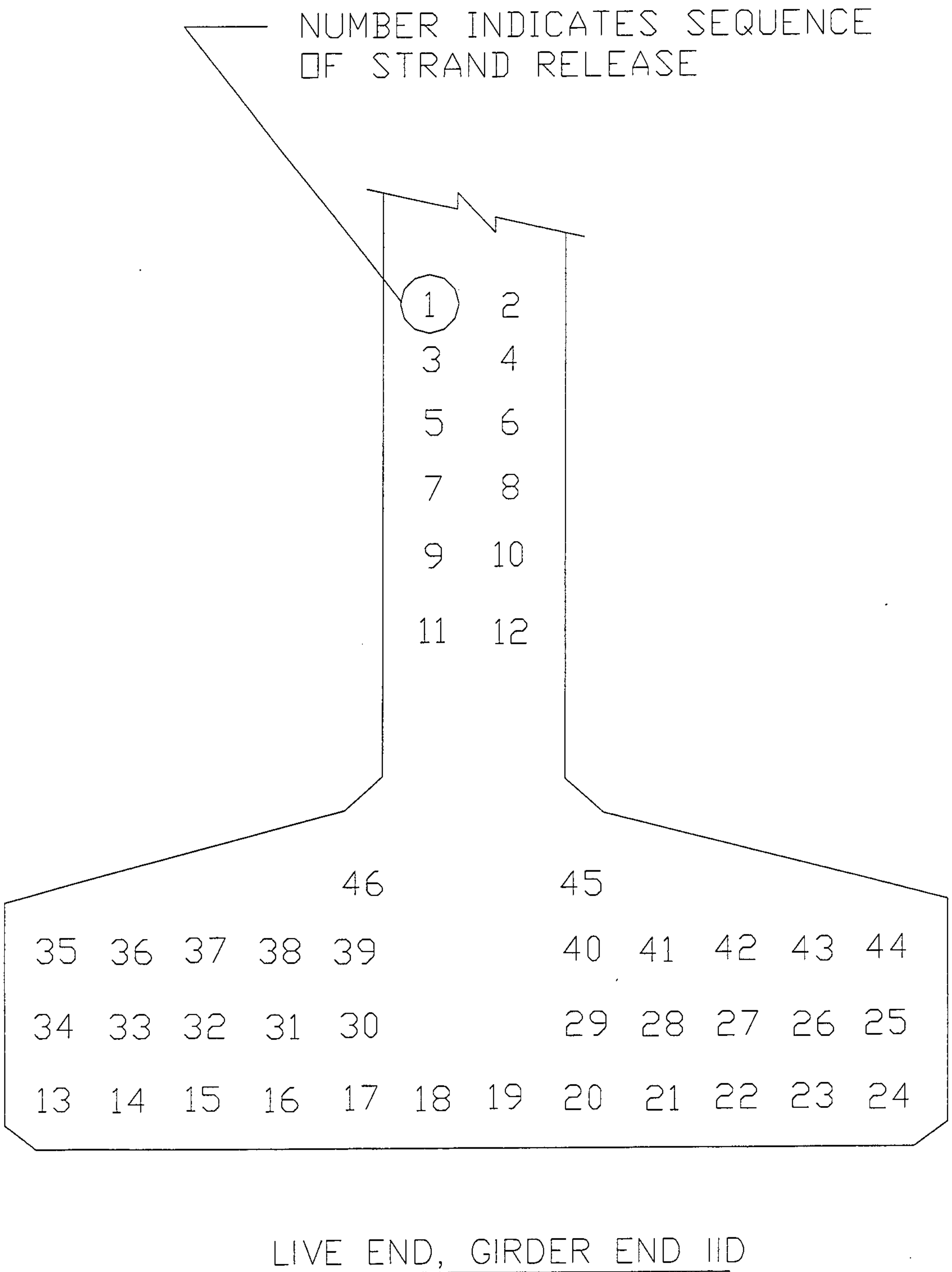
Placement of Curing Tarps
Figure 5.12



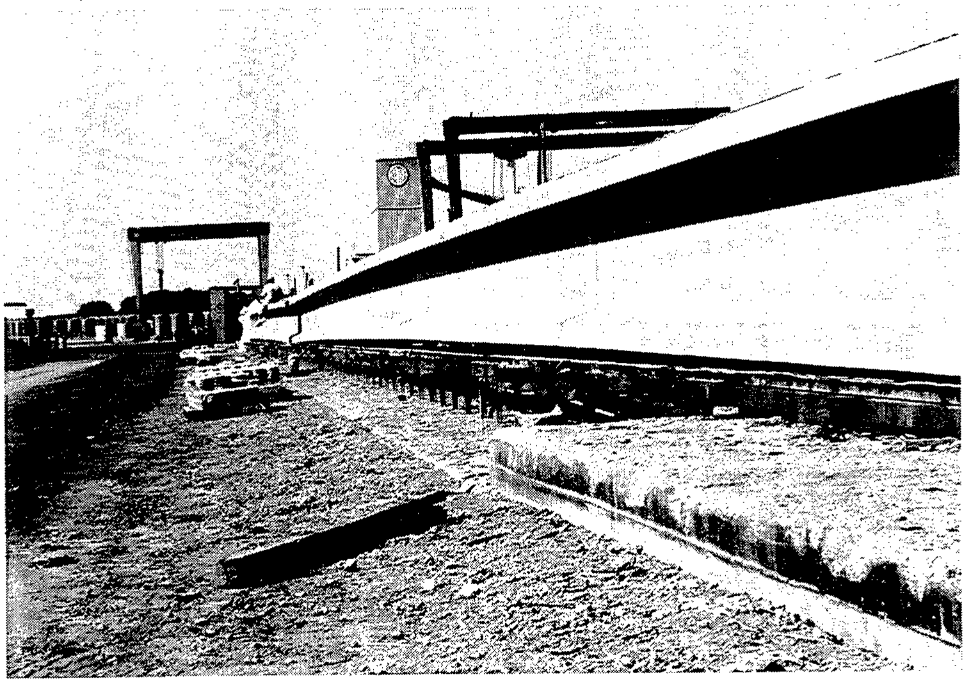
Shrinkage Cracking of Test Girder II
Figure 5.13



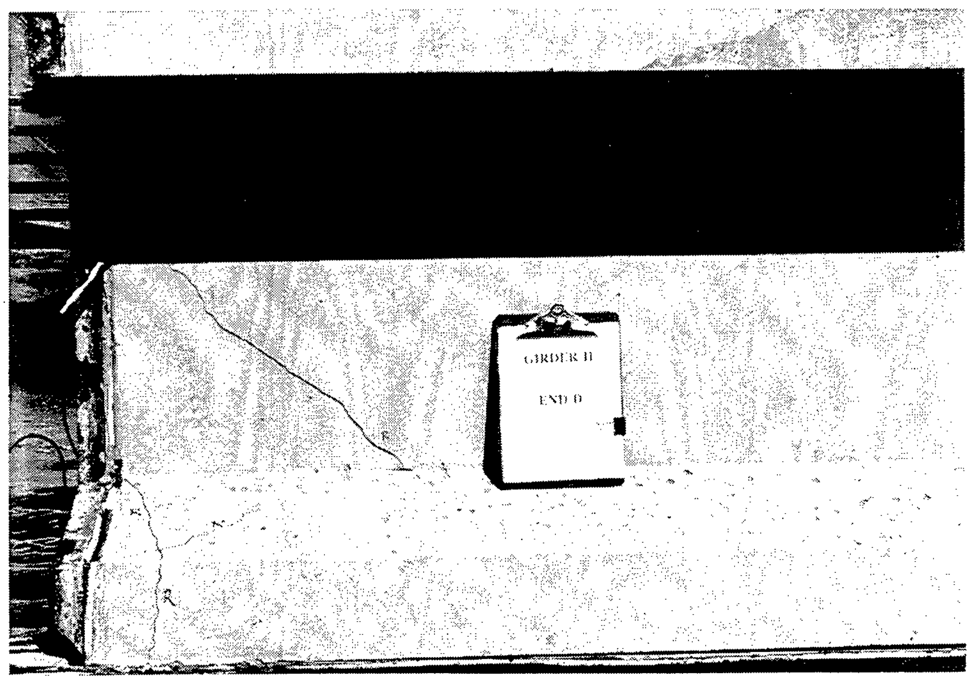
Released Strands and View of Strand Draping Hardware
Figure 5.14



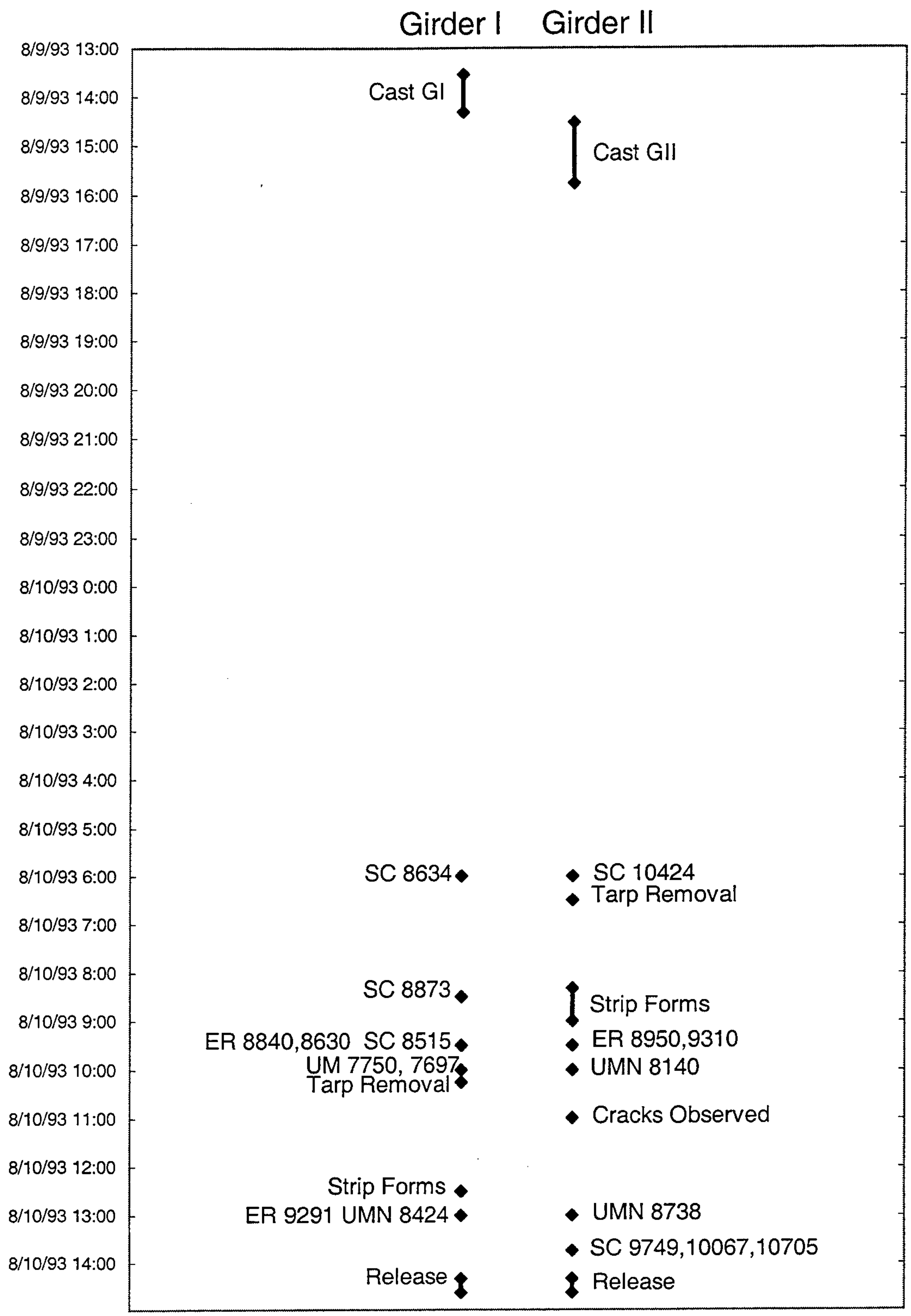
Strand Release Sequence
Figure 5.15



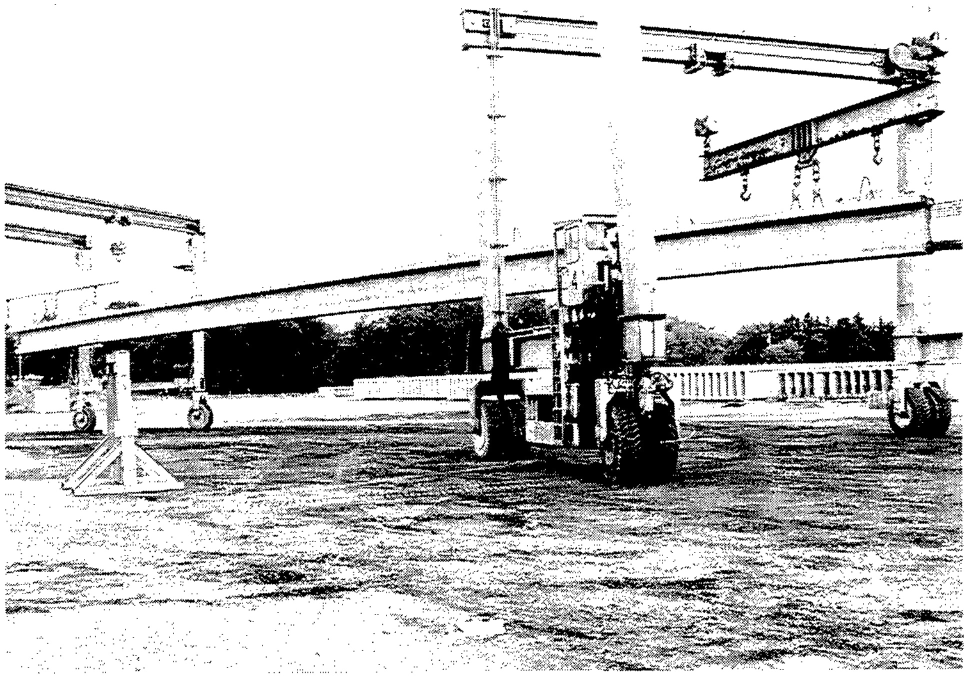
Camber of Test Girders on Prestressing Bed
Figure 5.16



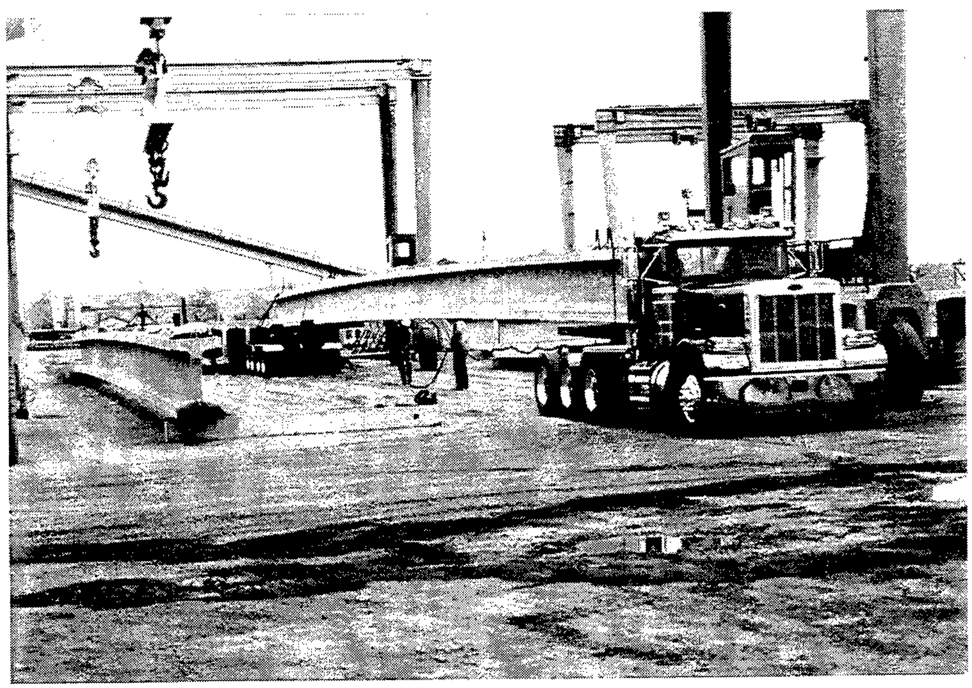
Cracking at End Regions
Figure 5.17



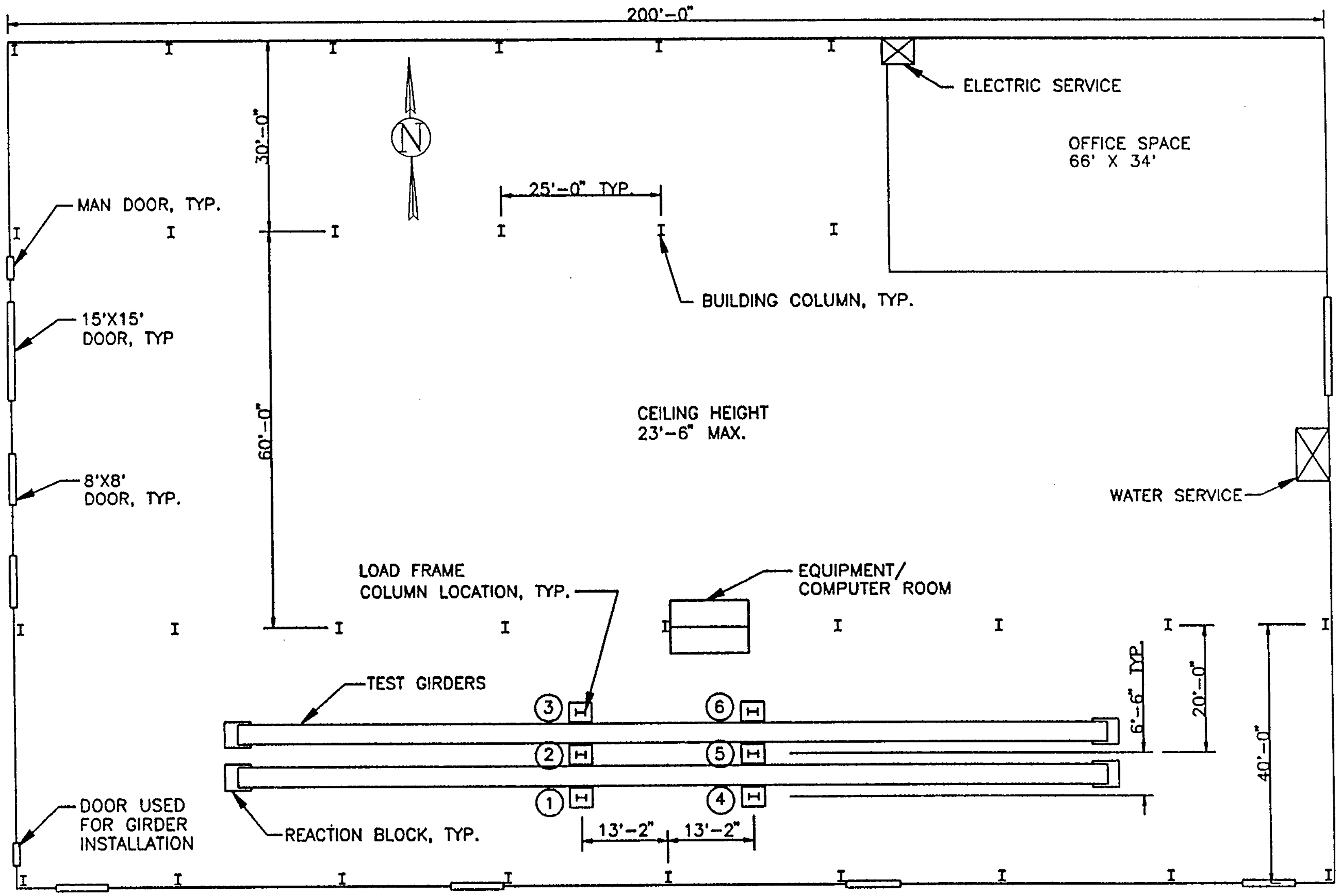
Girder Construction Timelines
Figure 5.18



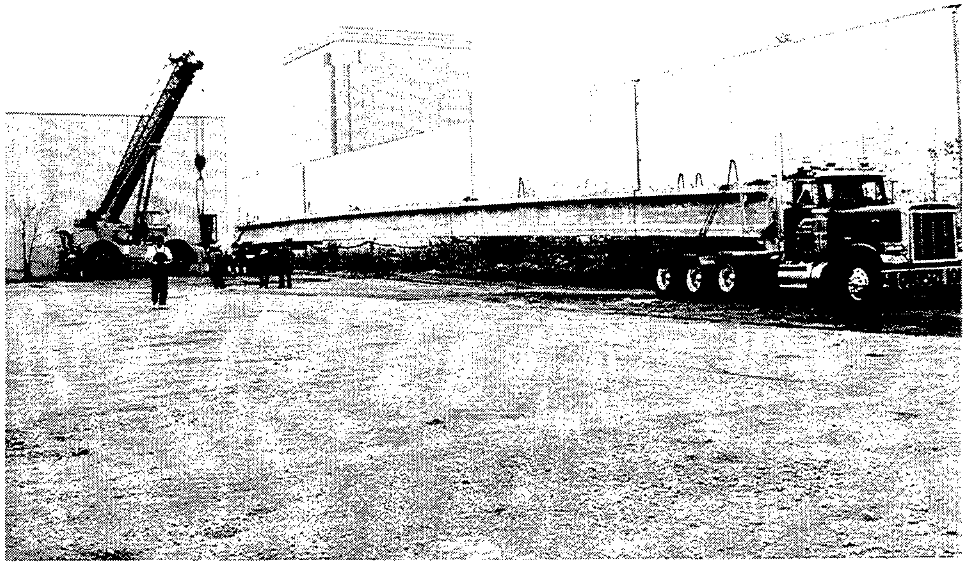
Moving Test Girder at Prestressing Yard
Figure 6.1



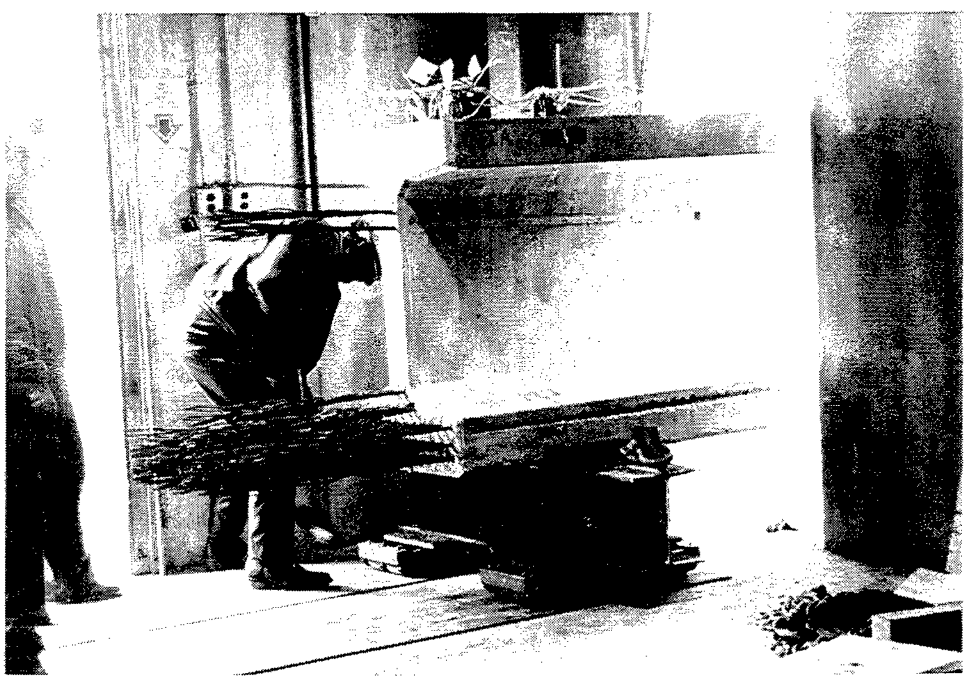
Test Girder on Truck
Figure 6.2



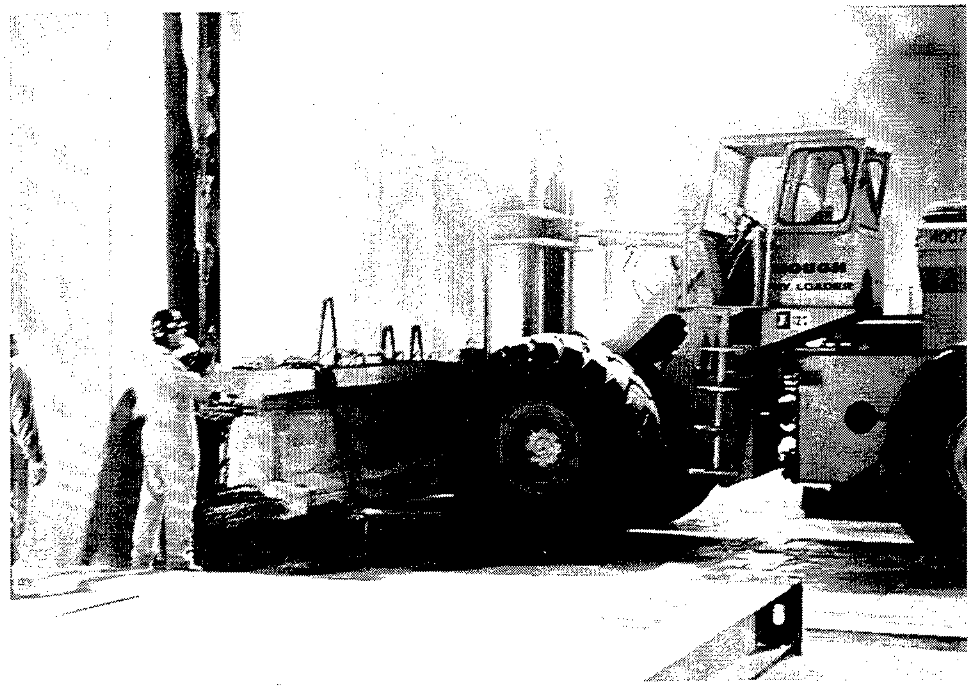
Layout of Off-Site Testing Laboratory
Figure 6.3



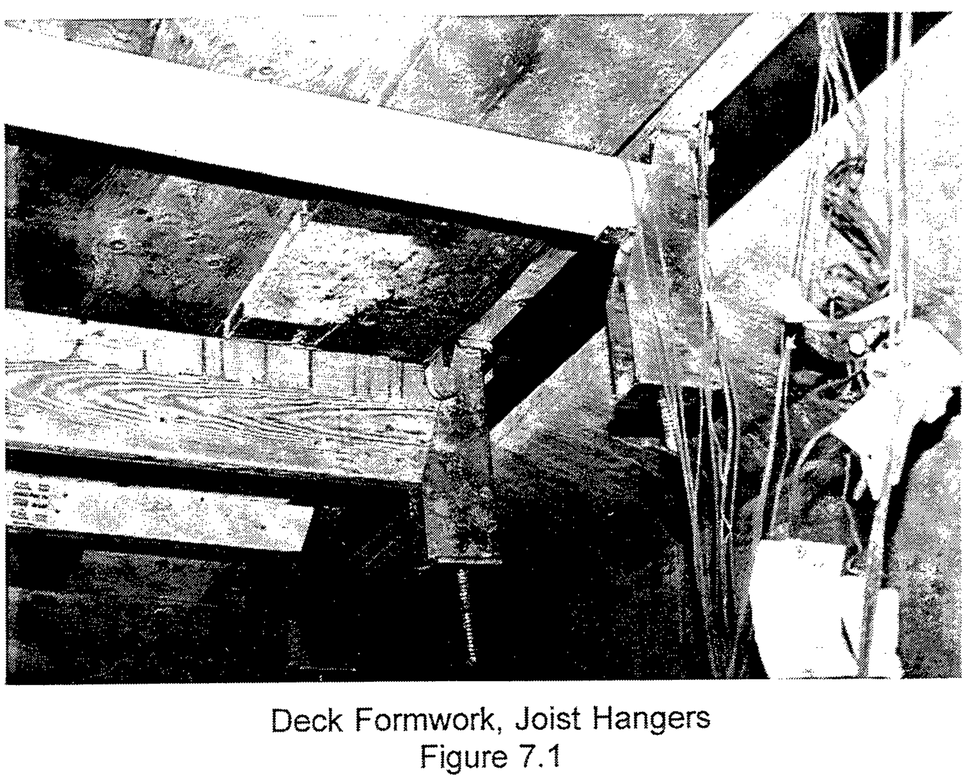
Arrival of Test Girder at Testing Laboratory
Figure 6.4



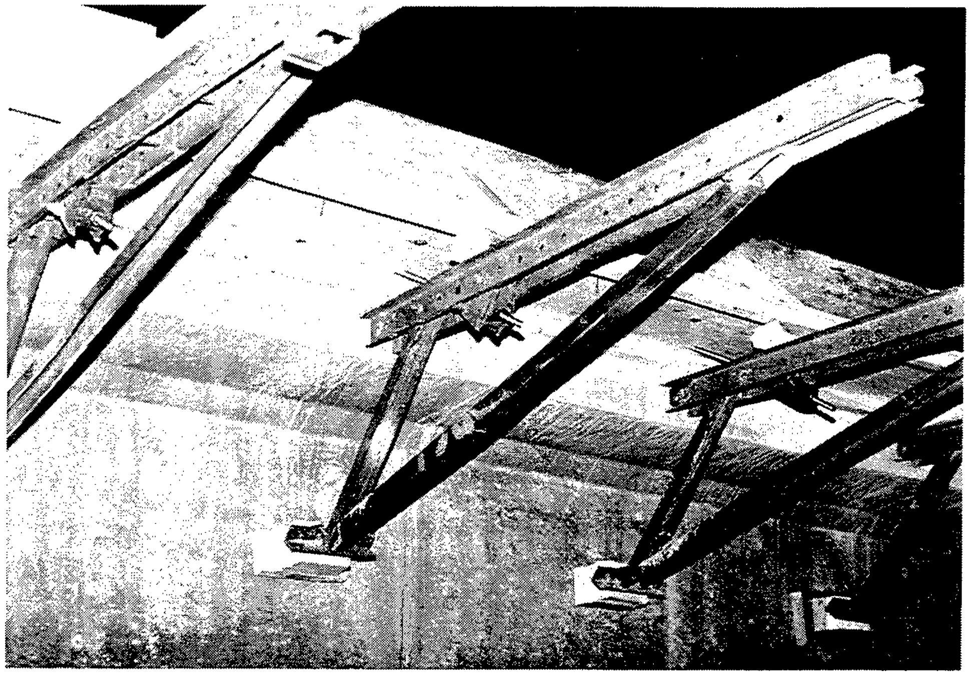
Test Girder End on Skate
Figure 6.5



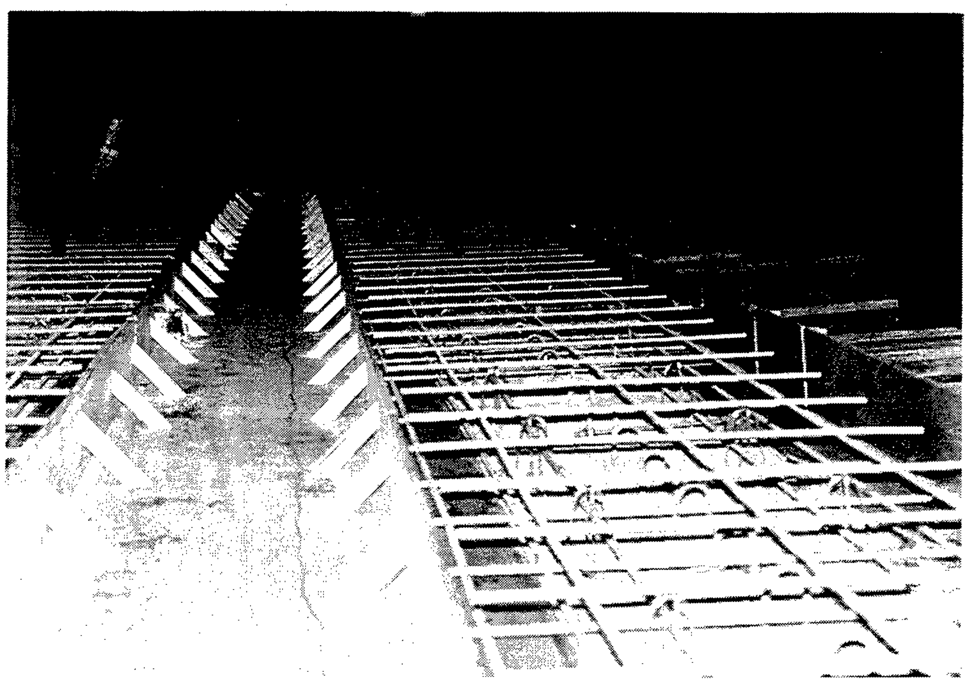
Lifting Test Girder In Laboratory
Figure 6.6



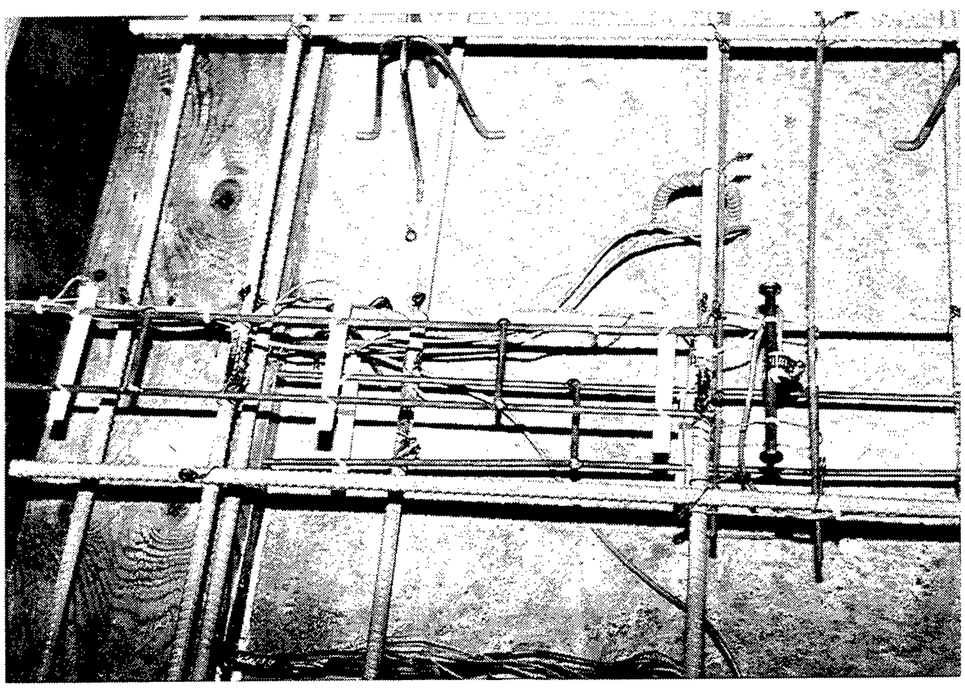
Deck Formwork, Joist Hangers
Figure 7.1



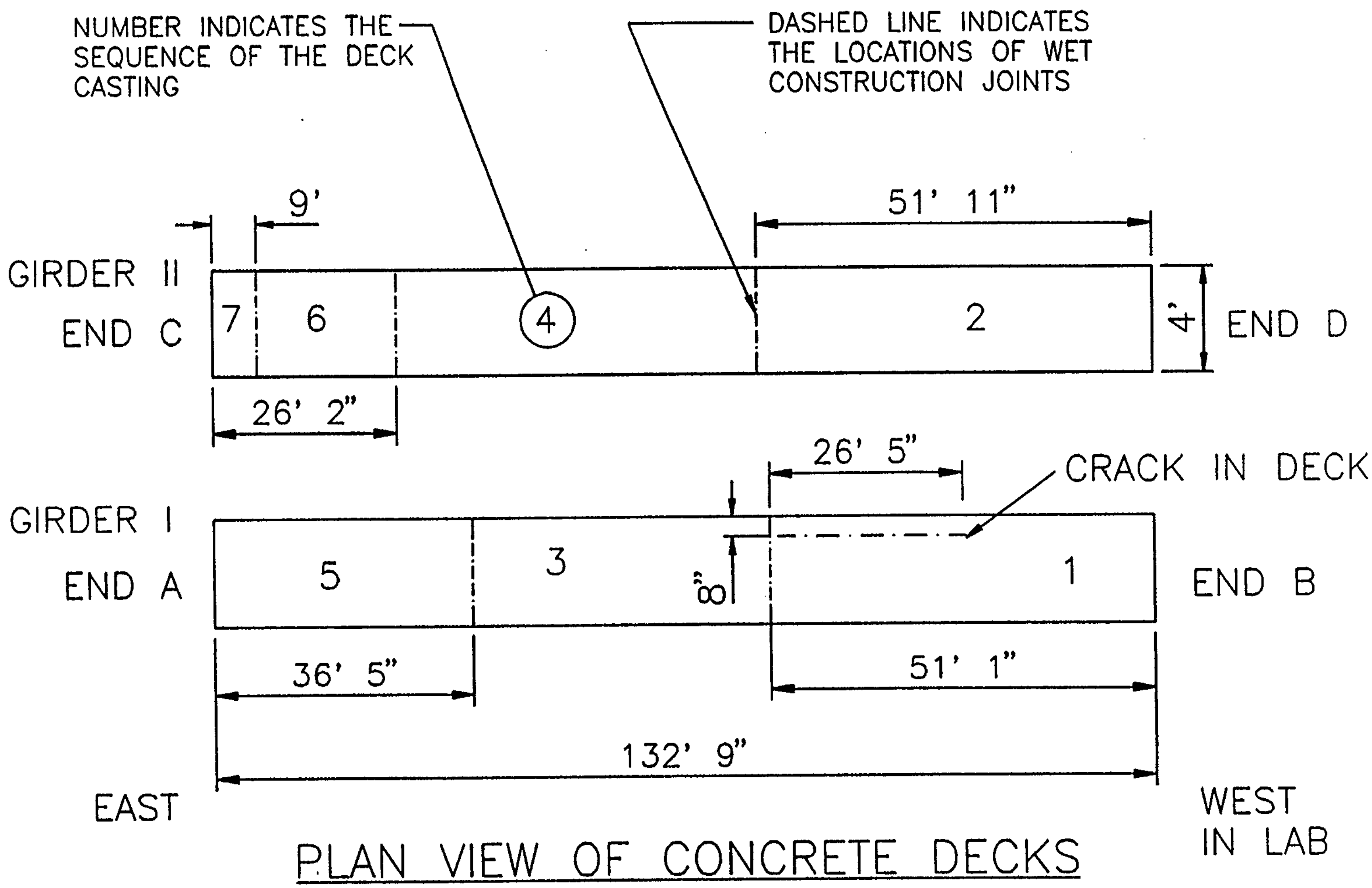
Deck Formwork, Overhang Brackets
Figure 7.2



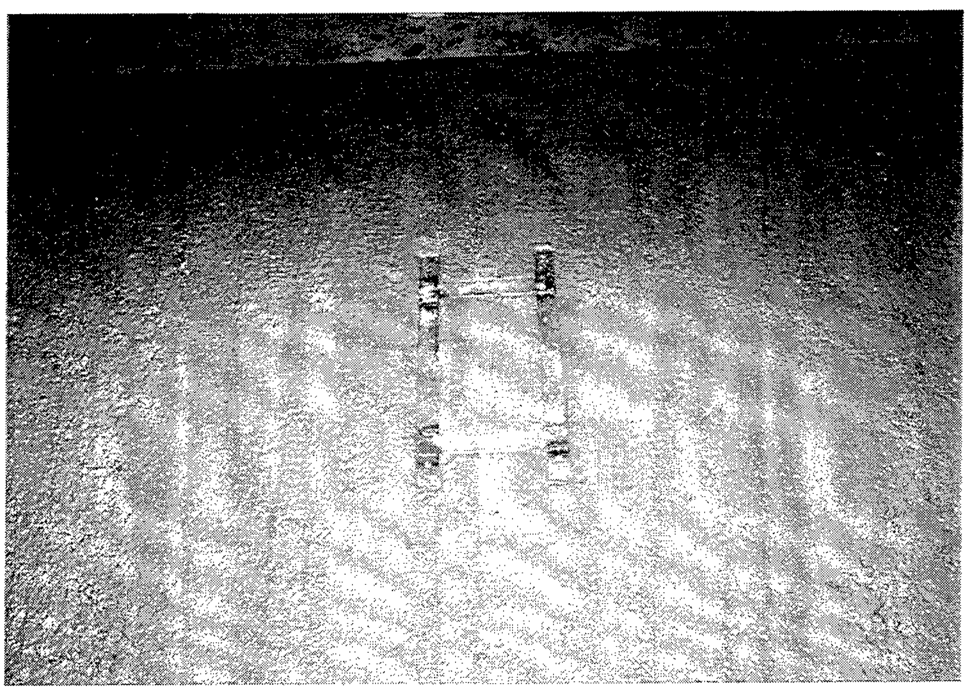
Concrete Deck Mild Steel Reinforcement
Figure 7.3



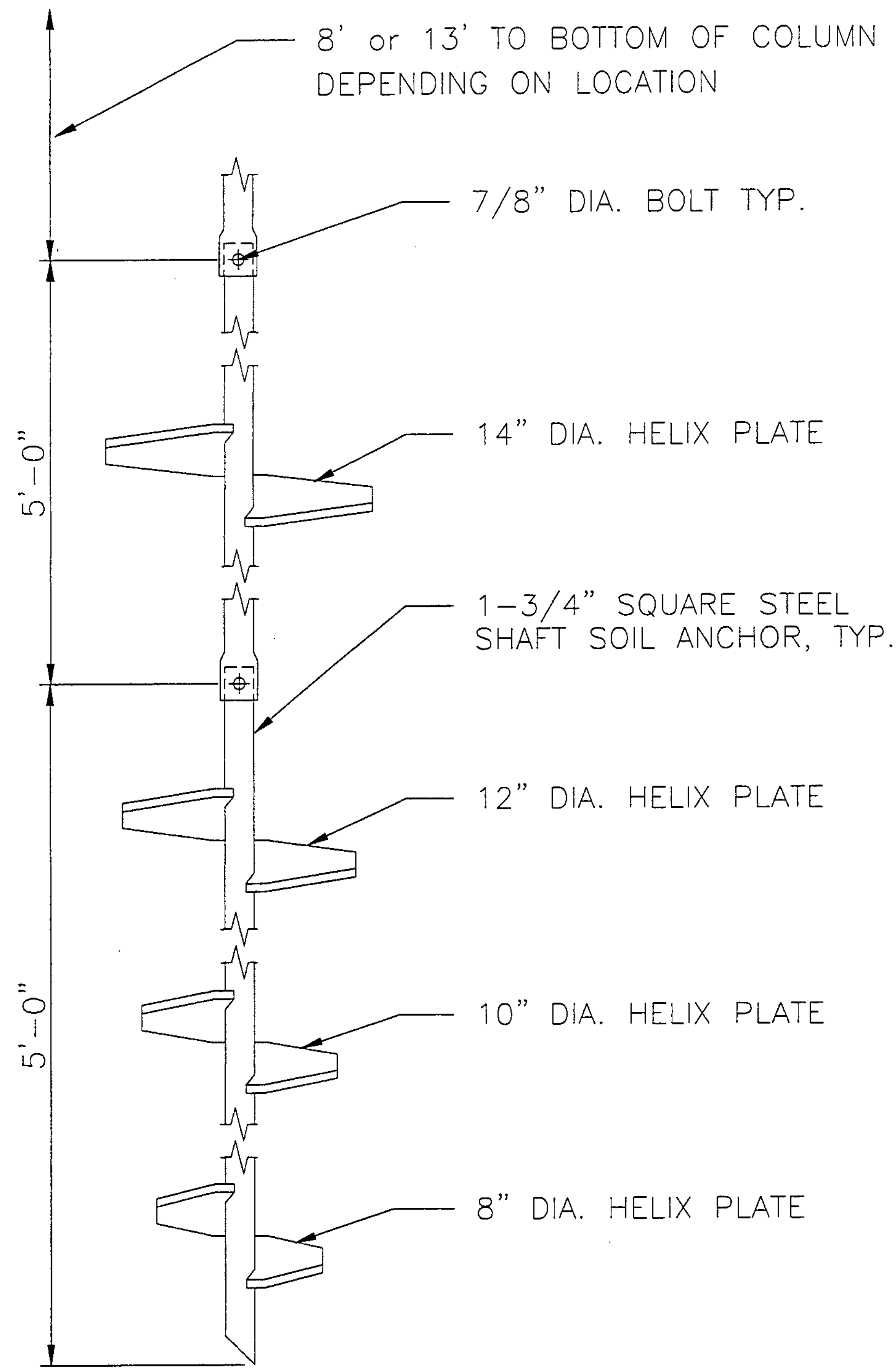
Deck Instrumentation
Figure 7.4



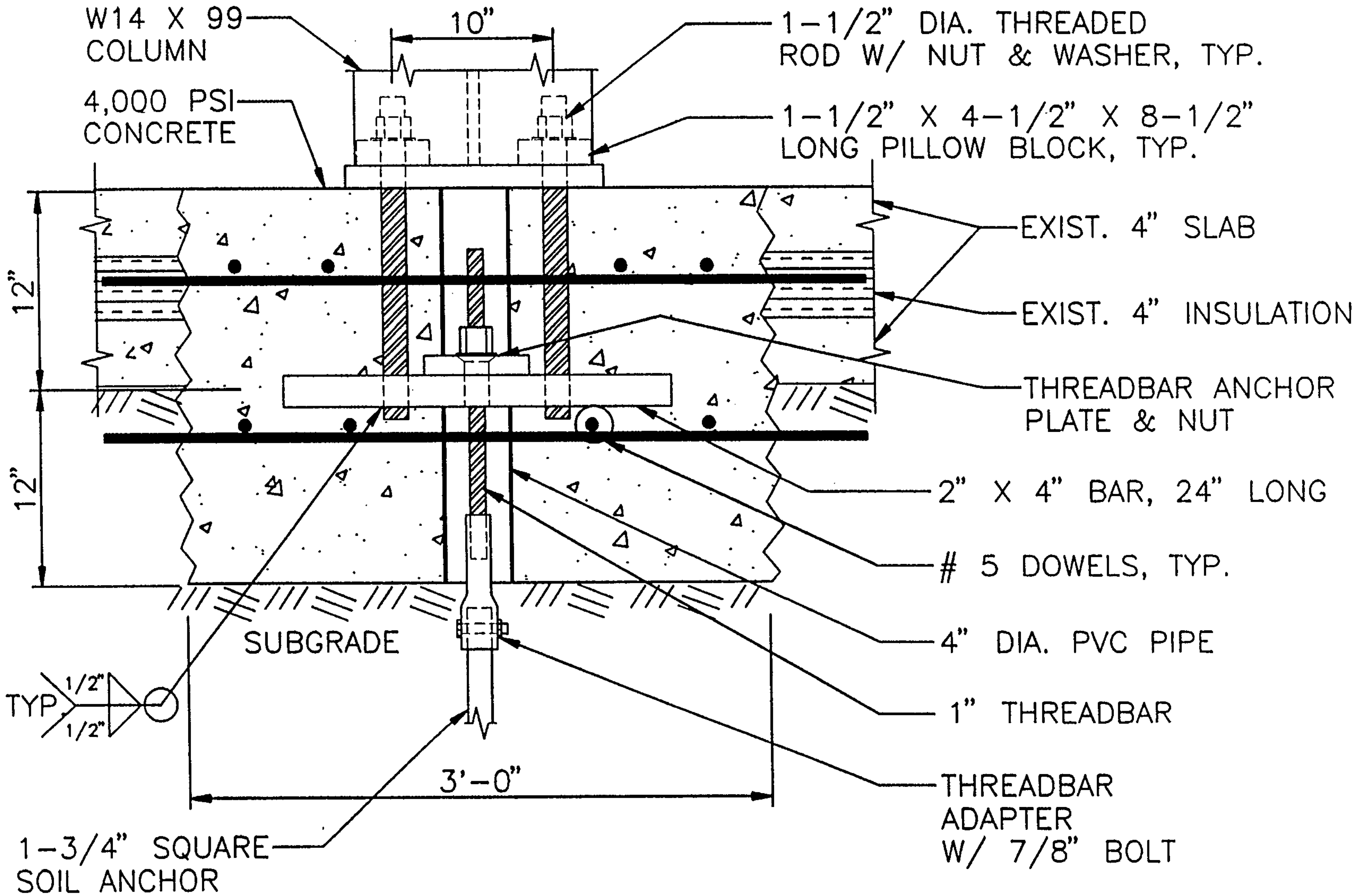
Sequence of Concrete Deck Casting
Figure 7.5



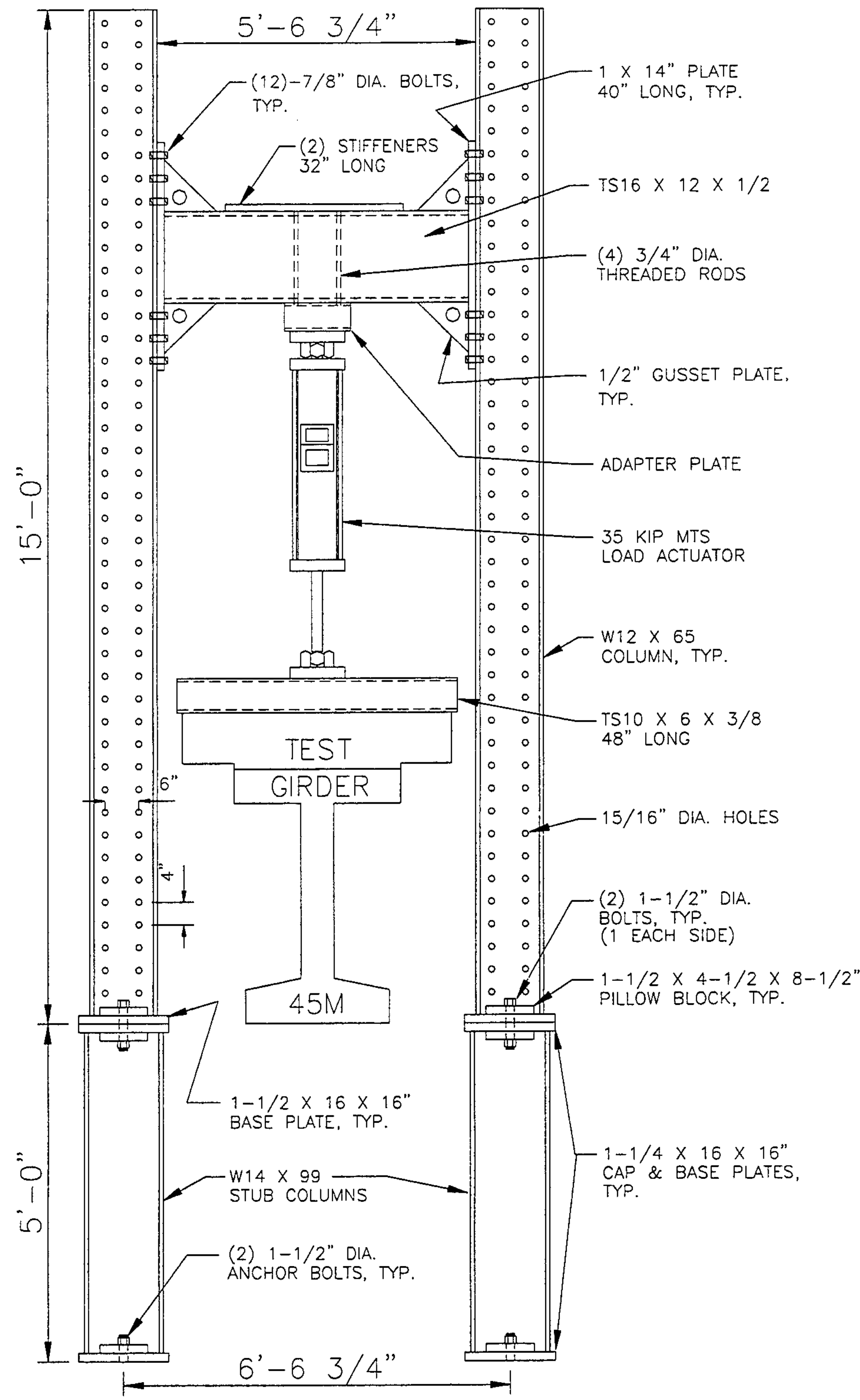
DEMEC Instrumentation Installed on Concrete Deck
Figure 7.6



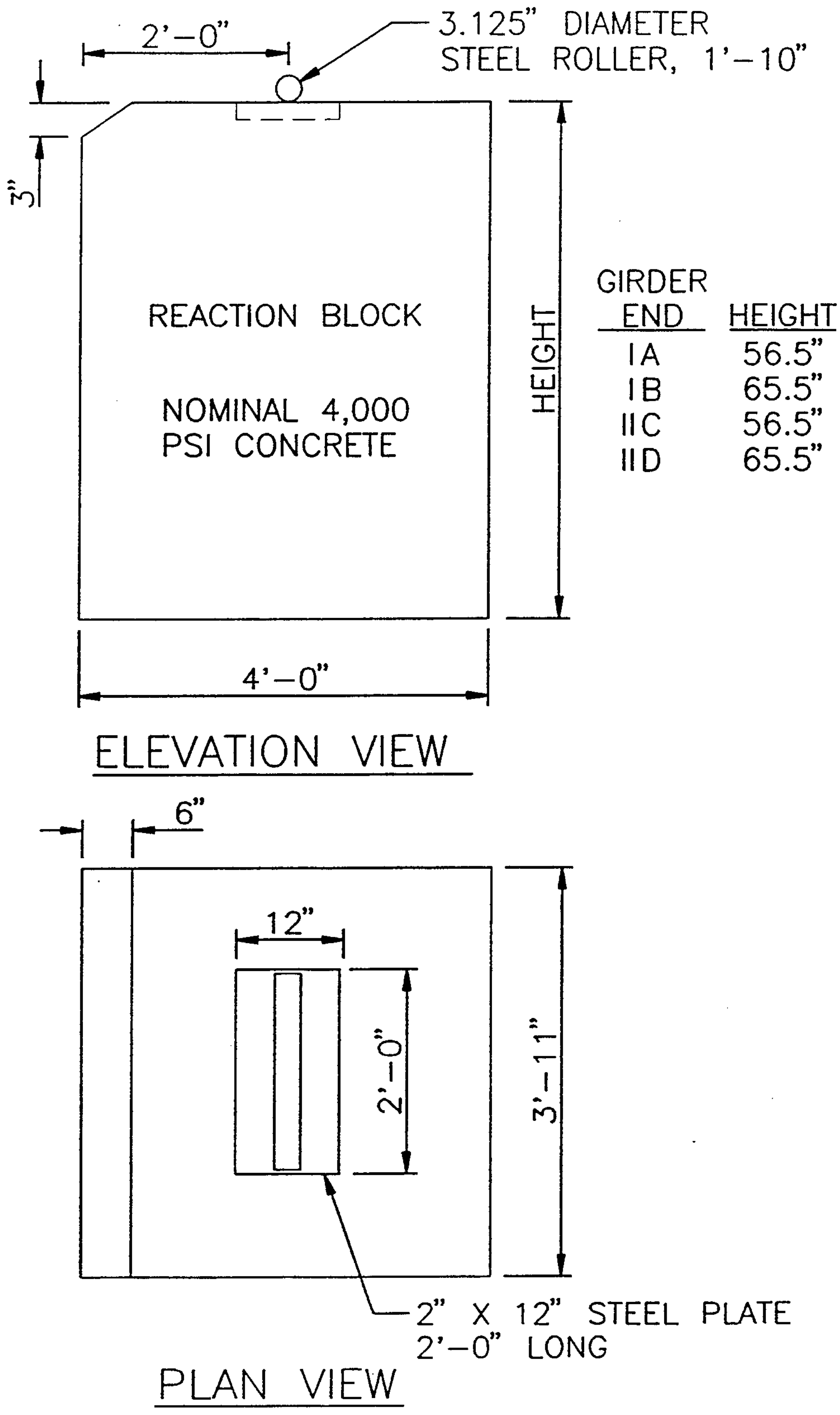
Detail of Typical Soil Anchor
Figure 8.1



Detail of Column to Soil Anchor Connection
Figure 8.2



Load Frame, Elevation
Figure 8.3



Reaction Block Details
Figure 8.4

Appendix A
Design of Test Girders: Computer Output

```
UNIVERSITY OF MINNESOTA                         PHONE: (612) 625-3877  | SHEET    OF
500 PILLSBURY DRIVE S.E.   MINNEAPOLIS,  MN  55495-0220                | JOB NO DRPTEST
PROGRAM: SPAN  -V0.1 DEVELOPED BY LEAP SOFTWARE, INC TAMPA FL          | DATE 04-11-1994
PHONE  : TOLL-FREE 1-800-451-LEAP ( TAMPA AREA 813-985-9170 )          | BY      TMA
-----------------------------------------------------------------------------------
State   : MN                                    State Job No : NONE
Project : UMN TEST GIRDER - DRAPED
Subtask : DESIGN:f'c=10.5, f'ci=8.925, f'cd=4, DRAPED 46/12

PRECAST DATA:                               GENERAL BRIDGE DATA:
-------------                               ---------------------
Section Id: MN/DOT 45M                       System: Longit. Stringer
Type       : Special                        Bridge width =  52.00 ft
Flange width =  30.00 in                    Curb-to-curb =  48.00 ft
Flange thick =   6.00 in                    Beam Spacing =   4.00 ft
Stems:  No.  =   1                          Lane width   =  12.00 ft
        Top  =   6.00 in                    No. of lanes =    4
        Bot  =   6.00 in                    No. of beams =   13
Shear width  =   6.00 in                    Int./Ext.    =    I

TOPPING DATA:                               GENERAL LOAD DATA:
-------------                               ------------------
Topg:  Thick =  10.00 in                    SDL (on precast) =   20.0 plf
       Width =  30.00 in                    SDL (on comp.)   =  203.0 plf
Gap:   Thick =   9.00 in                    Impact (Moment)  =  1.195
       Width =  18.00 in                    Impact (Shear)   =  1.195 (Assumed)
Eff. width   =  48.00 in                    Axle = HS25+Military

LOAD EQUATIONS:  Gamma  Beta    Beta  D.F.  ( Art. 3.22.2 )
---------------
Service Load  =   1.00*(1.00*DL+1.00*0.364*(LL+I))  Group I
                  Group IA loading NOT considered
Factored Load =   1.30*(1.00*DL+1.67*0.364*(LL+I))  Group I

SECTION PROPERTIES:                         GENERAL SPAN DATA:
-------------------                         ------------------
                   PRECAST    COMPOSITE     Overall Length = 132.75 ft
                   -------    ---------     Release Span   = 132.75 ft
Area            =  624.0 in2   895.5 in2*   Design Span    = 131.25 ft
Total Height    =  45.00 in    55.00 in     Shoring : NONE
Mom of Inertia = 167048 in4   315745 in4
Ht. of c.g.     =  22.34 in    30.78 in
Density         =  155.0 pcf   150.0 pcf
Self-weight     =  671.7 plf   481.3 plf    MISCELLANEOUS:
At 28-Days:                                 --------------
  Strength      =  10500 psi    4000 psi    Kern pts:   Upper = 34.32 in
  Max comp,top =   4200 psi    1600 psi                Lower = 10.53 in
  Max comp,bot =   4200 psi                 Trans len mult,Bonded=1.0
  Max tens,top =   -283 psi    -190 psi                  Debonded=1.0
  Max tens,bot =   -615 psi                 Dev len mult,  Bonded=1.6
  Elasticity    = 6525.4 Ksi  3834.3 Ksi                 Debonded=2.0
  Crack. tens. =   -786 psi
At Release:
  Strength      =   8925 psi
  Max comp,top =   5355 psi
  Max comp,bot =   5355 psi
  Max tens,top =   -200 psi
  Max tens,bot =      0 psi
  Elasticity    = 6016.1 Ksi   *Total transformed area using Ect/Ec = 0.5876
```

```
UNIVERSITY OF MINNESOTA                     PHONE: (612) 625-3877  | SHEET   OF
500 PILLSBURY DRIVE S.E.   MINNEAPOLIS,  MN  55495-0220            | JOB NO DRPTEST
PROGRAM: SPAN  -V0.1 DEVELOPED BY LEAP SOFTWARE, INC TAMPA FL      | DATE 04-11-1994
PHONE  : TOLL-FREE 1-800-451-LEAP ( TAMPA AREA 813-985-9170 )      | BY      TMA
------------------------------------------------------------------------------------
PRESTRESSED STEEL:  46  strands, 6/10 in-270K-Low-Lax
                    Depressed at .4L ( 53.25 ft from member end )
END PATTERN:      12 @  2.00 in  10 @  4.00 in  10 @  6.00 in   2 @  8.00 in
   2 @ 28.00 in    2 @ 30.00 in   2 @ 32.00 in   2 @ 34.00 in   2 @ 36.00 in
   2 @ 38.00 in
MID PATTERN:
(A) DRAPED   :     2 @  4.00 in   2 @  6.00 in   2 @  8.00 in   2 @ 10.00 in
   2 @ 12.00 in    2 @ 14.00 in
(B) BALANCE  :    12 @  2.00 in  10 @  4.00 in  10 @  6.00 in   2 @  8.00 in

Strand Dia         = 0.6000 in      Ult strength(f's) =    270 Ksi
Strand Area        = 0.2150 in2     Initial prestress = 0.750f's = 202.5 Ksi
Total strand area =  9.890 in2      Initial pull      = 2002.7 K = 43.54 K/str
Trans len,basic   =   2.50 ft       Dev len, basic    =    6.98 ft
Trans len,bonded  =   2.50 ft       Dev len, bonded   =   11.17 ft
Trans len,debonded=   2.50 ft       Dev len, debonded =   13.96 ft
------------------------------------------------------------------------------------
REINFORCING STEEL:  Tension steel: fy = 60K  fs = 24K.  Shear steel: fsy = 60K
------------------------------------------------------------------------------------
Midspan:  Strand area=  9.89 in2  Ycg=  5.39 in  P-init=2002.7 K  Ecc= 16.95 in
Hours to release = 24   Rel. Humid. (RH) = 75%  Es= 28,000 Ksi  Eci = 6,016 Ksi

AASHTO LOSSES:          Release                       Final      ( Art. 9.16.2 )
Steel relaxation    RET*  1760 psi   CRs (Eq 9-10)     890 psi
Elastic shortening ES   19353 psi    ES  (Eq 9-6)    19353 psi   ( Fcir= 4158 psi )
Concrete shrinkage                   SH  (Eq 9-4)     5750 psi
Concrete creep                       CRc (Eq 9-9)    37748 psi   ( Fcds=-1736 psi )
                        -----                        -----
Total                   21113 psi (10.4%)            63742 psi (31.5%)

Steel relaxation before release - Ref: PCI Journal Vol 20, No.4, Jul-Aug 1975
------------------------------------------------------------------------------------
SHIELDING AND REDUCED INITIAL PULLS:  NONE
------------------------------------------------------------------------------------
SERVICE LOAD MOMENTS:
```

	Trans 1.75ft	H/2 2.29ft	0.10L 13.13ft	0.20L 26.25ft	0.30L 39.38ft	0.40L 52.50ft	Midspan 65.63ft
Self wt. ,Kft	76.1	99.2	520.7	925.6	1214.9	1388.5	1446.3
DL-Prec. ,Kft	2.3	3.0	15.5	27.6	36.2	41.3	43.1
Topping ,Kft	54.5	71.1	373.1	663.2	870.5	994.8	1036.3
DL-Comp. ,Kft	23.0	30.0	157.4	279.8	367.2	419.6	437.1
Eff. LL+I ,Kft	62.7	81.7	425.5	748.4	968.5	1098.2	1131.3

UNIVERSITY OF MINNESOTA	PHONE: (612) 625-3877	SHEET OF
500 PILLSBURY DRIVE S.E.	MINNEAPOLIS, MN 55495-0220	JOB NO DRPTEST
PROGRAM: SPAN -V0.1 DEVELOPED BY LEAP SOFTWARE, INC TAMPA FL		DATE 04-11-1994
PHONE : TOLL-FREE 1-800-451-LEAP (TAMPA AREA 813-985-9170)		BY TMA

RELEASE STRESSES (Loss = 10.4%) :

	Sup-Rel -0.75ft	Trans 1.75ft	0.10L 13.13ft	0.20L 26.25ft	0.30L 39.38ft	0.40L 52.50ft	Depress 52.50ft	Crit 54.16ft
Prest.								
Precast-top	0.0	202.5	-122.9	-498.5	-874.0	-1249.5	-1249.5	-1249.5
Bottom	0.0	5509.5	5830.3	6200.5	6570.8	6941.0	6941.0	6941.0
Self wt.								
Precast-top	0.0	178.0	901.7	1560.9	2031.7	2314.3	2314.3	2336.6
Bottom	0.0	-175.5	-888.9	-1538.8	-2003.0	-2281.6	-2281.6	-2303.6
Total								
Precast-top	0.0	380.5	778.7	1062.4	1157.8	1064.8	1064.8	1087.1
Bottom	0.0	5334.0	4941.4	4661.7	4567.7	4659.4	4659.4	4637.4
As-top, in2	NONE	NONE	NONE	NONE	NONE	NONE	NONE	NONE

FINAL STRESSES (Loss = 31.5%) :

	Sup-Fin 0.00ft	Trans 1.75ft	H/2 2.29ft	0.10L 13.13ft	0.20L 26.25ft	0.30L 39.38ft	0.40L 52.50ft	Midspan 65.63ft
Prest.								
Precast-top	58.0	154.9	143.1	-94.0	-381.3	-668.6	-955.8	-955.8
Bottom	1253.1	4214.7	4226.4	4460.1	4743.3	5026.5	5309.8	5309.8
Self wt.								
Precast-top	0.0	123.9	161.6	847.5	1506.8	1977.6	2260.1	2354.3
Bottom	0.0	-122.1	-159.3	-835.6	-1485.5	-1949.7	-2228.2	-2321.1
DL-Prec.								
Precast-top	0.0	3.7	4.8	25.2	44.9	58.9	67.3	70.1
Bottom	0.0	-3.6	-4.7	-24.9	-44.2	-58.1	-66.3	-69.1
Topping								
Precast-top	0.0	88.8	115.8	607.3	1079.6	1417.0	1619.4	1686.9
Bottom	0.0	-87.5	-114.1	-598.7	-1064.3	-1397.0	-1596.5	-1663.0
DL-Comp.								
Topping-top	0.0	12.4	16.2	85.1	151.3	198.6	227.0	236.4
Precast-top	0.0	12.4	16.2	85.1	151.2	198.5	226.8	236.3
Bottom	0.0	-26.9	-35.1	-184.1	-327.2	-429.5	-490.9	-511.3
LL+I								
Topping-top	0.0	33.9	44.2	230.2	404.8	523.9	594.0	611.9
Precast-top	0.0	33.9	44.1	230.0	404.5	523.5	593.6	611.5
Bottom	0.0	-73.3	-95.5	-497.7	-875.4	-1132.9	-1284.6	-1323.3
Total								
Topping-top	0.0	46.3	60.4	315.3	556.1	722.5	821.0	848.4
Precast-top	58.0	417.6	485.5	1701.1	2805.6	3506.8	3811.4	4003.1
Bottom	1253.1	3901.2	3817.6	2319.1	946.6	59.4	-356.8	-578.1

VERTICAL SHEAR (Vu @ h/2 = 190.7 K E for fpc= 8.44 in Art. 9.20

	Sup-Fin 0.00ft	Trans 1.75ft	H/2 2.29ft	0.02L 2.63ft	0.04L 5.25ft	0.06L 7.88ft	0.08L 10.50ft
Vd,K	90.3	87.9	87.1	86.7	83.1	79.5	75.8
Md,Kft	0.0	155.9	203.3	232.3	455.1	668.4	872.2
Ml,Kft	0.0	62.7	81.7	93.3	182.5	267.6	348.6
Vu,K	190.7	190.7	190.7	189.9	183.5	177.1	170.7
Mu,Kft	0.0	338.7	441.6	504.5	987.9	1449.9	1890.8
Mmax,Kft	0.0	182.8	238.3	272.3	532.8	781.5	1018.5
Vu @ Mu,K	196.3	192.0	190.7	189.9	183.5	177.1	170.7
Vi,K	106.0	104.1	103.5	103.2	100.4	97.6	94.8
fpe,psi	1253.1	4214.7	4226.4	4233.5	4290.2	4346.8	4403.5
fd,psi	0.0	-240.2	-313.2	-357.9	-701.1	-1029.8	-1343.8
Mcr,Kft	1596.8	3923.3	3870.9	3838.9	3593.8	3361.3	3141.3
d	44.0	44.0	44.0	44.0	44.1	44.4	44.7
Vci-com,K	10000.0	2338.3	1784.9	1557.8	776.6	515.7	384.8
Vci-min,K	46.0	46.0	46.0	46.0	46.0	46.4	46.7
Vci,K	10000.0	2338.3	1784.9	1557.8	776.6	515.7	384.8
fpc,psi	435.7	1518.5	1538.6	1550.8	1644.6	1733.4	1817.4
Vp,K	4.0	13.4	13.4	13.4	13.4	13.4	13.4
Vcw,K	133.2	228.4	230.0	230.9	238.6	247.3	255.7
Vc,K	133.2	228.4	230.0	230.9	238.6	247.3	255.7
Vs-reqd,K	78.6	0.0	0.0	0.0	0.0	0.0	0.0
Vs-max,K	216.4	216.4	216.4	216.4	216.7	218.2	219.7
Av-com,in2/F	0.36	0.00	0.00	0.00	0.00	0.00	0.00
Av-min,in2/F	0.06	0.06	0.06	0.06	0.06	0.06	0.06
Av,in2/F	0.36	0.06	0.06	0.06	0.06	0.06	0.06
Vs-crit,K	108.2	108.2	108.2	108.2	108.3	109.1	109.9
Max spc,in	24.00	24.00	24.00	24.00	24.00	24.00	24.00

	0.10L 13.13ft	0.20L 26.25ft	0.30L 39.38ft	0.40L 52.50ft	Midspan 65.63ft	Depress 52.50ft
Vd,K	72.2	54.2	36.1	18.1	0.0	18.1
Md,Kft	1066.6	1896.2	2488.7	2844.3	2962.8	2844.3
Ml,Kft	425.5	748.4	968.5	1098.2	1131.3	1098.2
Vu,K	164.3	132.3	100.4	68.4	36.4	68.4
Mu,Kft	2310.4	4089.7	5338.0	6081.8	6307.6	6081.8
Mmax,Kft	1243.8	2193.6	2849.3	3237.5	3344.8	3237.5
Vu @ Mu,K	164.3	132.3	100.4	62.0	30.0	62.0
Vi,K	92.1	78.1	64.2	43.9	30.0	43.9
fpe,psi	4460.1	4743.3	5026.5	5309.8	5309.8	5309.8
fd,psi	-1643.2	-2921.3	-3834.2	-4382.0	-4564.5	-4382.0
Mcr,Kft	2933.7	2083.2	1544.9	1318.8	1162.7	1318.8
d	45.0	46.5	48.1	49.6	49.6	49.6
Vci-com,K	306.0	145.6	88.7	54.2	28.7	54.2
Vci-min,K	47.0	48.6	50.2	51.9	51.9	51.9
Vci,K	306.0	145.6	88.7	54.2	51.9	54.2
fpc,psi	1896.4	2218.1	2417.3	2494.1	2555.3	2494.1
Vp,K	13.4	13.4	13.4	13.4	0.0	13.4
Vcw,K	263.8	299.3	326.0	342.9	334.9	342.9
Vc,K	263.8	145.6	88.7	54.2	51.9	54.2
Vs-reqd,K	0.0	1.5	22.8	21.7	0.0	21.7
Vs-max,K	221.2	228.8	236.4	244.0	244.0	244.0
Av-com,in2/F	0.00	0.01	0.09	0.09	0.00	0.09
Av-min,in2/F	0.06	0.06	0.06	0.06	0.06	0.06
Av,in2/F	0.06	0.06	0.09	0.09	0.06	0.09
Vs-crit,K	110.6	114.4	118.2	122.0	122.0	122.0
Max spc,in	24.00	24.00	24.00	24.00	24.00	24.00

HORIZONTAL SHEAR (Q= 5265.6 in3 I= 315745 in4 b= 30.00 in) :

	Sup-Fin 0.00ft	Trans 1.75ft	H/2 2.29ft	0.02L 2.63ft	0.04L 5.25ft	0.06L 7.88ft	0.08L 10.50ft
Vu-cmp,K	96.2	94.6	94.1	93.8	91.4	89.0	86.6
vh,psi	53.5	52.6	52.3	52.1	50.8	49.5	48.2
Surf,in2/F	360.00	360.00	360.00	360.00	360.00	360.00	360.00
Avhmin,in2/F	0.22	0.22	0.22	0.22	0.22	0.22	0.22
Max spc,in	24.00	24.00	24.00	24.00	24.00	24.00	24.00
Avh-sm,in2/F	0.22	0.22	0.22	0.22	0.22	0.22	0.22
Avh-rg,in2/F	0.22	0.22	0.22	0.22	0.22	0.22	0.22

	0.10L 13.13ft	0.20L 26.25ft	0.30L 39.38ft	0.40L 52.50ft	Midspan 65.63ft	Depress 52.50ft
Vu-cmp,K	84.2	72.3	60.3	48.4	36.4	48.4
vh,psi	46.8	40.2	33.5	26.9	20.2	26.9
Surf,in2/F	360.00	360.00	360.00	360.00	360.00	360.00
Avhmin,in2/F	0.22	0.22	0.22	0.22	0.22	0.22
Max spc,in	24.00	24.00	24.00	24.00	24.00	24.00
Avh-sm,in2/F	0.22	0.22	0.22	0.22	0.22	0.22
Avh-rg,in2/F	0.22	0.22	0.22	0.22	0.22	0.22

ULTIMATE CAPACITY (f'c-eff = 4000 psi Tf-eff = 9.00 in Beta-1 = 0.85) :

	Trans 1.75ft	H/2 2.29ft	0.10L 13.13ft	0.20L 26.25ft	0.30L 39.38ft	0.40L 52.50ft	Midspan 65.63ft	Depress 52.50ft
A*s, in2	2.2132	2.6927	9.8900	9.8900	9.8900	9.8900	9.8900	9.8900
Ycg, in	11.36	11.29	10.02	8.48	6.93	5.39	5.39	5.39
p* (A*s/bd)	0.00106	0.00128	0.00458	0.00443	0.00429	0.00415	0.00415	0.00415
f*su, Ksi	260.4	258.3	228.3	229.6	230.9	232.2	232.2	232.2
a, in	3.57	4.31	13.99	14.08	14.16	14.23	14.23	14.23
Mu-req'd, Kft	338.7	441.6	2310.4	4089.7	5338.0	6081.8	6307.6	6081.8
Mu-prov'd,Kft	2009.2	2407.2	6915.8	7339.6	7690.2	8016.2	8016.2	8016.2

CRACKING LOAD (Art. 9.18.2.1 and 9.15.2.3. fcr = -785.6 psi) :
Cracking sec at 65.63 ft. Mu= 8016.2 Kft Mcr= 4271.5 Kft Ratio= 1.877 ok

CAMBER AND DEFLECTIONS : (Ref: PCI Design Handbook - 3rd Ed., Table 4.6.3)

	Release	Mult	Erection	Mult	Final
Prestress :	8.834	1.80	15.901	2.20	19.435
Self weight :	-4.670	1.85	-8.640	2.40	-11.208
SDL-Precast :			-0.123	3.00	-0.368
Topping weight :			-2.948	2.30	-6.780
SDL-Composite :			-0.658	3.00	-1.974
	-------		-------		-------
Total :	4.164		3.533		-0.894

Positive value indicates upward deflection

```
UNIVERSITY OF MINNESOTA                PHONE: (612) 625-3877  | SHEET   OF
500 PILLSBURY DRIVE S.E.   MINNEAPOLIS,  MN  55495-0220       | JOB NO DBDTEST
PROGRAM: SPAN  -V0.1 DEVELOPED BY LEAP SOFTWARE, INC TAMPA FL | DATE 04-11-1994
PHONE  : TOLL-FREE 1-800-451-LEAP ( TAMPA AREA 813-985-9170 ) | BY     tma
```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-------------------------------------------------------------------------------
State   : MN                           State Job No : NONE
Project : UMN TEST GIRDER - DEBONDED
Subtask : DESIGN:f'c=10.5, f'ci=8.925, f'cd=4, debonded 46/4/8

PRECAST DATA:                              GENERAL BRIDGE DATA:
-------------                              --------------------
Section Id: MN/DOT 45M                      System: Longit. Stringer
Type       : Special                       Bridge width =  52.00 ft
Flange width =  30.00 in                   Curb-to-curb =  48.00 ft
Flange thick =   6.00 in                   Beam Spacing =   4.00 ft
Stems:  No.  =   1                         Lane width   =  12.00 ft
        Top  =   6.00 in                   No. of lanes =    4
        Bot  =   6.00 in                   No. of beams =   13
Shear width  =   6.00 in                   Int./Ext.    =    I

TOPPING DATA:                              GENERAL LOAD DATA:
-------------                              ------------------
Topg:  Thick =  10.00 in                   SDL (on precast) =   20.0 plf
       Width =  30.00 in                   SDL (on comp.)   =  203.0 plf
Gap:   Thick =   9.00 in                   Impact (Moment)  =  1.195
       Width =  18.00 in                   Impact (Shear)   =  1.195 (Assumed)
Eff. width   =  48.00 in                   Axle = HS25+Military

LOAD EQUATIONS:  Gamma  Beta    Beta   D.F.  ( Art. 3.22.2 )
---------------
Service Load  =   1.00*(1.00*DL+1.00*0.364*(LL+I))  Group I
                  Group IA loading NOT considered
Factored Load =   1.30*(1.00*DL+1.67*0.364*(LL+I))  Group I

SECTION PROPERTIES:                        GENERAL SPAN DATA:
-------------------                        ------------------
                     PRECAST   COMPOSITE   Overall Length = 132.75 ft
                     -------   ---------   Release Span   = 132.75 ft
Area           =  624.0 in2    895.5 in2*  Design Span    = 131.25 ft
Total Height   =  45.00 in     55.00 in    Shoring : NONE
Mom of Inertia = 167048 in4   315745 in4
Ht. of c.g.    =  22.34 in     30.78 in
Density        =  155.0 pcf    150.0 pcf
Self-weight    =  671.7 plf    481.3 plf   MISCELLANEOUS:
At 28-Days:                                --------------
  Strength     =  10500 psi     4000 psi   Kern pts:   Upper = 34.32 in
  Max comp,top =   4200 psi     1600 psi               Lower = 10.53 in
  Max comp,bot =   4200 psi                Trans len mult,Bonded=1.0
  Max tens,top =   -283 psi     -190 psi                Debonded=1.0
  Max tens,bot =   -615 psi                Dev len mult,  Bonded=1.6
  Elasticity   = 6525.4 Ksi   3834.3 Ksi                Debonded=2.0
  Crack. tens. =   -786 psi
At Release:
  Strength     =   8925 psi
  Max comp,top =   5355 psi
  Max comp,bot =   5355 psi
  Max tens,top =   -200 psi
  Max tens,bot =      0 psi
  Elasticity   = 6016.1 Ksi

* Total transformed area using Ect/Ec = 0.5876
```

UNIVERSITY OF MINNESOTA PHONE: (612) 625-3877
500 PILLSBURY DRIVE S.E. MINNEAPOLIS, MN 55495-0220
PROGRAM: SPAN -V0.1 DEVELOPED BY LEAP SOFTWARE, INC TAMPA FL
PHONE : TOLL-FREE 1-800-451-LEAP (TAMPA AREA 813-985-9170)

SHEET OF
JOB NO DBDTEST
DATE 04-11-1994
BY tma

```
PRESTRESSED STEEL: 46 strands, 6/10 in-270K-Low-Lax
                   Depressed at .4L ( 53.25 ft from member end )
END PATTERN:      12 @  2.00 in  12 @  4.00 in  12 @  6.00 in   4 @  8.00 in
   2 @ 10.00 in   2 @ 32.00 in   2 @ 34.00 in
MID PATTERN:
(A) DRAPED   :     2 @ 12.00 in   2 @ 14.00 in
(B) BALANCE  :    12 @  2.00 in  12 @  4.00 in  12 @  6.00 in   4 @  8.00 in
   2 @ 10.00 in

Strand Dia         = 0.6000 in      Ult strength(f's) =    270 Ksi
Strand Area        = 0.2150 in2     Initial prestress = 0.750f's = 202.5 Ksi
Total strand area =  9.890 in2      Initial pull      = 2002.7 K = 43.54 K/str
Trans len,basic   =   2.50 ft       Dev len, basic    =   6.98 ft
Trans len,bonded  =   2.50 ft       Dev len, bonded   =  11.17 ft
Trans len,debonded=   2.50 ft       Dev len, debonded =  13.96 ft
-------------------------------------------------------------------------------
REINFORCING STEEL: Tension steel: fy = 60K  fs = 24K.  Shear steel: fsy = 60K
-------------------------------------------------------------------------------
Midspan: Strand area= 9.89 in2  Ycg= 5.39 in  P-init=2002.7 K  Ecc= 16.95 in
Hours to release = 24  Rel. Humid. (RH) = 75%  Es= 28,000 Ksi  Eci = 6,016 Ksi

AASHTO LOSSES:           Release                      Final      ( Art. 9.16.2 )
Steel relaxation    RET*  1760 psi   CRs (Eq 9-10)    890 psi
Elastic shortening ES    19353 psi   ES  (Eq 9-6)   19353 psi   ( Fcir= 4158 psi )
Concrete shrinkage                   SH  (Eq 9-4)    5750 psi
Concrete creep                       CRc (Eq 9-9)   37748 psi   ( Fcds=-1736 psi )
                         -----                      -----
Total                    21113 psi (10.4%)          63742 psi (31.5%)

* Steel relaxation before release - Ref: PCI Journal Vol 20, No.4, Jul-Aug 1975
-------------------------------------------------------------------------------
SHIELDING AND REDUCED INITIAL PULLS:
Group  Strands       Heights            Shielding          Initial Pull
                End(in)  Mid(in)    End(ft)  Mid(ft)     Frac    K/Str
  1      2       2.00     2.00       2.00     0.00      0.750    43.54
 21      2       2.00     2.00       4.00     0.00      0.750    43.54
 22      2       2.00     2.00      12.00     0.00      0.750    43.54
 23      2       2.00     2.00      20.00     0.00      0.750    43.54
-------------------------------------------------------------------------------
SERVICE LOAD MOMENTS:

                 Trans     H/2   0.10L   0.20L   0.30L   0.40L Midspan
                1.75ft  2.29ft 13.13ft 26.25ft 39.38ft 52.50ft 65.63ft

Self wt.  ,Kft    76.1    99.2   520.7   925.6  1214.9  1388.5  1446.3
DL-Prec.  ,Kft     2.3     3.0    15.5    27.6    36.2    41.3    43.1
Topping   ,Kft    54.5    71.1   373.1   663.2   870.5   994.8  1036.3
DL-Comp.  ,Kft    23.0    30.0   157.4   279.8   367.2   419.6   437.1
Eff. LL+I ,Kft    62.7    81.7   425.5   748.4   968.5  1098.2  1131.3
```

RELEASE STRESSES (Loss = 10.4%) :

	Sup-Rel -0.75ft	Trans 1.75ft	0.10L 13.13ft	0.20L 26.25ft	0.30L 39.38ft	0.40L 52.50ft	Depress 52.50ft	Crit 54.16ft
Prest.								
Precast-top	0.0	-503.4	-823.8	-1040.9	-1145.2	-1249.5	-1249.5	-1249.5
Bottom	0.0	5262.1	6211.0	6735.3	6838.1	6941.0	6941.0	6941.0
Self wt.								
Precast-top	0.0	178.0	901.7	1560.9	2031.7	2314.3	2314.3	2336.6
Bottom	0.0	-175.5	-888.9	-1538.8	-2003.0	-2281.6	-2281.6	-2303.6
Total								
Precast-top	0.0	-325.4*	77.9	520.0	886.5	1064.8	1064.8	1087.1
Bottom	0.0	5086.7	5322.1	5196.5	4835.1	4659.4	4659.4	4637.4
As-top, in2	NONE	0.550	NONE	NONE	NONE	NONE	NONE	NONE

FINAL STRESSES (Loss = 31.5%) :

	Sup-Fin 0.00ft	Trans 1.75ft	H/2 2.29ft	0.10L 13.13ft	0.20L 26.25ft	0.30L 39.38ft	0.40L 52.50ft	Midspan 65.63ft
Prest.								
Precast-top	-108.2	-385.1	-403.3	-630.2	-796.3	-876.1	-955.8	-955.8
Bottom	1189.0	4025.5	4084.6	4751.3	5152.4	5231.1	5309.8	5309.8
Self wt.								
Precast-top	0.0	123.9	161.6	847.5	1506.8	1977.6	2260.1	2354.3
Bottom	0.0	-122.1	-159.3	-835.6	-1485.5	-1949.7	-2228.2	-2321.1
DL-Prec.								
Precast-top	0.0	3.7	4.8	25.2	44.9	58.9	67.3	70.1
Bottom	0.0	-3.6	-4.7	-24.9	-44.2	-58.1	-66.3	-69.1
Topping								
Precast-top	0.0	88.8	115.8	607.3	1079.6	1417.0	1619.4	1686.9
Bottom	0.0	-87.5	-114.1	-598.7	-1064.3	-1397.0	-1596.5	-1663.0
DL-Comp.								
Topping-top	0.0	12.4	16.2	85.1	151.3	198.6	227.0	236.4
Precast-top	0.0	12.4	16.2	85.1	151.2	198.5	226.8	236.3
Bottom	0.0	-26.9	-35.1	-184.1	-327.2	-429.5	-490.9	-511.3
LL+I								
Topping-top	0.0	33.9	44.2	230.2	404.8	523.9	594.0	611.9
Precast-top	0.0	33.9	44.1	230.0	404.5	523.5	593.6	611.5
Bottom	0.0	-73.3	-95.5	-497.7	-875.4	-1132.9	-1284.6	-1323.3
Total								
Topping-top	0.0	46.3	60.4	315.3	556.1	722.5	821.0	848.4
Precast-top	-108.2	-122.4	-60.8	1164.9	2390.7	3299.4	3811.4	4003.1
Bottom	1189.0	3712.0	3675.9	2610.4	1355.7	263.9	-356.8	-578.1

VERTICAL SHEAR (Vu @ h/2 = 190.7 K E for fpc= 8.44 in Art. 9.20

	Sup-Fin	Trans	H/2	0.02L	0.04L	0.06L	0.08L
	0.00ft	1.75ft	2.29ft	2.63ft	5.25ft	7.88ft	10.50ft
Vd,K	90.3	87.9	87.1	86.7	83.1	79.5	75.8
Md,Kft	0.0	155.9	203.3	232.3	455.1	668.4	872.2
Ml,Kft	0.0	62.7	81.7	93.3	182.5	267.6	348.6
Vu,K	190.7	190.7	190.7	189.9	183.5	177.1	170.7
Mu,Kft	0.0	338.7	441.6	504.5	987.9	1449.9	1890.8
Mmax,Kft	0.0	182.8	238.3	272.3	532.8	781.5	1018.5
Vu @ Mu,K	196.3	192.0	190.7	189.9	183.5	177.1	170.7
Vi,K	106.0	104.1	103.5	103.2	100.4	97.6	94.8
fpe,psi	1189.0	4025.5	4084.6	4121.0	4459.1	4526.4	4542.2
fd,psi	0.0	-240.2	-313.2	-357.9	-701.1	-1029.8	-1343.8
Mcr,Kft	1542.1	3761.6	3749.7	3742.6	3738.3	3514.9	3259.8
d	46.8	46.9	47.0	47.0	47.3	47.4	47.6
Vci-com,K	10000.0	2247.2	1733.4	1522.5	805.0	536.0	396.9
Vci-min,K	48.9	49.1	49.1	49.2	49.4	49.6	49.7
Vci,K	10000.0	2247.2	1733.4	1522.5	805.0	536.0	396.9
fpc,psi	301.8	1089.4	1120.1	1138.9	1290.9	1402.1	1501.5
Vp,K	1.1	3.7	3.7	3.7	3.7	3.7	3.7
Vcw,K	127.3	196.8	199.6	201.3	215.3	225.5	234.7
Vc,K	127.3	196.8	199.6	201.3	215.3	225.5	234.7
Vs-reqd,K	84.5	15.1	12.2	9.6	0.0	0.0	0.0
Vs-max,K	230.3	230.9	231.2	231.3	232.5	233.3	234.0
Av-com,in2/F	0.36	0.06	0.05	0.04	0.00	0.00	0.00
Av-min,in2/F	0.06	0.06	0.06	0.06	0.06	0.06	0.06
Av,in2/F	0.36	0.06	0.06	0.06	0.06	0.06	0.06
Vs-crit,K	115.1	115.4	115.6	115.6	116.2	116.7	117.0
Max spc,in	24.00	24.00	24.00	24.00	24.00	24.00	24.00

	0.10L	0.20L	0.30L	0.40L	Midspan	Depress
	13.13ft	26.25ft	39.38ft	52.50ft	65.63ft	52.50ft
Vd,K	72.2	54.2	36.1	18.1	0.0	18.1
Md,Kft	1066.6	1896.2	2488.7	2844.3	2962.8	2844.3
Ml,Kft	425.5	748.4	968.5	1098.2	1131.3	1098.2
Vu,K	164.3	132.3	100.4	68.4	36.4	68.4
Mu,Kft	2310.4	4089.7	5338.0	6081.8	6307.6	6081.8
Mmax,Kft	1243.8	2193.6	2849.3	3237.5	3344.8	3237.5
Vu @ Mu,K	164.3	132.3	100.4	62.0	30.0	62.0
Vi,K	92.1	78.1	64.2	43.9	30.0	43.9
fpe,psi	4751.3	5152.4	5231.1	5309.8	5309.8	5309.8
fd,psi	-1643.2	-2921.3	-3834.2	-4382.0	-4564.5	-4382.0
Mcr,Kft	3182.7	2432.9	1719.8	1318.8	1162.7	1318.8
d	47.8	48.7	49.2	49.6	49.6	49.6
Vci-com,K	325.4	158.8	93.0	54.2	28.7	54.2
Vci-min,K	50.0	50.9	51.4	51.9	51.9	51.9
Vci,K	325.4	158.8	93.0	54.2	51.9	54.2
fpc,psi	1621.7	2063.6	2340.1	2494.1	2555.3	2494.1
Vp,K	3.7	3.7	3.7	3.7	0.0	3.7
Vcw,K	246.1	289.2	316.7	333.2	334.9	333.2
Vc,K	246.1	158.8	93.0	54.2	51.9	54.2
Vs-reqd,K	0.0	0.0	18.5	21.7	0.0	21.7
Vs-max,K	235.1	239.3	241.9	244.0	244.0	244.0
Av-com,in2/F	0.00	0.00	0.08	0.09	0.00	0.09
Av-min,in2/F	0.06	0.06	0.06	0.06	0.06	0.06
Av,in2/F	0.06	0.06	0.08	0.09	0.06	0.09
Vs-crit,K	117.6	119.7	120.9	122.0	122.0	122.0
Max spc,in	24.00	24.00	24.00	24.00	24.00	24.00

HORIZONTAL SHEAR (Q= 5265.6 in3 I= 315745 in4 b= 30.00 in) :

	Sup-Fin 0.00ft	Trans 1.75ft	H/2 2.29ft	0.02L 2.63ft	0.04L 5.25ft	0.06L 7.88ft	0.08L 10.50ft
Vu-cmp,K	96.2	94.6	94.1	93.8	91.4	89.0	86.6
vh,psi	53.5	52.6	52.3	52.1	50.8	49.5	48.2
Surf,in2/F	360.00	360.00	360.00	360.00	360.00	360.00	360.00
Avhmin,in2/F	0.22	0.22	0.22	0.22	0.22	0.22	0.22
Max spc,in	24.00	24.00	24.00	24.00	24.00	24.00	24.00
Avh-sm,in2/F	0.22	0.22	0.22	0.22	0.22	0.22	0.22
Avh-rg,in2/F	0.22	0.22	0.22	0.22	0.22	0.22	0.22

	0.10L 13.13ft	0.20L 26.25ft	0.30L 39.38ft	0.40L 52.50ft	Midspan 65.63ft	Depress 52.50ft
Vu-cmp,K	84.2	72.3	60.3	48.4	36.4	48.4
vh,psi	46.8	40.2	33.5	26.9	20.2	26.9
Surf,in2/F	360.00	360.00	360.00	360.00	360.00	360.00
Avhmin,in2/F	0.22	0.22	0.22	0.22	0.22	0.22
Max spc,in	24.00	24.00	24.00	24.00	24.00	24.00
Avh-sm,in2/F	0.22	0.22	0.22	0.22	0.22	0.22
Avh-rg,in2/F	0.22	0.22	0.22	0.22	0.22	0.22

ULTIMATE CAPACITY (f'c-eff = 4000 psi Tf-eff = 9.00 in Beta-1 = 0.85) :

	Trans 1.75ft	H/2 2.29ft	0.10L 13.13ft	0.20L 26.25ft	0.30L 39.38ft	0.40L 52.50ft	Midspan 65.63ft	Depress 52.50ft
A*s, in2	1.8437	2.2565	8.8975	9.6755	9.8900	9.8900	9.8900	9.8900
Ycg, in	8.06	8.00	7.20	6.34	5.82	5.39	5.39	5.39
p* (A*s/bd)	0.00082	0.00100	0.00388	0.00414	0.00419	0.00415	0.00415	0.00415
f*su, Ksi	262.5	260.9	234.7	232.2	231.8	232.2	232.2	232.2
a, in	3.00	3.65	12.94	13.93	14.21	14.23	14.23	14.23
Mu-req'd, Kft	338.7	441.6	2310.4	4089.7	5338.0	6081.8	6307.6	6081.8
Mu-prov'd,Kft	1832.4	2215.2	7126.0	7706.1	7925.5	8016.2	8016.2	8016.2

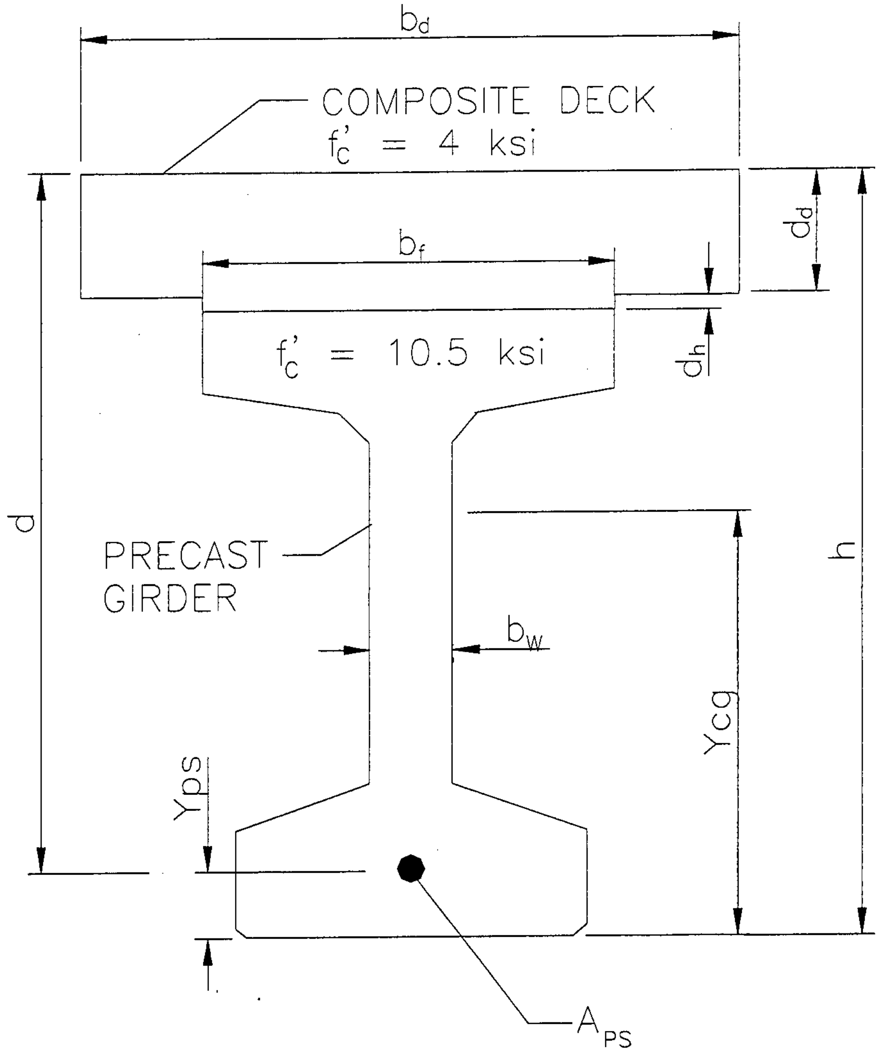
CRACKING LOAD (Art. 9.18.2.1 and 9.15.2.3. fcr = -785.6 psi) :
Cracking sec at 65.63 ft. Mu= 8016.2 Kft Mcr= 4271.5 Kft Ratio= 1.877 ok

CAMBER AND DEFLECTIONS : (Ref: PCI Design Handbook - 3rd Ed., Table 4.6.3)

	Release	Mult	Erection	Mult	Final
Prestress :	9.307	1.80	16.753	2.20	20.476
Self weight :	-4.670	1.85	-8.640	2.40	-11.208
SDL-Precast :			-0.123	3.00	-0.368
Topping weight :			-2.948	2.30	-6.780
SDL-Composite :			-0.658	3.00	-1.974
	-------		-------		-------
Total :	4.637		4.386		0.147

Positive value indicates upward deflection

Appendix B
Sample Calculations



GEOMETRIC PROPERTIES

GROSS CONCRETE SECTION ONLY
(DECK CONCRETE TRANSFORMED)

A = 1,047 in^2
I = 379,100 in
Ycg = 33.6 in^4

Test Girder Section Parameters
Figure B.1

SAMPLE CALCULATIONS FOR ULTIMATE FLEXURAL STRENGTH OF COMPOSITE TEST GIRDERS

Assumptions: Nominal material properties and strengths, and non-transformed section properties

See Figure B.1 on previous page for girder section parameters

$N := 46$ number of prestressing strands

$D := 0.228$ measured strand area, in^2

$A_{ps} := D \cdot N$ total area of strand $A_{ps} = 10.49$

$Y_{ps} := 5.13$ height to center of gravity of stands at midspan, in.

$f_{pu} := 270$ ultimate strand strength, ksi

$f_c := 10.5$ 28 day concrete strength of precast girder, ksi

$f_{cd} := 4$ 28 day concrete strength of composite deck, ksi

$h := 45 + 10$ total height of composite section, all dimensions are inches

$b_f := 30$ $d_h := 1$ $b_d := 48$ $d_d := 9$

$d := h - Y_{ps}$ $d = 49.87$

$\rho_p := \dfrac{A_{ps}}{(b_d \cdot d)}$ ratio of prestressing steel $\rho_p = 0.00438$

$\beta_1 := 0.85$ concrete strength factor

$\gamma_p := 0.28$ strand type factor

$f_{ps} := f_{pu} \cdot \left[1 - \dfrac{\gamma_p}{\beta_1} \cdot \left(\rho_p \cdot \dfrac{f_{pu}}{f_{cd}}\right)\right]$ average stress in strand at ultimate, ksi

$f_{ps} = 243.7$

$a := \frac{A_{ps} \cdot f_{ps}}{(0.85 \cdot f_{cd} \cdot b_d)}$ estimate of the depth of compression block, in.

$a = 15.66$ since the depth of the compression block is greater than the depth of the deck, analyze section as T-shape

$T := A_{ps} \cdot f_{ps}$ total tensile force in strands, kips $T = 2555.9$

calculate the average compressive forces in each compression block, kips

$C_1 := 0.85 \cdot f_{cd} \cdot b_d \cdot d_d$ in deck $C_1 = 1468.8$

$C_2 := 0.85 \cdot f_{cd} \cdot b_f \cdot d_h$ in haunch $C_2 = 102$

$y_f := \frac{T - C_1 - C_2}{0.85 \cdot f_c \cdot b_f}$ depth of compression block in precast top flange, in.

$y_f = 3.68$

$C_3 := 0.85 \cdot f_c \cdot b_f \cdot y_f$ in flange $C_3 = 985.1$

calculate ulimate flexural capacity

$\phi := 1.0$ capacity reduction factor, equals 1.0 for precast sections

$$\phi M_n := \phi \cdot C_1 \cdot \left(d - \frac{d_d}{2}\right) + C_2 \cdot \left(d - d_d - \frac{d_h}{2}\right) + C_3 \cdot \left(d - d_d - d_h - \frac{y_f}{2}\right)$$

$\frac{\phi M_n}{12} = 9018$ ft-kips

SAMPLE CALCULATIONS FOR ULTIMATE SHEAR STRENGTH OF COMPOSITE TEST GIRDERS

Assumptions: Nominal material properties and strengths, and section properties for deck concrete transformed to precast concrete (steel not transformed)

See Figure B.1 on previous page for girder section parameters

Calculate shear strength for test girder end with draped strands only (IID) at location of maximum shear: h/2 from support

$b_w := 6$ width of girder web, in.

$f_c := 10500$ 28 day concrete strength of precast girder, ksi

$I_c := 379100$ moment of inertia, in^4

$Y_{cg} := 33.6$ height to center of gravity of gross section, in.

the following parameters are from the structural analysis, see Appendix A for numerical values and Section 2.3.3.2.2 of the thesis for their description

$V_d := 87.1$ kips

$V_p := 13.4$ kips

$V_i := 103.5$ kips

$M_{max} := 238.3$ ft-kips

$f_{pe} := 4226$ psi

$f_d := 313$ psi

$f_{pc} := 1539$ psi

$$M_{cr} := \frac{I_c}{Y_{cg}} \cdot \frac{6 \cdot \sqrt{f_c} + f_{pe} - f_d}{12000}$$ cracking moment at section, ft-kips

$M_{cr} = 4257$

calculate shear strength of concrete section to resist flexure-shear cracking, kips

$$V_{ci} := \frac{0.6 \cdot \sqrt{f_c} \cdot b_w \cdot d}{1000} + V_d + V_i \cdot \frac{M_{cr}}{M_{max}} \qquad V_{ci} \geq \frac{1.7 \cdot \sqrt{f_c} \cdot b_w \cdot d}{1000}$$

$V_{ci} = 1955$

calculate shear strength of concrete section to resist web cracking, kips

$$V_{cw} := \frac{\left(3.5 \cdot \sqrt{f_c} + 0.3 \cdot f_{pc}\right) \cdot b_w \cdot d}{1000} + V_p$$

$V_{cw} = 259$

the nominal shear strength of the concrete section is the minimum of the two components:

$$V_c := \begin{pmatrix} V_{ci} \\ V_{cw} \end{pmatrix} \qquad V_c := \min\left(V_c\right)$$

$V_c = 259$ kips

Calculate shear strength for test girder ends with draped and debonded strands (IA, IB and IIC) at location of maximum shear: h/2 from support

$V_p := 3.7$ kips

$f_{pe} := 4085$ psi

$f_{pc} := 1120$ psi

$$M_{cr} := \frac{I_c}{Y_{cg}} \cdot \frac{6 \cdot \sqrt{f_c} + f_{pe} - f_d}{12000}$$

$M_{cr} = 4125$ ft-kips

$$V_{ci} := \frac{0.6 \cdot \sqrt{f_c} \cdot b_w \cdot d}{1000} + V_d + V_i \cdot \frac{M_{cr}}{M_{max}} \qquad V_{ci} \geq \frac{1.7 \cdot \sqrt{f_c} \cdot b_w \cdot d}{1000}$$

$V_{ci} = 1897$ kips

$$V_{cw} := \frac{\left(3.5 \cdot \sqrt{f_c} + 0.3 \cdot f_{pc}\right) \cdot b_w \cdot d}{1000} + V_p$$

$V_{cw} = 212$ kips

$$V_c := \begin{pmatrix} V_{ci} \\ V_{cw} \end{pmatrix} \qquad V_c := \min\left(V_c\right)$$

$V_c = 212$ kips

Appendix C

HSC Batch Data for Test Girders

CONCRETE BATCH MIXES USED IN TEST GIRDERS BY ELK RIVER CONCRETE

MATERIAL ITEM KEY:
751 = COARSE AGGREGATE
SD3 = FINE AGGREGATE (SAND)
CEM = TYPE III CEMENT
W19 = WATER REDUCING AGENT
WAT = WATER

LOAD NUMBER OUT OF SEQUENCE INDICATES REJECTION OF THAT CONCRETE BATCH

MIX: LIMESTONE
DATE: 8/9/93
TIME: 13:46

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
2	3	4

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5280	1	5880	5920	5860.8	58.6
SD3 [LB]	3880	4.2	4060	4000	3832.0	142.0
CEM [LB]	2250	0	2250	2250	2250.0	0.0
W19 [OZ]	450	0	450	450	0.0	0.0
WAT [GAL]	90	0	73	70	97.3	583.1
					TOTAL	783.7

ACTUAL WAT/CEM RATIO: 0.34
WATER TEMPERATURE [F] : 70

MIX: LIMESTONE
DATE: 8/9/93
TIME: 13:53

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
3	3	4

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5280	1	5880	5920	5860.8	58.6
SD3 [LB]	3880	4.2	4060	4000	3832.0	142.0
CEM [LB]	2250	0	2250	2240	2240.0	0.0
W19 [OZ]	450	0	450	450	0.0	0.0
WAT [GAL]	90	0	73	69	96.3	574.8
					TOTAL	775.4

ACTUAL WAT/CEM RATIO: 0.34
WATER TEMPERATURE [F] : 69

MIX: LIMESTONE
DATE: 8/9/93
TIME: 13:55

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
4	3	3.5

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5280	1	5880	5900	5841.0	58.4
SD3 [LB]	3880	4.2	4060	4020	3851.2	142.7
CEM [LB]	2250	0	2250	2240	2240.0	0.0
W19 [OZ]	450	0	450	450	0.0	0.0
WAT [GAL]	90	0	72	66	93.3	549.8
					TOTAL	750.9

ACTUAL WAT/CEM RATIO: 0.33
WATER TEMPERATURE [F] : 66

MIX: LIMESTONE
DATE: 8/9/93
TIME: 14:01

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
5	3	3

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5280	1	5880	5880	5821.2	58.2
SD3 [LB]	3880	4.2	4060	4080	3908.6	144.9
CEM [LB]	2250	0	2250	2255	2255.0	0.0
W19 [OZ]	450	0	450	450	0.0	0.0
WAT [GAL]	90	0	70	60	87.6	499.8
					TOTAL	702.9

ACTUAL WAT/CEM RATIO: 0.31
WATER TEMPERATURE [F] : 60

MIX: LIMESTONE
DATE: 8/9/93
TIME: 14:05

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
6	3	0

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5280	1	5880	5860	5801.4	58.0
SD3 [LB]	3880	4.2	4060	4080	3908.6	144.9
CEM [LB]	2250	0	2250	2240	2240.0	0.0
W19 [OZ]	450	0	450	450	0.0	0.0
WAT [GAL]	90	0	61	59	86.6	491.5
					TOTAL	694.4

ACTUAL WAT/CEM RATIO: 0.3
WATER TEMPERATURE [F] : 60

MIX: LIMESTONE
DATE: 8/9/93
TIME: 14:12

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
7	3	-1

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5280	1	5880	5860	5801.4	58.0
SD3 [LB]	3880	4.2	4060	4080	3908.6	144.9
CEM [LB]	2250	0	2250	2260	2260.0	0.0
W19 [OZ]	450	0	450	450	0.0	0.0
WAT [GAL]	90	0	58	58	85.6	483.1
					TOTAL	686.0

ACTUAL WAT/CEM RATIO: 0.3
WATER TEMPERATURE [F] : 63

MIX:	LIMESTONE
DATE:	8/9/93
TIME:	14:16

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
8	3	-1

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5280	1	5880	5860	5801.4	58.0
SD3 [LB]	3880	4.2	4020	4080	3949.4	106.7
CEM [LB]	2250	0	2250	2280	2280.0	0.0
W19 [OZ]	450	0	450	450	0.0	0.0
WAT [GAL]	90	0	63	63	85.7	524.8
					TOTAL	689.5

ACTUAL WAT/CEM RATIO:	0.3
WATER TEMPERATURE [F] :	70

MIX:	LIMESTONE
DATE:	8/9/93
TIME:	14:24

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
1	3	0

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5280	1	5880	5900	5841.0	58.4
SD3 [LB]	3880	4.2	4020	3960	3825.4	111.1
CEM [LB]	2250	0	2250	2280	2280.0	0.0
W19 [OZ]	450	0	450	450	0.0	0.0
WAT [GAL]	90	0	67	67	90.2	558.1
					TOTAL	727.6

ACTUAL WAT/CEM RATIO:	0.31
WATER TEMPERATURE [F] :	77

MIX: GLACIAL GRAVEL W/ MICRO SILICA
DATE: 8/9/93
TIME: 14:54

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
2	3	1

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5620	1	5660	5640	5583.6	55.8
SD3 [LB]	3740	3.6	3880	3860	3721.0	115.5
CEM [LB]	2080	0	2080	2085	2085.0	0.0
W19 [OZ]	370	0	370	370	0.0	0.0
WAT [GAL]	66	0	45	45	68.4	374.9
					TOTAL	546.2

ACTUAL WAT/CEM RATIO: 0.26
WATER TEMPERATURE [F] : 45

MIX: GLACIAL GRAVEL W/ MICRO SILICA
DATE: 8/9/93
TIME: 15:02

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
3	3	1

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5620	1	5660	5700	5643.0	56.4
SD3 [LB]	3740	1.7	3800	3760	3696.0	44.4
CEM [LB]	2080	0	2080	2095	2095.0	0.0
W19 [OZ]	370	0	370	370	0.0	0.0
WAT [GAL]	66	0	54	46	60.5	383.2
					TOTAL	484.0

ACTUAL WAT/CEM RATIO: 0.23
WATER TEMPERATURE [F] : 46

MIX: GLACIAL GRAVEL W/ MICRO SILICA
DATE: 8/9/93
TIME: 15:05

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
4	3	1

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5620	1	5660	5640	5583.6	55.8
SD3 [LB]	3740	1.7	3800	3780	3715.7	44.6
CEM [LB]	2080	0	2080	2080	2080.0	0.0
W19 [OZ]	370	0	370	370	0.0	0.0
WAT [GAL]	66	0	55	44	58.5	366.5
					TOTAL	467.0

ACTUAL WAT/CEM RATIO: 0.22
WATER TEMPERATURE [F] : 44

MIX: GLACIAL GRAVEL W/ MICRO SILICA
DATE: 8/9/93
TIME: 15:11

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
5	3	-2.5

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5620	1	5660	5680	5623.2	56.2
SD3 [LB]	3740	1.7	3800	3760	3696.1	44.4
CEM [LB]	2080	0	2080	2075	2075.0	0.0
W19 [OZ]	370	0	370	370	0.0	0.0
WAT [GAL]	66	0	44	44	58.5	366.5
					TOTAL	467.1

ACTUAL WAT/CEM RATIO: 0.22
WATER TEMPERATURE [F] : 44

MIX: GLACIAL GRAVEL W/ MICRO SILICA
DATE: 8/9/93
TIME: 15:16

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
6	3	-3

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5620	1	5660	5740	5682.6	56.8
SD3 [LB]	3740	1.7	3800	3760	3696.1	44.4
CEM [LB]	2080	0	2080	2070	2070.0	0.0
W19 [OZ]	370	0	370	370	0.0	0.0
WAT [GAL]	66	0	43	43	57.6	358.2
					TOTAL	459.4

ACTUAL WAT/CEM RATIO: 0.22
WATER TEMPERATURE [F] : 43

MIX: GLACIAL GRAVEL W/ MICRO SILICA
DATE: 8/9/93
TIME: 15:20

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
7	3	-3

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5620	1	5660	5700	5643.0	56.4
SD3 [LB]	3740	1.7	3800	3780	3715.7	44.7
CEM [LB]	2080	0	2080	2100	2100.0	0.0
W19 [OZ]	370	0	370	370	0.0	0.0
WAT [GAL]	66	0	43	43	57.6	358.2
					TOTAL	459.2

ACTUAL WAT/CEM RATIO: 0.21
WATER TEMPERATURE [F] : 43

MIX: GLACIAL GRAVEL W/ MICRO SILICA
DATE: 8/9/93
TIME: 15:25

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
8	3	-3

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5620	1	5660	5660	5603.4	56.0
SD3 [LB]	3740	1.7	3800	3840	3774.7	45.0
CEM [LB]	2080	0	2080	2080	2080.0	0.0
W19 [OZ]	370	0	370	370	0.0	0.0
WAT [GAL]	66	0	43	43	57.6	358.2
					TOTAL	459.5

ACTUAL WAT/CEM RATIO: 0.22
WATER TEMPERATURE [F] : 43

MIX: GLACIAL GRAVEL W/ MICRO SILICA
DATE: 8/9/93
TIME: 15:59

LOAD#	YDS. THIS LOAD	ADDED TRIM WATER [GAL/YD]
10	3	-3

ITEM	MIX DESIGN	MOISTURE CONTENT	DESIGN AMOUNT	ACTUAL AMOUNT	SSD CORRECTION	FREE WATER
751 [LB]	5620	1	5660	5660	5603.4	56.0
SD3 [LB]	3740	1.7	3800	3860	3794.4	45.5
CEM [LB]	2080	0	2080	2085	2085.0	0.0
W19 [OZ]	370	0	370	370	0.0	0.0
WAT [GAL]	66	0	43	43	57.7	358.2
					TOTAL	459.8

ACTUAL WAT/CEM RATIO: 0.22
WATER TEMPERATURE [F] : 43

Appendix D - Description and Locations of Instrumentation used in Precast Girders

TEST GIRDER INSTRUMENTATION LOCATION AND DESCRIPTION

Gage Designation:

Test phase - girder and end/slab - Gage #

Test Phase:
R = release end condition - bursting, splitting, displacement transducer
T = Transfer length strand gages
F = Flexure gages on strands and in concrete
S = Shear gages on stirrups and in concrete

Instrumentation Types, By Function:

Bursting / Stirrup Gage: TML Type WFLA-3
Transfer Length / Strand Gage: TML TypeFLK-1
PML Concrete Gage: TML Type PML-60
Concrete Rosette: TML Type PMR-60

Location:

X dimension is from the designated end
Y dimension is from the longitudinal centerline of girder as located on the bed, East = +, West = -
Z dimension is from the bottom of the girder, height above prestressing bed

GIRDER I - END A (in girder only, not slab)

Gage Designation	Gage Description	Design Location			Actual Field Location			
		X	Y	Z	X	Y	Z	
		(in)	(in)	(in)	(in)	(in)	(in)	
R-IA-D1	Displacement Transducer	15.00	13.00	4.67	15.00	13.00	4.65	
R-IA-B1	Bursting, G605E, east leg	2.00	1.70	11.50	3.00	1.70	11.00	
R-IA-B2	Bursting, G605E, west leg	2.00	-1.70	11.50	3.00	-1.70	11.00	
R-IA-B3	Bursting, G604E, west leg	8.00	-1.70	11.50	8.00	-1.70	11.50	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
T-IA-25A	Transfer length 2', debonded = 15"	3.25	3.00	2.00	3.33	3.00	2.25	
T-IA-28A	Transfer length 2', debonded = 15"	3.25	-3.00	2.00	3.12	-1.00	2.25	Strand 27 gaged
T-IA-25B	Transfer length 2', debonded = 22"	3.83	3.00	2.00	4.05	3.00	2.25	
T-IA-28B	Transfer length 2', debonded = 22"	3.83	-3.00	2.00	3.80	-1.00	2.25	Strand 27 gaged
T-IA-25C	Transfer length 2', debonded = 30"	4.50	3.00	2.00	4.55	3.00	2.25	
T-IA-28C	Transfer length 2', debonded = 30"	4.50	-3.00	2.00	4.44	-1.00	2.25	Strand 27 gaged

T-IA-44A	Transfer length 4', debonded = 15"	5.25	5.00	2.00	5.40	5.00	2.25	
T-IA-49A	Transfer length 4', debonded = 15"	5.25	-5.00	2.00	5.24	-5.00	2.25	
T-IA-44C	Transfer length 4', debonded = 30"	6.50	5.00	2.00	6.50	5.00	2.25	
T-IA-49C	Transfer length 4', debonded = 30"	6.50	-5.00	2.00	6.40	-5.00	2.25	
T-IA-123A	Transfer length 12', debonded = 15"	13.25	7.00	2.00	13.20	7.00	2.00	
T-IA-1210A	Transfer length 12', debonded = 15"	13.25	-7.00	2.00	13.26	-7.00	2.00	
T-IA-123B	Transfer length 12', debonded = 22"	13.83	7.00	2.00	13.80	7.00	2.00	
T-IA-1210B	Transfer length 12', debonded = 22"	13.83	-7.00	2.00	13.80	-7.00	2.00	
T-IA-123C	Transfer length 12', debonded = 30"	14.50	7.00	2.00	14.60	7.00	2.00	
T-IA-1210C	Transfer length 12', debonded = 30"	14.50	-7.00	2.00	14.50	-7.00	2.00	
T-IA-202A	Transfer length 20', debonded = 15"	21.25	9.00	2.00	21.25	9.00	2.00	
T-IA-2011A	Transfer length 20', debonded = 15"	21.25	-9.00	2.00	21.36	-9.00	2.00	
T-IA-202B	Transfer length 20', debonded = 22"	21.83	9.00	2.00	21.90	9.00	2.00	
T-IA-2011B	Transfer length 20', debonded = 22"	21.83	-9.00	2.00	21.95	-9.00	2.00	
T-IA-202C	Transfer length 20', debonded = 30"	22.50	9.00	2.00	22.50	9.00	2.00	
T-IA-2011C	Transfer length 20', debonded = 30"	22.50	-9.00	2.00	22.45	-9.00	2.00	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
V-IA-1	@ CG strands in transfer region	1.25	0.00	4.67	1.30		5.75	no centerline dimension recorded
V-IA-2	top flange over support	0.63	0.00	44.00	0.65		40.25	
V-IA-3	bottom flange @.45L, 2"	59.73	0.00	2.00	59.45		2.75	
V-IA-4	bottom flange @.45L, cg strands	59.73	0.00	5.39	59.45		5.00	
V-IA-5	bottom flange @.5L, 2"	66.38	0.00	2.00	66.40		2.50	
V-IA-6	bottom flange @.5L, cg strands	66.38	0.00	5.39	66.40		4.75	
V-IA-7	top flange @.5L	66.38	0.00	44.00	66.40		42.50	
		(in)	(in)	(in)	(in)	(in)	(in)	
R-IA-P1	PML over support for rupture	7.50	0.00	9.50	6.50		10.00	no centerline dimension recorded
R-IA-P2	PML over support for rupture	7.50	0.00	12.50	6.50		12.50	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
F-IA-P21	PML-60 @.2L - Center Line of X-Section	26.54	0.00	38.00	26.60	0.00	35.75	
F-IA-P22	PML-60 @.2L - Center Line of X-Section	26.54	0.00	44.00	26.60	0.00	44.25	
F-IA-P23	PML-60 @.2L - 3" from edge of flange	26.54	-12.00	44.00	26.60	-10.75	44.25	y in closer due to top steel
F-IA-P31	PML-60 @.3L - Center Line of X-Section	39.82	0.00	38.00	40.00	0.00	38.00	
F-IA-P32	PML-60 @.3L - Center Line of X-Section	39.82	0.00	44.00	40.00	0.00	44.00	
F-IA-P33	PML-60 @.3L - 3" from edge of flange	39.82	-12.00	44.00	40.00	-11.00	44.00	y in closer due to top steel

F-IA-P51	PML-60 @ .5L - cL of X-Section	66.38	0.00	25.00	66.50	0.00	25.25	
F-IA-P52	PML-60 @ .5L - cL of X-Section	66.38	0.00	35.00	66.50	0.00	35.00	
F-IA-P53	PML-60 @ .5L - cL of X-Section	66.38	0.00	38.00	66.50	0.00	38.00	
F-IA-P54	PML-60 @ .5L - cL of X-Section	66.38	0.00	41.00	66.50	0.00	41.00	
F-IA-P55	PML-60 @ .5L - cL of X-Section	66.38	0.00	44.00	66.50	0.00	43.50	
F-IA-P56	PML-60 @ .5L - 3" from edge of flange	66.38	-12.00	44.00	66.50	-8.75	43.50	y in closer due to lap splice
		(feet)	(in)	(in)	(feet)	(in)	(in)	
F-IA-12	Strand gage @ 0.45L	59.74	9.00	2.00	59.85	9.00	2.50	gages placed at top of strand
F-IA-15	Strand gage @ 0.45L	59.74	3.00	2.00	59.87	3.00	2.50	no apparent twist due to tensioning
F-IA-17	Strand gage @ 0.45L	59.74	-1.00	2.00	59.90	-1.00	2.50	y distance not measured
F-IA-19	Strand gage @ 0.45L	59.74	-5.00	2.00	59.66	-5.00	2.50	
F-IA-25	Strand gage @ 0.45L	59.74	3.00	4.00	59.65	3.00	4.00	
F-IA-35	Strand gage @ 0.45L	59.74	3.00	6.00	59.50	3.00	6.00	
F-IA-C12	Strand gage @ .5L	66.38	9.00	2.00	66.25	9.00	2.50	
F-IA-C15	Strand gage @ .5L	66.38	3.00	2.00	66.35	3.00	2.50	
F-IA-C17	Strand gage @ .5L	66.38	-1.00	2.00	66.80	-1.00	2.50	
F-IA-C19	Strand gage @ .5L	66.38	-5.00	2.00	66.40	-5.00	2.50	
F-IA-C25	Strand gage @ .5L	66.38	3.00	4.00	66.35	3.00	4.00	
F-IA-C35	Strand gage @ .5L	66.38	3.00	6.00	66.30	3.00	6.00	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
S-IA-R1A	Concrete Rosette - O degrees	2.92	0.00	22.50	2.67	1.25	22.50	on inside of east leg of stirrup
S-IA-R1B	Concrete Rosette - 45 degrees	2.92	0.00	22.50	2.67	1.25	22.50	all placed with hand level
S-IA-R1C	Concrete Rosette - 90 degrees	2.92	0.00	22.50	2.67	1.25	22.50	
S-IA-R2A	Concrete Rosette - O degrees	5.58	0.00	22.50	5.40	1.25	21.50	
S-IA-R2B	Concrete Rosette - 45 degrees	5.58	0.00	22.50	5.40	1.25	21.50	
S-IA-R2C	Concrete Rosette - 90 degrees	5.58	0.00	22.50	5.40	1.25	21.50	
S-IA-R3A	Concrete Rosette - O degrees	8.25	0.00	22.50	7.95	1.25	21.88	
S-IA-R3B	Concrete Rosette - 45 degrees	8.25	0.00	22.50	7.95	1.25	21.88	
S-IA-R3C	Concrete Rosette - 90 degrees	8.25	0.00	22.50	7.95	1.25	21.88	
S-IA-R4A	Concrete Rosette - O degrees	10.92	0.00	22.50	10.65	1.25	22.00	
S-IA-R4B	Concrete Rosette - 45 degrees	10.92	0.00	22.50	10.65	1.25	22.00	
S-IA-R4C	Concrete Rosette - 90 degrees	10.92	0.00	22.50	10.65	1.25	22.00	
S-IA-R5A	Concrete Rosette - O degrees	17.58	0.00	22.50	17.62	1.25	21.38	
S-IA-R5B	Concrete Rosette - 45 degrees	17.58	0.00	22.50	17.62	1.25	21.38	
S-IA-R5C	Concrete Rosette - 90 degrees	17.58	0.00	22.50	17.62	1.25	21.38	

		(feet)	(in)	(in)	(feet)	(in)	(in)	
S-IA-S1B	Stirrup gages - one leg only	2.25	1.50	22.50	2.10	1.50	22.25	east leg
S-IA-S1C	Stirrup gages - one leg only	2.25	1.50	11.50	2.10	1.50	11.00	e
S-IA-S3B	Stirrup gages - one leg only	4.92	1.50	22.50	4.90	-1.50	21.50	w
S-IA-S3C	Stirrup gages - one leg only	4.92	1.50	11.50	4.90	-1.50	11.00	w
S-IA-S4A	Stirrup gages - one leg only	6.25	1.50	35.00	6.15	-1.50	35.00	w
S-IA-S4B	Stirrup gages - one leg only	6.25	1.50	22.50	6.15	-1.50	22.25	w
S-IA-S4C	Stirrup gages - one leg only	6.25	1.50	11.50	6.15	-1.50	11.00	w
S-IA-S5A	Stirrup gages - one leg only	7.58	1.50	35.00	7.40	1.50	34.75	e
S-IA-S5B	Stirrup gages - one leg only	7.58	1.50	22.50	7.40	1.50	22.50	e
S-IA-S5C	Stirrup gages - one leg only	7.58	1.50	11.50	7.40	1.50	11.00	e
S-IA-S6A	Stirrup gages - one leg only	8.92	1.50	35.00	8.75	1.50	35.00	e
S-IA-S6B	Stirrup gages - one leg only	8.92	1.50	22.50	8.75	1.50	22.00	e
S-IA-S6C	Stirrup gages - one leg only	8.92	1.50	11.50	8.75	1.50	11.00	e
S-IA-S7A	Stirrup gages - one leg only	10.25	1.50	35.00	10.15	-1.50	35.00	w
S-IA-S7B	Stirrup gages - one leg only	10.25	1.50	22.50	10.15	-1.50	22.50	w
S-IA-S8A	Stirrup gages - one leg only	11.58	1.50	35.00	11.40	-1.50	35.00	w
S-IA-S8B	Stirrup gages - one leg only	11.58	1.50	22.50	11.40	-1.50	22.50	w
S-IA-S12A	Stirrup gages - one leg only	16.92	1.50	35.00	16.80	1.50	34.00	e
S-IA-S12B	Stirrup gages - one leg only	16.92	1.50	22.50	16.80	1.50	22.00	e
S-IA-S12C	Stirrup gages - one leg only	16.92	1.50	11.50	16.80	1.50	10.50	e
S-IA-S13A	Stirrup gages - one leg only	18.25	1.50	35.00	18.20	1.50	35.00	e
S-IA-S13B	Stirrup gages - one leg only	18.25	1.50	22.50	18.20	1.50	23.00	e
S-IA-S13C	Stirrup gages - one leg only	18.25	1.50	11.50	18.20	1.50	11.00	e
S-IA-S15A	Stirrup gages - one leg only	20.92	1.50	35.00	20.80	1.50	34.50	e
S-IA-S15B	Stirrup gages - one leg only	20.92	1.50	22.50	20.80	1.50	22.50	e
S-IA-S18A	Stirrup gages - one leg only	24.92	1.50	35.00	24.90	1.50	35.00	e
S-IA-S18B	Stirrup gages - one leg only	24.92	1.50	22.50	24.90	1.50	22.50	e
S-IA-S18C	Stirrup gages - one leg only	24.92	1.50	11.50	24.90	1.50	11.00	e
S-IA-S22B	Stirrup gages - one leg only	30.25	1.50	22.50	30.25	1.50	21.50	e
S-IA-S22C	Stirrup gages - one leg only	30.25	1.50	11.50	30.25	1.50	10.50	e

Other notes for End A:
Harp point at 53.1' from end
4 strands draped/ 8 strands debonded

GIRDER I - END B (in girder only, not slab)

Gage Designation	Gage Description	Design Location X	Design Location Y	Design Location Z	Actual Field Location X	Actual Field Location Y	Actual Field Location Z	
		(in)	(in)	(in)	(in)	(in)	(in)	
R-IB-D1	Displacement Transducer	15.00	13.00	4.67	15.00	13.00	4.65	
R-IB-B1	Bursting, G605E, east leg	2.00	1.70	11.50	2.50	1.70	10.25	
R-IB-B2	Bursting, G605E, west leg	2.00	-1.70	11.50	2.50	-1.70	10.25	
R-IB-B3	Bursting, G604E, east leg	8.00	1.70	11.50	7.50	1.70	11.25	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
T-IB-035A	Transfer length 0', 15"	1.25	3.00	6.00	1.00	3.00	6.00	y distance not measured
T-IB-07A	Transfer length 0', 15"	1.25	-1.00	2.00	1.00	-1.00	2.25	
T-IB-035B	Transfer length 0', 22"	1.83	3.00	6.00	1.77	3.00	6.00	
T-IB-07B	Transfer length 0', 22"	1.83	-1.00	2.00	1.61	-1.00	2.25	
T-IB-035C	Transfer length 0', 30"	2.50	3.00	6.00	2.46	3.00	6.00	
T-IB-07C	Transfer length 0', 30"	2.50	-1.00	2.00	2.40	-1.00	2.25	
T-IB-25A	Transfer length 2', debonded = 15"	3.25	3.00	2.00	3.10	3.00	2.25	
T-IB-28A	Transfer length 2', debonded = 15"	3.25	-3.00	2.00	3.20	-3.00	2.25	
T-IB-25B	Transfer length 2', debonded = 22"	3.83	3.00	2.00	3.70	3.00	2.25	
T-IB-28B	Transfer length 2', debonded = 22"	3.83	-3.00	2.00	3.72	-3.00	2.25	
T-IB-25C	Transfer length 2', debonded = 30"	4.50	3.00	2.00	4.30	3.00	2.25	
T-IB-28C	Transfer length 2', debonded = 30"	4.50	-3.00	2.00	4.33	-3.00	2.25	
T-IB-44A	Transfer length 4', debonded = 15"	5.25	5.00	2.00	5.18	5.00	2.25	
T-IB-49A	Transfer length 4', debonded = 15"	5.25	-5.00	2.00	5.20	-5.00	2.25	
T-IB-44B	Transfer length 4', debonded = 22"	5.83	5.00	2.00	5.68	5.00	2.25	
T-IB-49B	Transfer length 4', debonded = 22"	5.83	-5.00	2.00	5.60	-5.00	2.25	
T-IB-44C	Transfer length 4', debonded = 30"	6.50	5.00	2.00	6.47	5.00	2.25	
T-IB-49C	Transfer length 4', debonded = 30"	6.50	-5.00	2.00	6.22	-5.00	2.25	
T-IB-123A	Transfer length 12', debonded = 15"	13.25	7.00	2.00	13.26	7.00	2.25	
T-IB-1210A	Transfer length 12', debonded = 15"	13.25	-7.00	2.00	13.35	-7.00	2.25	
T-IB-123B	Transfer length 12', debonded = 22"	13.83	7.00	2.00	13.71	7.00	2.25	
T-IB-1210B	Transfer length 12', debonded = 22"	13.83	-7.00	2.00	13.88	-7.00	2.25	
T-IB-123C	Transfer length 12', debonded = 30"	14.50	7.00	2.00	14.45	7.00	2.25	

T-IB-1210C	Transfer length 12’, debonded = 30"	14.50	-7.00	2.00	14.48	-7.00	2.25	
T-IB-202A	Transfer length 20’, debonded = 15"	21.25	9.00	2.00	21.01	9.00	2.13	
T-IB-2011A	Transfer length 20’, debonded = 15"	21.25	-9.00	2.00	21.20	-9.00	2.13	
T-IB-202B	Transfer length 20’, debonded = 22"	21.83	9.00	2.00	21.82	9.00	2.13	
T-IB-2011B	Transfer length 20’, debonded = 22"	21.83	-9.00	2.00	21.80	-9.00	2.13	
T-IB-202C	Transfer length 20’, debonded = 30"	22.50	9.00	2.00	22.35	9.00	2.13	
T-IB-2011C	Transfer length 20’, debonded = 30"	22.50	-9.00	2.00	22.55	-9.00	2.13	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
V-IB-1	@ CG strands in transfer region	1.25	0.00	4.67	1.40		5.50	no centerline dimension recorded
V-IB-2	top flange over support	0.63	0.00	44.00	0.67		38.25	hairpin in way at 44"
V-IB-3	bottom flange @.45L, 2"	59.73	0.00	2.00	59.40		2.50	
V-IB-4	bottom flange @.45L, cg strands	59.73	0.00	5.39	59.40		4.75	
		(in)	(in)	(in)	(in)	(in)	(in)	
R-IB-P1	PML over support for rupture	7.50	0.00	9.50	7.00		9.75	no centerline dimension recorded
R-IB-P2	PML over support for rupture	7.50	0.00	12.50	7.00		12.25	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
F-IB-P21	PML-60 @.2L - Center Line of X-Section	26.54	0.00	38.00	27.20	0.00	38.50	x at stirrup, relocate
F-IB-P22	PML-60 @.2L - Center Line of X-Section	26.54	0.00	44.00	27.20	0.00	44.00	
F-IB-P23	PML-60 @.2L - 3" from edge of flange	26.54	-12.00	44.00	27.20	-9.00	44.00	y moved due to top steel location
F-IB-P31	PML-60 @.3L - Center Line of X-Section	39.83	0.00	38.00	40.60	0.00	38.50	
F-IB-P32	PML-60 @.3L - Center Line of X-Section	39.83	0.00	44.00	40.60	0.00	44.25	
F-IB-P33	PML-60 @.3L - 3" from edge of flange	39.83	-12.00	44.00	40.60	-9.00	44.25	y moved due to top steel location
		(feet)	(in)	(in)	(feet)	(in)	(in)	
F-IB-12	Strand gage @ 0.45L	59.74	9.00	2.00	59.75	9.00	2.50	gages placed at top of strand
F-IB-15	Strand gage @ 0.45L	59.74	3.00	2.00	59.90	3.00	2.50	no apparent twist due to tensioning
F-IB-17	Strand gage @ 0.45L	59.74	-1.00	2.00	59.95	-1.00	2.50	y distance not measured
F-IB-19	Strand gage @ 0.45L	59.74	-5.00	2.00	59.90	-5.00	2.50	
F-IB-25	Strand gage @ 0.45L	59.74	3.00	4.00	59.85	3.00	4.00	
F-IB-35	Strand gage @ 0.45L	59.74	3.00	6.00	59.88	3.00	6.00	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
S-IB-R1A	Concrete Rosette - O degrees	2.92	0.00	22.50	2.85	1.25	23.13	on inside of east leg of stirrup
S-IB-R1B	Concrete Rosette - 45 degrees	2.92	0.00	22.50	2.85	1.25	23.13	all placed with hand level
S-IB-R1C	Concrete Rosette - 90 degrees	2.92	0.00	22.50	2.85	1.25	23.13	

S-IB-R2A	Concrete Rosette - O degrees	5.58	0.00	22.50	5.50	1.25	22.75	
S-IB-R2B	Concrete Rosette - 45 degrees	5.58	0.00	22.50	5.50	1.25	22.75	
S-IB-R2C	Concrete Rosette - 90 degrees	5.58	0.00	22.50	5.50	1.25	22.75	
S-IB-R3A	Concrete Rosette - O degrees	8.25	0.00	22.50	8.17	1.25	22.75	
S-IB-R3B	Concrete Rosette - 45 degrees	8.25	0.00	22.50	8.17	1.25	22.75	
S-IB-R3C	Concrete Rosette - 90 degrees	8.25	0.00	22.50	8.17	1.25	22.75	
S-IB-R4A	Concrete Rosette - O degrees	10.92	0.00	22.50	10.85	1.25	22.50	
S-IB-R4B	Concrete Rosette - 45 degrees	10.92	0.00	22.50	10.85	1.25	22.50	
S-IB-R4C	Concrete Rosette - 90 degrees	10.92	0.00	22.50	10.85	1.25	22.50	
S-IB-R5A	Concrete Rosette - O degrees	17.58	0.00	22.50	17.80	1.25	22.63	
S-IB-R5B	Concrete Rosette - 45 degrees	17.58	0.00	22.50	17.80	1.25	22.63	
S-IB-R5C	Concrete Rosette - 90 degrees	17.58	0.00	22.50	17.80	1.25	22.63	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
S-IB-S1B	Stirrup gages - one leg only	2.25	-1.50	22.50	2.25	-1.50	23.00	west leg
S-IB-S1C	Stirrup gages - one leg only	2.25	-1.50	11.50	2.25	-1.50	10.25	w
S-IB-S3B	Stirrup gages - one leg only	4.92	-1.50	22.50	4.85	-1.50	23.00	w
S-IB-S3C	Stirrup gages - one leg only	4.92	-1.50	11.50	4.85	-1.50	11.50	w
S-IB-S4A	Stirrup gages - one leg only	6.25	-1.50	35.00	6.20	-1.50	35.25	w
S-IB-S4B	Stirrup gages - one leg only	6.25	-1.50	22.50	6.20	-1.50	23.00	w
S-IB-S4C	Stirrup gages - one leg only	6.25	-1.50	11.50	6.20	-1.50	11.00	w
S-IB-S5A	Stirrup gages - one leg only	7.58	-1.50	35.00	7.60	-1.50	35.25	w
S-IB-S5B	Stirrup gages - one leg only	7.58	-1.50	22.50	7.60	-1.50	23.00	w
S-IB-S5C	Stirrup gages - one leg only	7.58	-1.50	11.50	7.60	-1.50	11.00	w
S-IB-S6A	Stirrup gages - one leg only	8.92	-1.50	35.00	8.95	-1.50	35.50	w
S-IB-S6B	Stirrup gages - one leg only	8.92	-1.50	22.50	8.95	-1.50	23.00	w
S-IB-S6C	Stirrup gages - one leg only	8.92	-1.50	11.50	8.95	-1.50	11.50	w
S-IB-S7A	Stirrup gages - one leg only	10.25	-1.50	35.00	10.30	-1.50	35.50	w
S-IB-S7B	Stirrup gages - one leg only	10.25	-1.50	22.50	10.30	-1.50	22.25	w
S-IB-S8A	Stirrup gages - one leg only	11.58	-1.50	35.00	11.60	-1.50	36.00	w
S-IB-S8B	Stirrup gages - one leg only	11.58	-1.50	22.50	11.60	-1.50	23.00	w
S-IB-S12A	Stirrup gages - one leg only	16.92	-1.50	35.00	17.10	-1.50	35.75	w
S-IB-S12B	Stirrup gages - one leg only	16.92	-1.50	22.50	17.10	-1.50	23.00	w
S-IB-S12C	Stirrup gages - one leg only	16.92	-1.50	11.50	17.10	-1.50	11.50	w
S-IB-S13A	Stirrup gages - one leg only	18.25	-1.50	35.00	18.30	-1.50	35.00	w
S-IB-S13B	Stirrup gages - one leg only	18.25	-1.50	22.50	18.30	-1.50	22.50	w
S-IB-S13C	Stirrup gages - one leg only	18.25	-1.50	11.50	18.30	-1.50	11.00	w
S-IB-S15A	Stirrup gages - one leg only	20.92	-1.50	35.00	21.00	-1.50	36.00	w

S-IB-S15B	Stirrup gages - one leg only	20.92	-1.50	22.50	21.00	-1.50	23.00	w
S-IB-S18A	Stirrup gages - one leg only	24.92	-1.50	35.00	25.10	-1.50	35.25	w
S-IB-S18B	Stirrup gages - one leg only	24.92	-1.50	22.50	25.10	-1.50	23.00	w
S-IB-S18C	Stirrup gages - one leg only	24.92	-1.50	11.50	25.10	-1.50	11.25	w
S-IB-S22B	Stirrup gages - one leg only	30.25	-1.50	22.50	30.50	-1.50	23.00	w
S-IB-S22C	Stirrup gages - one leg only	30.25	-1.50	11.50	30.50	-1.50	11.00	w

Other notes for End B:
Last hook stirrup at 41.2'
Harp point at 53.9'
4 strand draped/ 8 strands debonded

GIRDER II - END C (in girder only, not slab)

Gage Designation	Gage Description	Design Location			Actual Field Location			
		X	Y	Z	X	Y	Z	
		(in)	(in)	(in)	(in)	(in)	(in)	
R-IIC-D1	Displacement Transducer	15.00	13.00	4.67	15.00	13.00	4.65	
R-IIC-B1	Bursting, G605E, east leg	2.00	1.70	11.50	2.50	1.70	11.00	
R-IIC-B2	Bursting, G605E, west leg	2.00	-1.70	11.50	2.50	-1.70	11.00	
R-IIC-B3	Bursting, G604E, west leg	8.00	-1.70	11.50	8.00	-1.70	11.50	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
T-IIC-035A	Transfer length 0', 15"	1.25	3.00	6.00	1.50	3.00	6.25	y distance not measured
T-IIC-07A	Transfer length 0', 15"	1.25	-1.00	2.00	1.40	-1.00	2.00	
T-IIC-035B	Transfer length 0', 22"	1.83	3.00	6.00	2.15	3.00	6.25	
T-IIC-07B	Transfer length 0', 22"	1.83	-1.00	2.00	2.20	-1.00	2.00	
T-IIC-035C	Transfer length 0', 30"	2.50	3.00	6.00	2.60	3.00	6.25	
T-IIC-07C	Transfer length 0', 30"	2.50	-1.00	2.00	2.50	-1.00	2.00	
T-IIC-25A	Transfer length 2', debonded = 15"	3.25	3.00	2.00	3.80	3.00	2.25	
T-IIC-28A	Transfer length 2', debonded = 15"	3.25	-3.00	2.00	3.50	-3.00	2.25	
T-IIC-25B	Transfer length 2', debonded = 22"	3.83	3.00	2.00	4.15	3.00	2.25	

T-IIC-28B	Transfer length 2', debonded = 22"	3.83	-3.00	2.00	4.22	-3.00	2.25	
T-IIC-25C	Transfer length 2', debonded = 30"	4.50	3.00	2.00	4.70	3.00	2.25	
T-IIC-28C	Transfer length 2', debonded = 30"	4.50	-3.00	2.00	4.60	-3.00	2.25	
T-IIC-44A	Transfer length 4', debonded = 15"	5.25	5.00	2.00	5.30	5.00	2.25	
T-IIC-49A	Transfer length 4', debonded = 15"	5.25	-5.00	2.00	5.45	-5.00	2.25	
T-IIC-44B	Transfer length 4', debonded = 22"	5.83	5.00	2.00	6.00	5.00	2.25	
T-IIC-49B	Transfer length 4', debonded = 22"	5.83	-5.00	2.00	5.82	-5.00	2.25	
T-IIC-44C	Transfer length 4', debonded = 30"	6.50	5.00	2.00	6.60	5.00	2.25	
T-IIC-49C	Transfer length 4', debonded = 30"	6.50	-5.00	2.00	6.35	-5.00	2.25	
T-IIC-123A	Transfer length 12', debonded = 15"	13.25	7.00	2.00	13.30	9.00	2.00	strand 12-2 gaged, debonded
T-IIC-1210A	Transfer length 12', debonded = 15"	13.25	-7.00	2.00	13.10	-7.00	2.00	
T-IIC-123B	Transfer length 12', debonded = 22"	13.83	7.00	2.00	13.90	9.00	2.00	strand 12-2 gaged, debonded
T-IIC-1210B	Transfer length 12', debonded = 22"	13.83	-7.00	2.00	13.70	-7.00	2.00	
T-IIC-123C	Transfer length 12', debonded = 30"	14.50	7.00	2.00	14.60	9.00	2.00	strand 12-2 gaged, debonded
T-IIC-1210C	Transfer length 12', debonded = 30"	14.50	-7.00	2.00	14.40	-7.00	2.00	
T-IIC-202A	Transfer length 20', debonded = 15"	21.25	9.00	2.00	21.38	9.00	2.25	
T-IIC-2011A	Transfer length 20', debonded = 15"	21.25	-9.00	2.00	21.35	-9.00	2.25	
T-IIC-202B	Transfer length 20', debonded = 22"	21.83	9.00	2.00	21.90	9.00	2.25	
T-IIC-2011B	Transfer length 20', debonded = 22"	21.83	-9.00	2.00	21.85	-9.00	2.25	
T-IIC-202C	Transfer length 20', debonded = 30"	22.50	9.00	2.00	22.45	9.00	2.25	
T-IIC-2011C	Transfer length 20', debonded = 30"	22.50	-9.00	2.00	22.45	-9.00	2.25	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
V-IIC-1	@ CG strands in transfer region	1.25	0.00	4.67	1.40		5.25	no centerline dimension recorded
V-IIC-2	top flange over support	0.63	0.00	44.00	0.58		38.25	
V-IIC-3	bottom flange @.45L, 2"	59.74	0.00	2.00	59.45		2.50	
V-IIC-4	bottom flange @.45L, cg strands	59.74	0.00	5.39	59.45		5.50	
		(in)	(in)	(in)	(in)	(in)	(in)	
R-IIC-P1	PML over support for rupture	7.50	0.00	9.50	7.50		9.50	no centerline dimension recorded
R-IIC-P2	PML over support for rupture	7.50	0.00	12.50	7.50		12.50	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
F-IIC-P21	PML-60 @.2L - Center Line of X-Section	26.55	0.00	38.00	27.00	0.00	38.00	
F-IIC-P22	PML-60 @.2L - Center Line of X-Section	26.55	0.00	44.00	27.00	0.00	44.00	
F-IIC-P23	PML-60 @.2L - 3" from edge of flange	26.55	-12.00	44.00	27.00	-9.25	44.00	y moved due to top steel location

F-IIC-P31	PML-60 @.3L - Center Line of X-Section	39.83	0.00	38.00	40.10	0.00	38.00	
F-IIC-P32	PML-60 @.3L - Center Line of X-Section	39.83	0.00	44.00	40.10	0.00	44.00	
F-IIC-P33	PML-60 @.3L - 3" from edge of flange	39.83	-12.00	44.00	40.10	-9.25	44.00	y moved due to top steel location
F-IIC-12	Strand gage @ 0.45L	59.74	9.00	2.00	60.20	9.00	2.25	
F-IIC-15	Strand gage @ 0.45L	59.74	3.00	2.00	59.75	3.00	2.25	
F-IIC-17	Strand gage @ 0.45L	59.74	-1.00	2.00	59.80	-1.00	2.25	
F-IIC-19	Strand gage @ 0.45L	59.74	-5.00	2.00	59.70	-5.00	2.25	
F-IIC-25	Strand gage @ 0.45L	59.74	3.00	4.00	59.75	3.00	4.00	
F-IIC-35	Strand gage @ 0.45L	59.74	3.00	6.00	59.75	3.00	6.25	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
S-IIC-R1A	Concrete Rosette - O degrees	2.92	0.00	22.50	2.70	1.25	22.00	on inside of east leg of stirrup
S-IIC-R1B	Concrete Rosette - 45 degrees	2.92	0.00	22.50	2.70	1.25	22.00	all placed with hand level
S-IIC-R1C	Concrete Rosette - 90 degrees	2.92	0.00	22.50	2.70	1.25	22.00	
S-IIC-R2A	Concrete Rosette - O degrees	5.58	0.00	22.50	5.30	1.25	22.75	
S-IIC-R2B	Concrete Rosette - 45 degrees	5.58	0.00	22.50	5.30	1.25	22.75	
S-IIC-R2C	Concrete Rosette - 90 degrees	5.58	0.00	22.50	5.30	1.25	22.75	
S-IIC-R3A	Concrete Rosette - O degrees	8.25	0.00	22.50	8.00	1.25	23.50	
S-IIC-R3B	Concrete Rosette - 45 degrees	8.25	0.00	22.50	8.00	1.25	23.50	
S-IIC-R3C	Concrete Rosette - 90 degrees	8.25	0.00	22.50	8.00	1.25	23.50	
S-IIC-R4A	Concrete Rosette - O degrees	10.92	0.00	22.50	10.50	1.25	23.25	
S-IIC-R4B	Concrete Rosette - 45 degrees	10.92	0.00	22.50	10.50	1.25	23.25	
S-IIC-R4C	Concrete Rosette - 90 degrees	10.92	0.00	22.50	10.50	1.25	23.25	
S-IIC-R5A	Concrete Rosette - O degrees	17.58	0.00	22.50	17.60	1.25	22.25	
S-IIC-R5B	Concrete Rosette - 45 degrees	17.58	0.00	22.50	17.60	1.25	22.25	
S-IIC-R5C	Concrete Rosette - 90 degrees	17.58	0.00	22.50	17.60	1.25	22.25	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
S-IIC-S1B	Stirrup gages - one leg only	2.25	1.50	22.50	2.10	1.50	22.75	east leg
S-IIC-S1C	Stirrup gages - one leg only	2.25	1.50	11.50	2.10	1.50	10.75	e
S-IIC-S3B	Stirrup gages - one leg only	4.92	1.50	22.50	4.65	1.50	23.00	e
S-IIC-S3C	Stirrup gages - one leg only	4.92	1.50	11.50	4.65	1.50	11.00	e
S-IIC-S4A	Stirrup gages - one leg only	6.25	1.50	35.00	6.00	1.50	35.00	e
S-IIC-S4B	Stirrup gages - one leg only	6.25	1.50	22.50	6.00	1.50	23.00	e
S-IIC-S4C	Stirrup gages - one leg only	6.25	1.50	11.50	6.00	1.50	11.00	e
S-IIC-S5A	Stirrup gages - one leg only	7.58	1.50	35.00	7.45	1.50	36.00	e
S-IIC-S5B	Stirrup gages - one leg only	7.58	1.50	22.50	7.45	1.50	23.25	e
S-IIC-S5C	Stirrup gages - one leg only	7.58	1.50	11.50	7.45	1.50	11.25	e

S-IIC-S6A	Stirrup gages - one leg only	8.92	1.50	35.00	8.70	1.50	35.25	e
S-IIC-S6B	Stirrup gages - one leg only	8.92	1.50	22.50	8.70	1.50	23.00	e
S-IIC-S6C	Stirrup gages - one leg only	8.92	1.50	11.50	8.70	1.50	11.00	e
S-IIC-S7A	Stirrup gages - one leg only	10.25	1.50	35.00	10.00	1.50	35.50	e
S-IIC-S7B	Stirrup gages - one leg only	10.25	1.50	22.50	10.00	1.50	23.00	e
S-IIC-S8A	Stirrup gages - one leg only	11.58	1.50	35.00	11.30	1.50	35.50	e
S-IIC-S8B	Stirrup gages - one leg only	11.58	1.50	22.50	11.30	1.50	23.00	e
S-IIC-S12A	Stirrup gages - one leg only	16.92	1.50	35.00	16.90	1.50	35.50	e
S-IIC-S12B	Stirrup gages - one leg only	16.92	1.50	22.50	16.90	1.50	23.00	e
S-IIC-S12C	Stirrup gages - one leg only	16.92	1.50	11.50	16.90	1.50	11.00	e
S-IIC-S13A	Stirrup gages - one leg only	18.25	1.50	35.00	18.20	1.50	35.50	e
S-IIC-S13B	Stirrup gages - one leg only	18.25	1.50	22.50	18.20	1.50	23.00	e
S-IIC-S13C	Stirrup gages - one leg only	18.25	1.50	11.50	18.20	1.50	11.00	e
S-IIC-S15A	Stirrup gages - one leg only	20.92	1.50	35.00	20.90	1.50	35.25	e
S-IIC-S15B	Stirrup gages - one leg only	20.92	1.50	22.50	20.90	1.50	23.00	e
S-IIC-S18A	Stirrup gages - one leg only	24.92	1.50	35.00	24.90	1.50	36.00	e
S-IIC-S18B	Stirrup gages - one leg only	24.92	1.50	22.50	24.90	1.50	23.00	e
S-IIC-S18C	Stirrup gages - one leg only	24.92	1.50	11.50	24.90	1.50	11.50	e
S-IIC-S22B	Stirrup gages - one leg only	30.25	1.50	22.50	30.10	1.50	23.00	e
S-IIC-S22C	Stirrup gages - one leg only	30.25	1.50	11.50	30.10	1.50	11.50	e

Other notes for End C:
Last hook stirrup at 40.9'
Harp point at 51.5'
4 strands draped/ 8 debonded

GIRDER II - END D (in girder only, not slab)

Gage Designation	Gage Description	Design Location			Actual Field Location		
		X	Y	Z	X	Y	Z
		(in)	(in)	(in)	(in)	(in)	(in)
R-IID-D1	Displacement Transducer	15.00	13.00	4.67	15.00	13.00	4.65
R-IID-B1	Bursting, G605E, east leg	2.00	1.70	11.50	3.00	1.70	11.25
R-IID-B2	Bursting, G605E, west leg	2.00	-1.70	11.50	2.50	-1.70	11.25
R-IID-B3	Bursting, G604E, east leg	8.00	1.70	11.50	9.00	-1.70	11.75

		(feet)	(in)	(in)	(feet)	(in)	(in)	
T-IID-02A	Transfer length 0', 15"	1.25	9.00	2.00	1.10	9.00	2.00	
T-IID-05A	Transfer length 0', 15"	1.25	3.00	2.00	1.30	3.00	2.00	
T-IID-035A	Transfer length 0', 15"	1.25	-5.00	6.00	1.15	-5.00	6.00	
T-IID-09A	Transfer length 0', 15"	1.25	3.00	2.00	1.40	3.00	2.00	
T-IID-02B	Transfer length 0', 22"	1.83	9.00	2.00	1.85	9.00	2.00	
T-IID-05B	Transfer length 0', 22"	1.83	3.00	2.00	1.90	3.00	2.00	
T-IID-035B	Transfer length 0', 22"	1.83	-5.00	6.00	1.75	-5.00	6.00	
T-IID-09B	Transfer length 0', 22"	1.83	3.00	2.00	2.00	3.00	2.00	
T-IID-02C	Transfer length 0', 30"	2.50	9.00	2.00	2.50	9.00	2.00	
T-IID-05C	Transfer length 0', 30"	2.50	3.00	2.00	2.50	3.00	2.00	
T-IID-035C	Transfer length 0', 30"	2.50	-5.00	6.00	2.45	-5.00	6.00	
T-IID-09C	Transfer length 0', 30"	2.50	3.00	2.00	2.45	3.00	2.00	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
V-IID-1	@ CG strands in transfer region	1.25	0.00	4.67	1.30	0.00	5.50	
V-IID-2	top flange over support	0.63	0.00	44.00	0.63	0.00	39.50	
V-IID-3	bottom flange @.45L, 2"	59.74	0.00	2.00	59.80	0.00	2.25	
V-IID-4	bottom flange @.45L, cg strands	59.74	0.00	5.39	59.80	0.00	5.50	
V-IID-5	bottom flange @.5L, 2"	66.38	0.00	2.00	66.40	0.00	2.25	
V-IID-6	bottom flange @.5L, cg strands	66.38	0.00	5.39	66.40	0.00	5.25	
V-IID-7	top flange @.5L	66.38	0.00	44.00	66.40	0.00	42.25	
		(in)	(in)	(in)	(in)	(in)	(in)	
R-IID-P1	PML over support for rupture	7.50	0.00	9.50	7.00	0.00	8.50	
R-IID-P2	PML over support for rupture	7.50	0.00	12.50	7.00	0.00	12.25	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
F-IID-P21	PML-60 @.2L - Center Line of X-Section	26.55	0.00	38.00	26.90	0.00	38.00	
F-IID-P22	PML-60 @.2L - Center Line of X-Section	26.55	0.00	44.00	26.90	0.00	44.00	
F-IID-P23	PML-60 @.2L - 3" from edge of flange	26.55	-12.00	44.00	26.90	-8.75	44.00	y relocated due to top steel
F-IID-P31	PML-60 @.3L - Center Line of X-Section	39.83	0.00	38.00	39.70	0.00	38.00	
F-IID-P32	PML-60 @.3L - Center Line of X-Section	39.83	0.00	44.00	39.70	0.00	44.00	
F-IID-P33	PML-60 @.3L - 3" from edge of flange	39.83	-12.00	44.00	39.70	-9.00	44.00	y relocated due to top steel
F-IID-P51	PML-60 @ .5L - cL of X-Section	66.38	0.00	25.00	66.42	0.00	25.00	
F-IID-P52	PML-60 @ .5L - cL of X-Section	66.38	0.00	35.00	66.42	0.00	5.50	
F-IID-P53	PML-60 @ .5L - cL of X-Section	66.38	0.00	38.00	66.42	0.00	38.00	

F-IID-P54	PML-60 @ .5L - cL of X-Section	66.38	0.00	41.00	66.42	0.00	41.00	
F-IID-P55	PML-60 @ .5L - cL of X-Section	66.38	0.00	44.00	66.42	0.00	43.50	
F-IID-P56	PML-60 @ .5L - 3" from edge of flange	66.38	-12.00	44.00	66.42	-7.75	43.50	y relocated due to lap splice
		(feet)	(in)	(in)	(feet)	(in)	(in)	
F-IID-12	Strand gage @ 0.45L	59.74	9.00	2.00	59.70	9.00	2.00	gages placed at top of strand
F-IID-15	Strand gage @ 0.45L	59.74	3.00	2.00	59.90	3.00	2.00	no apparent twist due to tensioning
F-IID-17	Strand gage @ 0.45L	59.74	-1.00	2.00	59.80	-1.00	2.00	
F-IID-19	Strand gage @ 0.45L	59.74	-5.00	2.00	59.80	-5.00	2.00	
F-IID-25	Strand gage @ 0.45L	59.74	3.00	4.00	59.70	3.00	4.00	
F-IID-35	Strand gage @ 0.45L	59.74	3.00	6.00	59.75	3.00	6.25	
F-IID-C12	Strand gage @ .5L	66.38	9.00	2.00	66.20	9.00	2.25	
F-IID-C15	Strand gage @ .5L	66.38	3.00	2.00	66.00	3.00	2.25	
F-IID-C17	Strand gage @ .5L	66.38	-1.00	2.00	66.25	-1.00	2.25	
F-IID-C19	Strand gage @ .5L	66.38	-5.00	2.00	66.35	-5.00	2.25	
F-IID-C25	Strand gage @ .5L	66.38	3.00	4.00	66.45	3.00	4.25	
F-IID-C35	Strand gage @ .5L	66.38	3.00	6.00	66.45	3.00	6.25	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
S-IID-R1A	Concrete Rosette - O degrees	2.92	0.00	22.50	2.70	2.00	23.25	on outside of east leg of stirrup
S-IID-R1B	Concrete Rosette - 45 degrees	2.92	0.00	22.50	2.70	2.00	23.25	all placed with hand level
S-IID-R1C	Concrete Rosette - 90 degrees	2.92	0.00	22.50	2.70	2.00	23.25	
S-IID-R2A	Concrete Rosette - O degrees	5.58	0.00	22.50	5.45	2.00	23.00	
S-IID-R2B	Concrete Rosette - 45 degrees	5.58	0.00	22.50	5.45	2.00	23.00	
S-IID-R2C	Concrete Rosette - 90 degrees	5.58	0.00	22.50	5.45	2.00	23.00	
S-IID-R3A	Concrete Rosette - O degrees	8.25	0.00	22.50	8.10	2.00	22.50	
S-IID-R3B	Concrete Rosette - 45 degrees	8.25	0.00	22.50	8.10	2.00	22.50	
S-IID-R3C	Concrete Rosette - 90 degrees	8.25	0.00	22.50	8.10	2.00	22.50	
S-IID-R4A	Concrete Rosette - O degrees	10.92	0.00	22.50	10.80	2.00	22.50	
S-IID-R4B	Concrete Rosette - 45 degrees	10.92	0.00	22.50	10.80	2.00	22.50	
S-IID-R4C	Concrete Rosette - 90 degrees	10.92	0.00	22.50	10.80	2.00	22.50	
S-IID-R5A	Concrete Rosette - O degrees	17.58	0.00	22.50	17.65	2.00	23.50	
S-IID-R5B	Concrete Rosette - 45 degrees	17.58	0.00	22.50	17.65	2.00	23.50	
S-IID-R5C	Concrete Rosette - 90 degrees	17.58	0.00	22.50	17.65	2.00	23.50	
		(feet)	(in)	(in)	(feet)	(in)	(in)	
S-IID-S1B	Stirrup gages - one leg only	2.25	1.50	22.50	2.10	-1.50	22.50	west leg
S-IID-S1C	Stirrup gages - one leg only	2.25	1.50	11.50	2.10	-1.50	10.50	w

S-IID-S3B	Stirrup gages - one leg only	4.92	1.50	22.50	4.80	-1.50	23.00	w
S-IID-S3C	Stirrup gages - one leg only	4.92	1.50	11.50	4.80	-1.50	11.00	w
S-IID-S4A	Stirrup gages - one leg only	6.25	1.50	35.00	6.10	-1.50	35.50	w
S-IID-S4B	Stirrup gages - one leg only	6.25	1.50	22.50	6.10	-1.50	23.00	w
S-IID-S4C	Stirrup gages - one leg only	6.25	1.50	11.50	6.10	-1.50	11.00	w
S-IID-S5A	Stirrup gages - one leg only	7.58	1.50	35.00	7.60	-1.50	35.50	w
S-IID-S5B	Stirrup gages - one leg only	7.58	1.50	22.50	7.60	-1.50	23.50	w
S-IID-S5C	Stirrup gages - one leg only	7.58	1.50	11.50	7.60	-1.50	11.00	w
S-IID-S6A	Stirrup gages - one leg only	8.92	1.50	35.00	8.80	-1.50	35.00	w
S-IID-S6B	Stirrup gages - one leg only	8.92	1.50	22.50	8.80	-1.50	23.00	w
S-IID-S6C	Stirrup gages - one leg only	8.92	1.50	11.50	8.80	-1.50	11.50	w
S-IID-S7A	Stirrup gages - one leg only	10.25	1.50	35.00	10.20	-1.50	35.50	w
S-IID-S7B	Stirrup gages - one leg only	10.25	1.50	22.50	10.20	-1.50	23.00	w
S-IID-S8A	Stirrup gages - one leg only	11.58	1.50	35.00	11.50	-1.50	35.25	w
S-IID-S8B	Stirrup gages - one leg only	11.58	1.50	22.50	11.50	-1.50	22.50	w
S-IID-S12A	Stirrup gages - one leg only	16.92	1.50	35.00	16.95	-1.50	35.50	w
S-IID-S12B	Stirrup gages - one leg only	16.92	1.50	22.50	16.95	-1.50	23.00	w
S-IID-S12C	Stirrup gages - one leg only	16.92	1.50	11.50	16.95	-1.50	11.50	w
S-IID-S13A	Stirrup gages - one leg only	18.25	1.50	35.00	18.30	-1.50	35.75	w
S-IID-S13B	Stirrup gages - one leg only	18.25	1.50	22.50	18.30	-1.50	23.00	w
S-IID-S13C	Stirrup gages - one leg only	18.25	1.50	11.50	18.30	-1.50	11.50	w
S-IID-S15A	Stirrup gages - one leg only	20.92	1.50	35.00	20.10	-1.50	35.50	w
S-IID-S15B	Stirrup gages - one leg only	20.92	1.50	22.50	20.10	-1.50	23.50	w
S-IID-S18A	Stirrup gages - one leg only	24.92	1.50	35.00	24.80	-1.50	35.25	w
S-IID-S18B	Stirrup gages - one leg only	24.92	1.50	22.50	24.80	-1.50	23.00	w
S-IID-S18C	Stirrup gages - one leg only	24.92	1.50	11.50	24.80	-1.50	11.00	w
S-IID-S22B	Stirrup gages - one leg only	30.25	1.50	22.50	30.30	-1.50	23.00	w
S-IID-S22C	Stirrup gages - one leg only	30.25	1.50	11.50	30.30	-1.50	11.50	w

Other notes for End D:
Last hook stirrup at 41.9'
Harp point at 55.0'
12 strands draped/ 0 debonded

Appendix E

Description and Locations of Instrumentation used in Deck

CONCRETE DECK INSTRUMENTATION LOCATION AND DESCRIPTION

Gage Designation:

Test phase - girder and end/slab - Gage #

Test Phase:

V = Vibrating wire gages for shrinkage and temperature, etc.
F = Flexure gages on rebar and in concrete

Instrumentation Types, By Function:

Rebar Gage: TML Type WFLA-3
Vibrating Wire Gage: GEOKON Type VCE-4200
PML Concrete Gage: TML Type PML-60

Location:

X dimension is from the designated end
Y dimension is from the longitudinal centerline of girder as located in the lab, South = +, North = -
Z dimension is from the top of the girder

GIRDER I - END A, Slab gages

Gage Designation	Gage Description	Design Location			Actual Field Location		
		X	Y	Z	X	Y	Z
		(in)	(in)	(in)	(in)	(in)	(in)
V-IAS-1	over support @ x-s cL	7.50	0.00	5.50	7.50	3.00	6.00
V-IAS-2	over support @ x-s cL+21"	7.50	21.00	5.50	7.50	20.00	6.00
V-IAS-3	0.5L @ x-s cL	66.38 ft	0.00	5.50	66.38	-2.50	6.50
		(feet)	(in)	(in)	(feet)	(in)	(in)
F-IAS-P21	SLAB-PML @.2L x-s cL	26.55	0.00	6.50	26.55	1.00	6.50
F-IAS-P22	SLAB-PML @.2L x-s cL+12"	26.55	12.00	6.50	26.55	12.00	6.50
F-IAS-P23	SLAB-PML @.2L x-s cL+21"	26.55	21.00	6.50	26.55	21.00	6.50
F-IAS-P31	SLAB-PML @.3L x-s cL	39.83	0.00	6.50	39.99	1.00	6.50
F-IAS-P32	SLAB-PML @.3L x-s cL+12"	39.83	12.00	6.50	39.99	12.00	6.50
F-IAS-P33	SLAB-PML @.3L x-s cL+21"	39.83	21.00	6.50	39.99	21.00	6.50
F-IAS-P51	SLAB-PML @.5L x-s cL	66.38	0.00	6.50	66.38	1.00	6.50
F-IAS-P52	SLAB-PML @.5L x-s cL+12"	66.38	12.00	6.50	66.38	12.00	6.50

F-IAS-P53	SLAB-PML @.5L x-s cL+21"	66.38	21.00	6.50	66.38	21.00	6.50
F-IAS-P54	SLAB-PML @.5L x-s cL	66.38	0.00	3.00	66.38	1.00	3.50
F-IAS-P55	SLAB-PML @.5L x-s cL+12"	66.38	12.00	3.00	66.38	12.00	3.50
		(feet)	(in)	(in)	(feet)	(in)	(in)
F-IAS-2B	Top steel @.2L x-s cL	26.55	0.00	6.50	26.55	0.00	6.50
F-IAS-2E	Bottom steel @.2L x-s cL	26.55	9.00	3.00	26.55	9.00	3.00
F-IAS-3B	Top steel @.3L x-s cL	39.83	0.00	6.50	39.83	0.00	6.50
F-IAS-3E	Bottom steel @.3L x-s cL	39.83	9.00	3.00	39.83	9.00	3.00
F-IAS-45A	Top steel @.45L x-s cL	59.74	-18.00	6.50	59.31	-15.00	6.50
F-IAS-45B	Top steel @.45L x-s cL	59.74	0.00	6.50	59.31	0.00	6.50
F-IAS-45C	Top steel @.45L x-s cL	59.74	18.00	6.50	59.31	15.00	6.50
F-IAS-45D	Bottom steel @.45L x-s cL	59.74	-9.00	3.00	59.31	-9.00	3.00
F-IAS-45E	Bottom steel @.45L x-s cL	59.74	9.00	3.00	59.31	9.00	3.00
F-IAS-5A	Top steel @.5L x-s cL	66.38	-18.00	6.50	66.50	-15.00	6.50
F-IAS-5B	Top steel @.5L x-s cL	66.38	0.00	6.50	66.50	0.00	6.50
F-IAS-5C	Top steel @.5L x-s cL	66.38	18.00	6.50	66.50	15.00	6.50
F-IAS-5D	Bottom steel @.5L x-s cL	66.38	-9.00	3.00	66.50	-9.00	3.00
F-IAS-5E	Bottom steel @.5L x-s cL	66.38	9.00	3.00	66.50	9.00	3.00

GIRDER I - END B, Slab gages

Gage Designation	Gage Description	Design Location X	Design Location Y	Design Location Z	Actual Field Location X	Actual Field Location Y	Actual Field Location Z
		(in)	(in)	(in)	(in)	(in)	(in)
V-IBS-4	over support @ x-s cL	7.50	0.00	5.50	7.50	2.00	6.00
V-IBS-5	over support @ x-s cL+21"	7.50	21.00	5.50	7.50	20.00	6.00
		(feet)	(in)	(in)	(feet)	(in)	(in)
F-IBS-P21	SLAB-PML @.2L x-s cL	26.55	0.00	6.50	27.08	1.00	6.50
F-IBS-P22	SLAB-PML @.2L x-s cL+12"	26.55	12.00	6.50	27.08	12.00	6.50
F-IBS-P23	SLAB-PML @.2L x-s cL+21"	26.55	21.00	6.50	27.08	21.00	6.50
F-IBS-P31	SLAB-PML @.3L x-s cL	39.83	0.00	6.50	39.83	1.00	6.50
F-IBS-P32	SLAB-PML @.3L x-s cL+12"	39.83	12.00	6.50	39.83	12.00	6.50
F-IBS-P33	SLAB-PML @.3L x-s cL+21"	39.83	21.00	6.50	39.83	21.00	6.50
F-IBS-3B	Top steel @.3L x-s cL	39.83	0.00	6.50	40.59	0.00	6.50
F-IBS-3E	Bottom steel @.3L x-s cL	39.83	9.00	3.00	40.59	9.00	3.00
F-IBS-45B	Top steel @.45L x-s cL	59.74	0.00	6.50	59.44	0.00	6.50
F-IBS-45E	Bottom steel @.45L x-s cL	59.74	9.00	3.00	59.44	9.00	3.00

GIRDER II - END C, Slab gages

Gage Designation	Gage Description	Design Location			Actual Field Location		
		X	Y	Z	X	Y	Z
		(in)	(in)	(in)	(in)	(in)	(in)
V-IICS-1	over support @ x-s cL	7.50	0.00	5.50	7.50	1.00	6.00
V-IICS-2	over support @ x-s cL+21"	7.50	21.00	5.50	7.50	-21.00	6.00
		(feet)	(in)	(in)	(feet)	(in)	(in)
F-IICS-P21	SLAB-PML @.2L x-s cL	26.55	0.00	6.50	26.91	2.00	6.50
F-IICS-P22	SLAB-PML @.2L x-s cL+12"	26.55	12.00	6.50	26.91	12.00	6.50
F-IICS-P23	SLAB-PML @.2L x-s cL+21"	26.55	21.00	6.50	26.91	21.00	6.50
F-IICS-P31	SLAB-PML @.3L x-s cL	39.83	0.00	6.50	39.89	2.00	6.50
F-IICS-P32	SLAB-PML @.3L x-s cL+12"	39.83	12.00	6.50	39.89	13.00	6.50
F-IICS-P33	SLAB-PML @.3L x-s cL+21"	39.83	21.00	6.50	39.89	22.00	6.50
F-IICS-3B	Top steel @.3L x-s cL	39.83	0.00	6.50	39.83	1.00	6.50
F-IICS-3E	Bottom steel @.3L x-s cL	39.83	9.00	3.00	39.83	9.00	3.00
F-IICS-45A	Top steel @.45L x-s cL	59.74	-18.00	6.50	59.50	-15.00	6.50
F-IICS-45B	Top steel @.45L x-s cL	59.74	0.00	6.50	59.50	1.00	6.50
F-IICS-45C	Top steel @.45L x-s cL	59.74	18.00	6.50	59.50	15.00	6.50
F-IICS-45D	Bottom steel @.45L x-s cL	59.74	-9.00	3.00	59.50	-9.00	3.00
F-IICS-45E	Bottom steel @.45L x-s cL	59.74	9.00	3.00	59.50	9.00	3.00

GIRDER II - END D, Slab gages

Gage Designation	Gage Description	Design Location			Actual Field Location		
		X	Y	Z	X	Y	Z
		(in)	(in)	(in)	(in)	(in)	(in)
V-IIDS-4	over support @ x-s cL	7.50	0.00	5.50	7.50	2.00	6.00
V-IIDS-5	over support @ x-s cL+21"	7.50	21.00	5.50	7.50	20.00	6.00
V-IIDS-3	0.5L @ x-s cL	66.38 ft	0.00	5.50	66.38 ft	-2.00	6.50
		(feet)	(in)	(in)	(feet)	(in)	(in)
F-IIDS-P21	SLAB-PML @.2L x-s cL	26.55	0.00	6.50	27.16	1.00	6.50
F-IIDS-P22	SLAB-PML @.2L x-s cL+12"	26.55	12.00	6.50	27.16	12.00	6.50
F-IIDS-P23	SLAB-PML @.2L x-s cL+21"	26.55	21.00	6.50	27.16	21.00	6.50
F-IIDS-P31	SLAB-PML @.3L x-s cL	39.83	0.00	6.50	39.80	1.00	6.50
F-IIDS-P32	SLAB-PML @.3L x-s cL+12"	39.83	12.00	6.50	39.80	12.00	6.50
F-IIDS-P33	SLAB-PML @.3L x-s cL+21"	39.83	21.00	6.50	39.80	21.00	6.50
F-IIDS-P51	SLAB-PML @.5L x-s cL	66.38	0.00	6.50	66.38	1.00	6.50
F-IIDS-P52	SLAB-PML @.5L x-s cL+12"	66.38	12.00	6.50	66.38	12.00	6.50
F-IIDS-P53	SLAB-PML @.5L x-s cL+21"	66.38	21.00	6.50	66.38	21.00	6.50
F-IIDS-P54	SLAB-PML @.5L x-s cL	66.38	0.00	3.00	66.38	1.00	3.50
F-IIDS-P55	SLAB-PML @.5L x-s cL+12"	66.38	12.00	3.00	66.38	12.00	3.50
		(feet)	(in)	(in)	(feet)	(in)	(in)
F-IIDS-3B	Top steel @.3L x-s cL	39.83	0.00	6.50	39.83	0.00	6.50
F-IIDS-3E	Bottom steel @.3L x-s cL	39.83	9.00	3.00	39.83	9.00	3.00
F-IIDS-45A	Top steel @.45L x-s cL	59.74	-18.00	6.50	59.65	-15.00	6.50
F-IIDS-45B	Top steel @.45L x-s cL	59.74	0.00	6.50	59.65	0.00	6.50
F-IIDS-45C	Top steel @.45L x-s cL	59.74	18.00	6.50	59.65	15.00	6.50
F-IIDS-45D	Bottom steel @.45L x-s cL	59.74	-9.00	3.00	59.65	-9.00	3.00
F-IIDS-45E	Bottom steel @.45L x-s cL	59.74	9.00	3.00	59.65	9.00	3.00
F-IIDS-5A	Top steel @.5L x-s cL	66.38	-18.00	6.50	66.30	-15.00	6.50
F-IIDS-5B	Top steel @.5L x-s cL	66.38	0.00	6.50	66.30	0.00	6.50
F-IIDS-5C	Top steel @.5L x-s cL	66.38	18.00	6.50	66.30	15.00	6.50
F-IIDS-5D	Bottom steel @.5L x-s cL	66.38	-9.00	3.00	66.30	-9.00	3.00
F-IIDS-5E	Bottom steel @.5L x-s cL	66.38	9.00	3.00	66.30	9.00	3.00

Appendix F

Construction Field Log for Precast Test Girders

FIELD LOG - ELK RIVER PRESTRESSED PLANT
HIGH-STRENGTH CONCRETE PROJECT
UNIVERSITY OF MINNESOTA
COMPILED BY TESS AHLBORN

AUGUST 5, 1993 - THURSDAY

8:00 am The two test girder will be constructed on bed B1 at the far west location of the plant. The bed is 316 ft long and is rated at a 2 million pound capacity.

Twenty-two of the 0.6"- Gr.270 ksi strands have been pulled through the dead end and loosely anchored at the live end. The remaining 24 strands are currently being pulled through by Elk River employees.

11:00 am All strands have been pulled through. The strands are measured and locations to be instrumented with strain gages are taped. It is difficult to determine which strand is to be work on, since they are all laying in the bottom of the bed in no particular order.

11:30 am Light rain. All areas that are to be instrumented are covered with tarps.

12:00 pm Due to difficulty in locating the correct strands to be instrumented, the decision is made to pre-load the strands to 4 kips each. This will allow the strands to be separated for easier working conditions, however some sag is expected and the strands will need to be separated with 2x2 wood blocks. Before the pre-load is applied, 4 strands are instrumented and will be read with P3500 strain indicator boxes:

F-IA-19
F-IA-C15
F-IID-C12
F-IID-C15

1:15 pm The four designated gages are installed, initial readings are recorded. Pre-load begins and readings are as indicated in the P3500 notes.

Looking south at the live end, strands are preloaded to 4 kips in the following pattern:

1	2
3	4
5	6
7	8
9	10
11	12

				46	xx	xx	45				
35	36	37	38	39	xx	xx	40	41	42	43	44
34	33	32	31	30	xx	xx	29	28	27	26	25
13	14	15	16	17	18	19	20	21	22	23	24

2:15 pm Preloading completed. Elk River employees note that the chucks used at the live end are of two different heights. Shims will be made to level the chucks during the tensioning process (scheduled for noon the next day).

2:30 pm Strands are blocked for easier gage installation. Two of the 4 gages installed for preloading were destroyed during the process. (Preloading appears to have been a wise decision, else 50% of the gages could have been lost during tensioning.) The two lost gages will be reinstalled at the same location.
GAGING BEGINS FOR ALL LOCATIONS.

3:30 pm Light rain, working under the tarps is difficult but doable. Rain stops by 3:45 pm.

5:00 pm Sunny, gaging.

6:00 pm Still gaging.

6:30 pm Heavy rain spurt for 15 minutes. Still gaging.

9:25 pm Last strain gage is installed. Nearly half of the gages have been checked with the gage tester and immediately replaced when found to be defective.

9:45 pm Stop for the night. It is difficult to work in the dark, even with the overhead spotlights and flashlights.

AUGUST 6, 1993 - FRIDAY

5:45 am Sunny and 51F. (Weather data will be collected from the MNROAD project for the duration of this project. MNROAD is located NW 10

miles. However, general weather conditions are noted.)
All gages must be labeled and preattached cables are to be routed to the proper location for reading.

9:00 am Beam dimensions are rechecked and sole plates are positioned accordingly. Labeling and cabling continue.

12:15 pm Ends A and D, and centerline gages from both girders will be read manually by P3500 strain indicator boxes. Ends B and C will be read automatically by the OPTIM - Megadac 3008AC system. All channels are connected and appear to be reading correctly.

12:30 pm Ready to tension strands. Initial readings on the P3500 strain indicator boxes are taken and the OPTIM system has been set on automatic record mode. Elk River employees are notified. The tensioning pump has been borrowed from another local prestressing company to accommodate the large force that each strand must be pulled to (43.4 k/strand).

12:49 pm Optim: *ELKTEN1.001 started. (*=R TCS format, *=A ASCII format)
Duration = 2 hr. 30 min.

12:50 pm The first strand is tensioned. A pretension of 5 kips is conducted, then full load is applied. Specs (as determined by Elk River) are as follows:

Draped:	4350# 22 3/8" Elongation
Straight:	4450# 23" Elongation

Gage readings are taken after the following strand number has been fully tensioned:
Initial, 1, 10, 14 (first gaged strand), 15, 16, 17, 19, 20, 21, 22, 23, 29, 39 (last gaged strand), 46 (last strand)

Looking south at the live end, the strands are tensioned in the following pattern:

```
                  1  2
                  3  4
                  5  6
                  7  8
                  9  10
                 11  12

              45 xx  xx 46
 35 36 37 38 39 xx  xx 40 41 42 43 44
 25 26 27 28 29 xx  xx 30 31 32 33 34
 13 14 15 16 17 18  19 20 21 22 23 24
```

3:00 pm	Lift 4 draped strands at centerline. Read all gages on strands.
3:10 pm	ELKTEN1.001 stopped.
3:30 pm	End blanks placed by Elk River employees.
4:00 pm	Rebar cages are placed and tied. Elk River employees and UMN employees work together so that no damage occurs to the installed instrumentation.
5:00 pm	Clean up for evening, light breeze, partly cloudy, 71F.
6:00 pm	Done for evening.

AUGUST 7, 1993 - SATURDAY

7:00 am	Sunny, no breeze, 51F/49F wet. Place rosettes with angled leg in plane of compression strut.
8:00 pm	Read P3500 boxes at IA, IA-cL, IID, IID-cL.
8:18 am	ELKTEN1.002 started. Duration = 2 hr 20 min. To determine relaxation of strands from previous night.
9:00 am	PMLs and vibrating wire gages are being installed.
10:38 am	ELKTEN1.002 stopped.
1:00 pm	Begin installing demec strips on forms.
3:00 pm	Label all gages, route cables. Sunny, scattered clouds, light breeze, 71F/58F wet.
5:00 pm	Still labeling, cabling, installing demecs.
6:00 pm	Install end reinforcing/bursting steel, and instrumentation in end region.
7:00 pm	Done for evening.

AUGUST 8, 1993 - SUNDAY

8:00 am	Heavy rain from 6:00-7:30 am. Sunny, but wet, 68F/58F wet. Finish placing end region reinforcing and instrumentation.

9:00 am Locate every gage and record dimensions.

12:00 pm Sunny, 78F/62F wet, 15-20 mph wind.
Route vibrating wire harnesses and install vibrating wire board replacement.

2:30 pm Done for the day at Elk River.

3:30 pm At UMN to load truck with cylinders, beam forms, and misc. supplies for casting cylinders the next day.

5:00 pm Done for the day.

AUGUST 9, 1993 - MONDAY

6:00 am Sunny, 72F/69F wet, heavy rain overnight.
Elk River crew tie C-stirrups in bottom flanges of both girders, and set lift hooks. Double hooks are set at 8 ft and 10 ft from each end for yard handling and transportation. Single hooks are set at 4 ft and 36 ft from each end for shear specimen handling.

8:30 am Set forms in place, starting with south end of bed. East side first, then west side. One form must be bolted in place before continuing on to the next section. Typical sections are 30 ft long.

9:30 am Forms are in place. Route cables to OPTIM to be connected.

12:00 pm Set up area for casting cylinders. Shake table, wheel borrows, and molds in place.

12:30 pm Program OPTIM for casting.

12:45 pm Met with Elk River Crew to discuss vibration and care to be taken around instrumentation. Red tape on one side of stirrup for no vibration between form and stirrup or between stirrup, red tape on both side of stirrup for no vibration at that location. Side/bottom flange vibrator okay along entire length.
Sunny, 89F, hot and humid.

1:08 pm ELKCAST.001 started. Duration = 3 hr 30 min.

1:33 pm Start casting Girder I (Limestone mix) as noted by Elk River QC. Slumps: 7 3/4", 5 3/4", 5 1/4". 82F concrete temp.
Sure-cure cylinders are cast, 4 from each girder (South 1, Middle 2, North 1)
Cylinders cast by UMN crew as noted in cylinder schedule.

2:20 pm	Girder I finished. Cylinders for girder I are placed on form edge. Tarps are placed over girder I.
2:33 pm	Start casting Girder II (GG with microsilica) as noted by Elk River QC. Slumps: 7.0", 7.0", 6.5". 82F concrete temp.
3:47 pm	Girder II finished. Cylinders are placed on form edge. Tarps are placed over girder II.
4:38 pm	ELKCAST.001 stopped.
5:10 pm	ELKCST2.001 started with slow scan rate for overnight (0.40 samples/sec). Duration = 13 hr.
5:30 pm	Done for the evening.

AUGUST 10, 1993 - TUESDAY

6:00 am	Sunny, 66F/58F wet. Sure-cure cylinders tested: Girder I = 8634 psi Girder II = 10424 psi (north sample)
6:10 am	ELKCST2.001 stopped.
6:30 am	Tarps are removed from girder II only. Prepare girder II for stripping of forms and wait for girder I strength to increase.
6:53 am	ELKCST3.001 started with higher scan rate (1.0 samples/sec), monitor while preparing to strip and during stripping. Duration = 6 hr 1 min.
7:00 am	Connect P3500 boxes and set on top of girder II. Cylinders to be tested at release at UMN transported to test lab. Demec strips unscrewed for form removal.
8:00 am	Sunny, 71F and humid.
8:20 am	Begin stripping girder II, starting with north end (D).
8:30 am	Sure-cure break of girder I = 8873 psi, still not to strength.
9:00 am	Finish stripping of girder II. Place tiltmeters and level. (Note: during release, only girder II will have

tiltmeters. Use 6: 3 longitudinally, 3 transversely.)

9:30 am	Sure-cure break of girder I = 8515 psi, still not to strength. Elk River test lab (Dave Anderson): Limestone = 8440 psi 2.4 Adsorption 8630 psi GG w/micro= 8950 psi 9310 psi Cylinders were heat cure to 135F, need to check on specs and age of specimens when tested.
10:00 am	UMN test lab Limestone = 7750 psi 7697 psi GC w/micro = 8140 psi Cylinders were cured under tarps at plant overnight, expect lower strength.
10:10 am	Decision to remove tarps from girder I and prepare instrumentation for stripping. Wait for strength gain of girder I before stripping forms.
10:15 am	Tarps removed from girder I.
10:30 am	Connect P3500 boxes and set on top of girder I. Demec strips unscrewed for form removal.
11:00 am	Noticeable cracks occurring in girder II, cracks are marked in red and noted as ST (occurring after stripping). Most appear to be through the top flange and into the web. Refer to crack figures for locations.
11:15 am	Cracks continue to grow in girder II. Can watch some cracks migrate to bottom flange and new cracks form in top flange and web. Recall: girder II has had forms removed for 3 hours and strands have not been cut. These cracks are most likely due to shrinkage of concrete.
12:30 pm	Forms are removed from girder I, starting with north end (B).
12:54 pm	ELKCST3.001 stopped.
1:00 pm	Elk River test lab (Dave Anderson): Limestone = 9291 psi (24 hr) UMN tests instructed to start. Limestone = 8140 psi 8692 psi 8327 psi 8537 psi

GG w/micro= 8142 psi
8858 psi
8692 psi
8864 psi
9135 psi

1:45 pm Remaining sure-cure cylinders for girder II are tested:

South =	9749 psi
Middle =	10067 psi
Middle =	10705 psi

1:52 pm ELKREL2.001 started for monitoring during release. Duration = 1 hr 17 min.
P3500 boxes and demec points are read for initial readings.

2:20 pm Looking south at the live end, strands are released (flame cut) in the following pattern:

```
            1  2
            3  4
            5  6
            7  8
            9 10
           11 12


        46 xx  xx 45
35 36 37 38 39 xx  xx 40 41 42 43 44
34 33 32 31 30 xx  xx 29 28 27 26 25
13 14 15 16 17 18  19 20 21 22 23 24
```

Harp points were disconnected after 12 strands were cut. Strands were cut simultaneously at 3 locations, south end, center and north end.

2:38 pm All strands released.
P3500 boxes and tiltmeters read.
Demec points read.

3:30 pm Note large camber on girders.
Girder I =
Girder II=
Elk River employees very surprised by large camber.

Cracks noted and marked in black and labeled R (1 hr).
Photograph cracks.

Note: both beams bow 1-1/4" to the west.
Sunny, 89F and humid.

3:07 pm ELKREL2.001 stopped.

4:45 pm No additional cracks noted.

5:10 pm ELKREL3.001 started to monitor beams overnight. Duration = 13 hr 18 min.

5:30 pm Done for the evening.

AUGUST 11, 1993 - WEDNESDAY

6:00 am Sunny, clear, 71F.
Note each beam bowed 1/4"-3/8" to the west.
Read camber.

6:28 am ELKREL3.001 stopped.

8:15 am LIFTII2.003 started to monitor the lifting of girder II. Duration = 23 minutes.
Cracks inspected, new cracks from overnight marked in red and labeled R (15 hr).

8:20 am Lift II, hang for 5 minutes and set down. Read camber.

8:38 am LIFTII2.003 stopped.
Disconnect cabling. NO additional cracks due to lifting.

8:48 am LIFTII2.002 started. Power fault. Stopped at 9:47 am.

9:20 am Girder II relocated in yard. NO additional cracks due to handling.

10:04 am LIFTI2.004 started. Duration = 48 min.
Prep girder, tiltmeters calibrated.

10:30 am Lift I, hang for 5 minutes and set down. Read camber.

10:52 am LIFTI2.003 stopped.
Disconnect cabling. NO additional cracks due to lifting.

11:30 am Girder I relocated in yard. NO additional cracks due to handling.

1:00 pm Relocate van.

Reconnect cables.
Read demec points, camber and P3500 boxes.

3:47 pm ELKPREP.001 started, to monitor changes in girder while in prep area. Duration = 41 hr 24 min.

4:00 pm Done for the evening.

AUGUST 12, 1993 - THURSDAY

xx Read demec points, camber and P3500 boxes.

AUGUST 13, 1993 - FRIDAY

xx Read demec points, camber and P3500 boxes.

9:11 am ELKPREP.001 stopped.

11:06 am ELKPREP.002 started. Duration = 70 hr 30 min.

11:10 am Additional cracks in END D noted. Marked in blue and labeled 4 DAYS.

AUGUST 14, 1993 - SATURDAY

xx Read demec points, camber and P3500 boxes

AUGUST 16, 1993 - MONDAY

xx Read demec points, camber and P3500 boxes

9:42 am ELKPREP.002 stopped.

10:51 am ELKPREP.003 started. Duration = 47 hr 30 min.

AUGUST 17, 1993 - TUESDAY

xx Read demec points, camber and P3500 boxes

AUGUST 18, 1993 - WEDNESDAY

10:21 am ELKPREP.003 stopped.
Read P3500 boxes.
OPTIM and P3500 boxes disconnected.
Read demecs, camber.